Displacement Rate Effects on the Residual Strength of Soils

by

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ERRATA

Page iv, line 5:	"pressures" should read "pressures.".
Page iv, line 20:	"pressures" should read "pressures.".
Page iv, line 22:	"displacement" should read "displacement.".
Page vi, line 7:	"Test" should read "Tests".
Page vi, line 9:	"Test" should read "Tests".
Page 4, line 13:	"will be not be" should read "are not".
Page 6, line 29:	"Tchalenco" should read "Tchalenko".
Page 7, line 5:	"jointed" should read "joined".
Page 7, line 7:	"Tchalenco" should read "Tchalenko".
Page 13, line 14:	"a" should read "an".
Page 19, line 15:	"cm/sec" should read "cm/min".
Page 21, line 2:	"tent" should read "tend".
Page 21, line 30:	"matrix." should read "matrix,".
Page 41, line 2:	"Figure 2.4.5" should read "Figure 2.4.4".
Page 42, line 2:	"Figure 2.3.6" should read "Figure 2.4.6".
Page 49, line 24:	"crashed" should read "crushed".
Page 50, line 2:	"sandy silty" should read "sandy silt".
Page 60, line 17:	"displacement" should read "displacements".
Page 64, line 2:	"have be" should read "have to be".
Page 64, line 25:	"hopped" should read "hoped".
Page 66, line 20:	"were" should read "where".
Page 66, line 28:	"groves" should read "grooves".
Page 67, line 23:	"platted" should read "plated".
Page 94, line 13:	"chapter" should read "Chapter".
Page 94, line 16:	"contend" should read "content".
Page 97, line 6:	"became" should read "become".
Page 99, line 6:	"to" should read "by".
Page 101, line 6:	"Test" should read "Tests".
Page 104, line 17:	"contend" should read "content"
Page 112, line 21:	"Test" should read "Tests".
Page 114, line 18:	"of rate" should read "of the rate".
Page 116, line 11:	"a" should read "an".
Page 125, line 21:	"is" should read "has".
Page 127, column 7:	"oscillating" should read "undulating".
Page 128, column 7:	"oscillating" should read "undulating".
Page 160, line 26:	"crashing" should read "crushing".
Page 162, line 2:	"extend" should read "extent".
Page 164, line 17:	"Thus, the" should read "Thus, as the".
Page 166, line 9:	"Tests" should read "Test".
Page 171, line 11:	"part these" should read "part of these".
Page 202, Fig. 6.3.38:	"oscillating" should read "undulating".
Page 206, Fig. 6.3.46:	"oscillating" should read "undulating".
Page 233, line 12:	"6.5.1.b" should read "6.5.1.d".
Page 233, line 23:	"10 %" should read "12 %".
Page 238, line 5:	"Tchalenco" should read "Tchalenko".

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Abstract

Investigations on the effects of fast rates of displacement, such as those induced by earthquakes, on the behaviour and residual strength of soils have been handicapped by the limitations of the ring shear apparatus. Loss of soil through the gap of the confining rings of this apparatus alters the material of the shear zone and limits the duration for which fast rates of displacement can be sustained. In order to minimise soil loss and prolong fast shear testing, a new set of confining rings was designed and manufactured.

The new confining rings are described in this thesis and their evaluation procedure is presented. It is shown that the residual strength measured with the new confining rings varies by 0 to 2 % from the residual strength measured with the original confining rings. More importantly, the use of the new confining rings results in as little as seven times less soil loss than the original rings. It is concluded that the new confining rings improve substantially the capabilities of the ring shear apparatus, particularly to investigate the behaviour of soils under fast rates of displacement.

The experimental programme which was followed in this investigation involves the shearing of remoulded samples of plastic and non-plastic soils at fast rates of displacement. In analysing the test results, the *fast residual strength* (*i.e.* the steady residual strength measured at fast rates of displacement) is compared with the *slow residual strength* (*i.e.* the residual strength measured at 0.09 mm/min). It is concluded that in the absence of an undulating shear surface all plastic soils exhibit an increase in strength and their fast residual strength may be as much as 250 % higher than their slow residual strength. Moreover, all non-plastic soils exhibit a decrease in strength and their fast residual strength may be as little as 10 % lower than their slow residual strength. In the presence of an undulating shear surface all submerged plastic soils exhibit a decrease in strength and their fast residual strength may be as little as 78 % lower than their slow residual strength. Furthermore, it is indicated that the fast residual strength of all plastic soils which exhibit an increase in strength decreases with increasing normal stress and reaches a maximum value when the rate of displacement is about 1.5 cm/sec. Finally, it is suggested that the stress history and the direction of shearing have no effect on the behaviour of soils during fast shear testing and no relation between the magnitude of the increase in fast residual strength and the plasticity index of soils is observed.

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TERMINOLOGY

Apart from the terms which are defined as they appear in this thesis, the following general terminology applies:

Fast, or **Undrained Rate of Displacement**: rate of displacement which does not allow dissipation of pore pressures

Fast, or **Undrained Shearing**: test in which the sample is sheared at fast rates of displacement.

Fast Residual Strength: steady strength achieved during fast shearing of samples which have previously reached their slow residual strength.

IC/NGI Ring Shear Apparatus: ring shear apparatus designed in collaboration between the Soil Mechanics Section of Imperial College and the Norwegian Geotechnical Institute.

New Confining Rings: confining rings designed and manufactured for the purposes of this investigation.

Original, or **Old Confining Rings**: confining rings associated with the original design of the IC/NGI ring shear apparatus.

Post-threshold Strength: strength observed during fast shearing and mobilised within 20 cm after the threshold strength has been established.

Slow, or **Drained Rate of Displacement**: rate of displacement which allows dissipation of pore pressures

Slow, or **Drained Shearing**: test in which the sample is sheared at slow rates of displacement

Slow, or **Drained Residual Strength**: minimum post-peak strength of soils achieved during slow shearing.

Threshold Strength: strength observed during fast shearing and mobilised at zero shear displacement.

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CHAPTER ONE

INTRODUCTION

1.1 Background and Aims of Research

The sudden decrease in soil strength on the application of earthquake loading has been considered as the cause of several catastrophic landslides which have been recorded in history. Therefore, the effect of varying rates of displacement on the residual strength of soils has been the focus of several preceding investigations. In the years which followed the detailed examination of the residual strength concept by Skempton (1964), most investigations concentrated on determining the range of displacement rates within which the residual strength remains unaffected, with a view to employing the maximum rates possible in the laboratory and, therefore, accelerate the testing procedure. Since 1981, investigations have concentrated on the effect of high rates of displacements, such as those encountered in earthquakes or rapid landslides on the residual strength. In particular, extensive research on this subject has been conducted in the Soil Mechanics Section of Imperial College by means of the ring shear apparatus.

Lemos (1986) examined the effect of high, earthquake-induced, rates of displacement on the strength of samples from the Kalabagh (Pakistan) dam site in which a shear surface had already been established. He concluded that the increase in the rate of displacement is associated with an instantaneous increase in strength which is followed by a drop to below, above or around its residual level. He speculated that the soil strength will stabilise below, above or around its residual value depending on the shear mode of the sample at drained rates of displacement and the amount of clay which is present in the soil under investigation.

Tika (1989) continued the investigation on the phenomena observed by Lemos, and presented five types of variation of the residual strength with increasing rates of displacement. She argued that the behaviour of soils upon fast shearing does not depend only on their drained shear mode and their index properties, but also on the grading of

their granular phase. Furthermore, she described the general characteristics of the soils which exhibit each of the five types of behaviour upon fast shearing. Finally, she concluded that any pore pressure which may be generated during rapid loading does not affect significantly the test results.

However, previous investigations using the IC/NGI ring shear apparatus have been handicapped by its limitations. The excessive soil loss which has been observed through the gap of the confining rings of the apparatus limited significantly the duration of shearing at high rates of displacement (1 to 10 cm/sec), and it was suggested that a more comprehensive picture of the behaviour of soils upon fast shearing would emerge if the high rates of displacement could be sustained for a longer duration.

The first aim of this research was to design and manufacture a modification of the IC/NGI ring shear apparatus which would enable prolonged shearing at high rates of displacement. Furthermore, the modified system had to be evaluated both for its compatibility with the existing one and for its efficiency in controlling the amount of soil loss. Once the evaluation of the modified design had been completed successfully, an experimental programme was proposed to investigate the different behaviour of various soil types upon prolonged fast shearing. Thus, the second aim of this research was to establish a comprehensive model for the fast-shearing behaviour of soils related to their physical properties and the environment in which they have been tested.

1.2 Layout of Thesis

This thesis consists of seven chapters. Following this introductory chapter, the second chapter contains the literature review which is divided in four subject areas. The topics which have been considered are the drained residual strength of soils, the ring shear apparatus, the effects of the rate of displacement on residual strength, and the theories which have been advanced in order to explain the flow of particulate materials.

Chapter 3 contains a description of the apparatus and instrumentation which have been used in this investigation, the testing methods and procedures which have been employed in the course of this research, formulae for the calculation of the test results, and the classification tests which have been conducted.

Chapter 4 contains a detailed presentation of the modified design of the IC/NGI ring shear apparatus, and the procedure for evaluating the efficiency and compatibility of this modified system.

Chapter 5 contains a summarised description of the tests which have been conducted on the various soil types. The tests have been divided according to the associated soil type and the testing procedure which was followed.

Chapter 6 contains the analysis and the discussion of the experimental results. For the purpose of analysing the results, the tests have been divided in two categories: those which were conducted on plastic soils, and those which were conducted on non-plastic soils. Furthermore, the discussion concentrates on the rate of displacement, shear displacement and pore pressure effects, the rate-dependent characteristics of soils and the effects of pre-shearing and reversing the direction of shear on the measured strength.

Finally, Chapter 7 contains the conclusions of this investigation, the practical implications of the results, and the recommendations for further research on this subject.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This chapter focuses on previous investigations relevant to the subject of the effect of varying rates of displacement on the residual strength of soils. The discussion is divided into four categories. Literature relevant to the residual strength of soils is presented first, and is followed by introducing the various ring shear devices which have been used to assess the shear behaviour of soils at large displacements. The findings of previous researchers on rate of displacement effects on the undrained soil strength is then presented with emphasis on investigations involving dynamic loading and large shear displacements. Finally, existing theories of the behaviour of soils on undrained shearing and of the flow of particulate materials is also presented. The terminology and units used in each work which has been reviewed will be not be altered.

2.2 Residual Strength of Soils

The post-peak behaviour of soils has received attention from the early days of soil mechanics. Haefeli (1950), referring on his earlier work (Haefeli 1937, 1938) suggested the use of the ring shear apparatus in experiments on the *residual shearing stress* of snow and clay which he defined as the friction which remains in the shear surface after shearing is terminated. This he distinguished from the minimum post-failure shear strength. Haefeli argued that the *residual shearing stress* represents a static condition that is not associated with any additional vertical or horizontal movement after the cessation of shearing and it involves relieved pore pressures. Furthermore, he acknowledged that the *residual shearing stress* is associated with the presence of sliding surfaces and with a loss of the cohesion intercept of the strength envelope and that it should be considered in cases of progressive failure.

Skempton (1964) presenting results from slow, drained, shear box tests showed that when shearing is continued after the peak strength of the sample has been mobilised, the resistance decreases. The limit to what he referred to as *"strain softening"* he defined as the *"residual strength"* of a sample which is subjected to large shear displacements. In analysing the available experimental data Skempton concluded that the cohesion intercept of the strength envelope decreases to a value close to zero as the sample approaches its residual state, and that the residual strength of a sample can be best expressed by the following equation:

$$s_r' = \sigma' \tan \phi_r' \qquad (2.1).$$

He suggested that the decrease in sample strength from its peak to its residual value is partly due to an increase in the water content within the soil matrix which is involved in shearing, and partly due to the orientation of clay particles which takes place upon unidirectional shearing. Moreover, Skempton argued that the residual strength of a clayey soil may be considered independent of its stress history, and that decreases with increasing clay fraction. Ultimately, he added, the residual strength of a pure clay would not be different from the interparticle friction of its clay mineral (ϕ_μ). In cases where the particles involved cannot became oriented, the residual angle ϕ_r' can be given approximately by the following equation (Caquot, 1934 as presented by Skempton, 1964), assuming constant volume shearing:

$$\tan \phi_r' = \frac{\pi}{2} \tan \phi_\mu \qquad (2.2).$$

Finally, in the general case where the soil consists of a mixture of platy (clay) and roughly spherical (silt or sand) particles, the residual strength would be above ϕ_μ since the presence of the rotund particles would interfere in the orientation of the platy particles and they would increase the shearing resistance. Skempton associated the decrease of the sample strength from its peak value to residual with a progressive failure mechanism which is ceased only when all particles involved in shearing have become oriented.

Skempton and Petley (1967) presented several case records concerning the strength of overconsolidated clays along structural discontinuities. They emphasised that the soil strength in *landslides and tectonic shear zones* must be very close to residual and they considered the stages through which a clay sample under simple shear passes until it

reaches its residual state. The first stage, they suggested, involves a *continuous non-homogeneous strain*, followed by the formation of *'Riedel shears'* which, with further movement, develop into *'displacement shears'*. As shearing continues, these are connected to form an *undulating 'slip surface'* which becomes more planar with further movement. This sub-planar shear surface involves particles which have reached their maximum degree of orientation and, thus, their residual strength. Tests carried-out on samples with already existing *'slip surfaces'* showed that the peak strength has been minimised and the residual strength is attained at relatively small shear displacements. Finally, presenting a summary of results from tests on brown London clay, Skempton and Petley indicated that there is strong evidence to suggest that at low normal stresses the residual strength envelope is non-linear.

Kenney (1967) investigated the influence of the mineral composition of soils on their residual strength. He postulated that, since soils are mixtures of different minerals, their residual strength might be controlled by the different minerals of which they consist. Direct shear tests on undisturbed samples were initially conducted from which he concluded that the irregular shape of the slip surfaces often lead to increased apparent shear resistance and therefore over-estimation of the residual strength of soils. The pre-cut sample test technique was then used in order to obtain more reproducible results. He reaffirmed that remoulded samples can be successfully used in determining the residual strength of soils. From his test results on natural soils Kenney concluded that residual strength is independent of plasticity, clay fraction and water content and that it depends on the mineral composition of such soils. Kenney extended his investigation on pure minerals and indicated that the residual strength of massive-type minerals is independent of particle size, but dependent on particle shape since angular mineral particles exhibit a higher residual strength than rounded ones. Furthermore, he argued that the residual strength of layer-lattice minerals depends mainly on the system chemistry and increases with increased ion concentration in the pore fluid and with the formation of interparticle bonds.

Morgenstern and Tchalenco (1967) studied the microscopic features developed within a kaolin sample subjected to direct shear. They prepared identical samples from a slurry and sheared them under drained conditions. Each test was terminated at a different stage of shearing, the samples were impregnated with Carbowax 6000 and their

shear-induced fabrics compared [Fig. 2.2.1]. It was suggested from the results of their investigation that ***displacement discontinuities*** appear at the edges of the shear box after the sample has reached its peak strength. These discontinuities were developed at high angles to the horizontal and, as shearing progressed, they extended through the middle two thirds of the sample until they eventually jointed -towards the end of the travel of the shear box in approaching the residual strength- to form a ***'wavy surface'***. This wavy continuous plane of shearing, Morgenstern and Tchalenco argued, was achieved by the mobilisation of displacement discontinuities which lay in the direction of shearing and at low angles to it. Furthermore, at the end of the travel of the shear box, additional discontinuities were developed aligned at high angles to the direction of shear and which were not part of the shearing surface. It was speculated that the ***ultimate structure*** which governs the residual strength of soils was not achieved in any of the tests described in their investigation, and that the wavy slip surface which had been observed at the end of travel of the shear box would eventually become planar with further shearing. Also, they indicated that high stress concentrations may be encountered at the loading sides of the shear box and that a non-uniform stress distribution may have led to a non-homogenous strength distribution.

Ramiah *et al* (1970) conducted direct shear tests on a silty clay which had been pre-treated with flocculant and dispersant agents. They investigated the influence of chemicals, stress history and initial water content on the residual strength of the samples tested. The results from this series of tests suggested that the sample which had been pre-treated with the flocculant agent had a higher residual strength by 2° to 5° than the sample which had been pre-treated with the dispersant agent. They, reasoned that the high residual strength of the flocculated state is due to the increased ion concentration of the pore fluid and the fact that this state has more interparticle contacts than the dispersed. Furthermore, they concluded that the stress history and the initial water content appeared to have no effect on the residual strength of the samples tested.

Smart (1970) employed a deterministic approach in order to define the dependence of the residual strength of soils on the clay content. He assumed that once a distinct failure surface has been established in a soil matrix, the shearing mechanism can be analysed by utilising a rigid block model. Smart concluded that if c is the clay fraction *i.e.* the volume of clay divided by the volume of solids, ϕ_c is the clay interparticle residual

angle, ϕ_s is the sand interparticle residual angle and ϕ_m is the clay-sand residual angle, and if the direct stress is distributed uniformly over the failure plane, the residual angle of shearing resistance of the soil (ϕ_r) can be estimated by the following equation:

$$\tan \phi_r = c^2 \tan \phi_c + 2c\,(1 - c) \tan \phi_m + (1 - c)^2 \tan \phi_s \qquad (2.3).$$

Furthermore, Smart argued that if the clay content was based on the volume of clay plasma (defined as: $c^* = \frac{Vc+Va}{Vs+Vc+Va}$, where V_c, V_s and V_a are the volumes of clay solids, sand solids, and water within the clay plasma, respectively) then a closer approximation of the experimental results can be achieved.

Cullen and Donald (1971) conducted a thorough investigation of the determination of the residual strength by means of the direct shear apparatus, and of the effects of various parameters on the experimental results. They indicated a substantial decrease in residual strength with a moderate increase in moisture content and they suggested that a correction to a *standard moisture content* of 22% should be applied upon analysing the results. Furthermore, they argued that the cohesion intercept of residual strength may not always be zero, and that the technique of stage testing, where the same sample is sheared to residual at subsequently increased normal stresses, may be used to produce valid results at substantially less time.

La Gatta (1971) presented the results of his investigation on the residual strength of soils by means of a then newly developed ring shear apparatus. He defined as residual strength the drained shearing resistance of a sample undergoing *continuous deformation under a constant state of effective stress and at a constant void ratio* and he repeated the suggestion that the residual and *critical* states are identical. La Gatta indicated that the reduction in shear resistance from peak to the residual value is a result of bond breakage and particle orientation for normally consolidated soils, whereas for overconsolidated soils it is also a result of an increase in void ratio with shearing. Furthermore, he suggested that when the shear strength is plotted versus the shear displacement in order to estimate the residual strength of a sample, the logarithmic scale should be employed in order to avoid over-estimating the residual strength.

Townsend and Gilbert (1973) reported on the residual strength of various clay shales. In comparing results from ring shear and direct shear tests, they indicated that there is little difference between the values attained by the two methods for hard, highly overconsolidated soils, but this is not necessarily the case for softer materials.

Furthermore, they argued that the residual strength of soils is independent of the sample preparation procedures.

Kenney (1977) reported additional findings from his continuing investigation on the residual strength of mineral mixtures. He suggested that the residual strength of a mineral mixture depends on its composition and the chemical state of the clay mineral. He argued that the residual strength should be inversely proportional to the volume ratio of a sample (r_{vc}) which he defined as the ratio between the volume of clay minerals and water and the total volume of the sample. Furthermore, he argued that in a mixture of clay mineral particles, water, and quartz (massive mineral) particles, the clay particles and water form a cohesive matrix and the quartz particles are discretely included within this matrix. In soils where the cohesive matrix exceeds 40% or 50% of the total volume, the residual strength is controlled by the clay mineral. When the relative volume of the massive minerals increases, the residual strength of the soil also increases, as the quartz particles form a continuous structure which resists shearing and the cohesive matrix does not contribute substantially to the residual strength.

Lupini *et al* (1981) presented a thorough investigation of the shearing mechanisms associated with the residual state of soils. They performed ring shear tests on mixtures of medium/fine sand and powdered mica, Happisburgh clay mixed with London clay, and quartz-silt mixed with sodium bentonite. Results from the tests on sand-mica mixtures indicated that increasing normal stress may result in closer packing of the granular particles, interlocking, and an increase in measured shear resistance. The opposite effect may be observed in the parts of the shearing surface where sliding occurs. Lupini *et al* also suggested that the residual strength decreases with decreasing particle size, is higher for dry than saturated materials, and for single-mineral soils appears to be higher than the mineral's interparticle friction (ϕ_μ). The observation of post-failure structures in Happisburgh-London clay mixtures revealed that for samples with a clay fraction below 30% no distinct zone of oriented clay particles is formed, and that the residual strength is close to the normally consolidated peak strength. Furthermore, the post-failure structures of samples with a clay fraction above 35% or 40% contain distinct shear zones of oriented clay particles which may be bounded by continuous undulating shear surfaces and which are associated with low residual strengths [Fig. 2.2.2]. Similar results were observed from the quartz-bentonite mixtures. Lupini *et al* postulated that

the three residual shearing modes are the *turbulent*, the *sliding* and the *transitional* [Fig. 2.2.3]. The *turbulent mode* operates in samples with a high content of rotund particles or with platy particles with high interparticle friction, and is associated with high residual strengths which depend on the particle shape and packing. The *sliding mode* operates in samples with a high content of platy particles of low interparticle friction, which form distinct shear surfaces, and is associated with low residual strengths which depend on mineralogy and pore water chemistry. In cases where a shear surface is formed but where enough rotund particles exist to interfere with sliding on a single plane, the residual strength appears to be higher. The *transitional mode* involves discontinuous sliding surfaces and *'pockets of soils operating in the turbulent mode'*, exhibits characteristics of both previously mentioned shearing mechanisms and is very sensitive to small changes in soil gradation. Lupini *et al* suggest that since the residual strength of soils depends largely on the presence of rotund particles within the soil matrix, it would be more closely associated with the void ratio of the granular phase of the soil e_g, where:

$$e_g = \frac{\text{volume of platy particles and water}}{\text{volume of rotund particles}} \qquad (2.4)$$

Finally, in reference to correlations between the residual strength and index tests of natural soils [Fig. 2.2.4], they indicate that such correlations would be expected to lead to acceptable results only in cases where clay mineralogy and pore water chemistry are constant.

Pyles (1984) suggested the use of the vane shear apparatus for measuring the residual strength of soils and proposed the term *"ultimate undrained residual strength"* to describe the minimum shear resistance associated with large landslide movements. He also argued that the residual strength of soils decreases with increasing water content but *'is relatively constant with respect to liquidity index'*.

Skempton (1985) summarised the most important developments in the subject of residual strength during the period which followed his Rankine Lecture in 1964. He emphasised the difference between the *fully softened* or *critical state* value, which is the first stage of post-peak drop in drained shear strength caused by an increase in water content, and the residual value which is the second stage of the drop in strength, caused by the reorientation of clay particles parallel to the direction of shearing [Fig. 2.2.5]. He postulated that for overconsolidated intact clays the post-peak drop is attributed to both

increase in water content and particle reorientation, whereas, for normally consolidated clays this drop is caused entirely by particle reorientation. Skempton indicated that shear displacements exceeding 100 mm, and as high as 500 mm, may be necessary to ensure residual conditions in the ring shear apparatus, though, displacements 50% or 80% smaller may be sufficient to achieve shear strengths acceptably close to the residual value ($\phi_r + 1°$). Furthermore, he argued that the residual strength estimated by slip surface tests is in close agreement with the back analysis results from reactivated landslides, and that the ring shear apparatus may lead to an over estimate of the residual strength by 1° or 2°. Finally, Skempton suggested that on comparing the residual strengths of two clays one should concentrate on the *'standard'* normal pressure of 100 kPa in order to avoid complications due to *the non-linear relation between residual strength and normal effective pressure*.

Bromhead and Dixon (1986) indicated that the field residual strength is depended on the porewater pressures acting on the slip surface. They argued that in assessing the long-term stability of a slope the effect of porewater changes is much more important than the accuracy with which the ring shear apparatus can estimate the residual strength in the laboratory.

Maksimovic (1989) reported on the non-linearity of the residual strength envelope, and argued that the most convenient way of representing that envelope is:

$$\phi'_r = \phi'_{B,r} + \Delta\phi'_r / (1 + \sigma'_n / p_{N,r}) \qquad (2.5)$$

where $\phi'_{B,r}$ is the residual angle at high confining stresses, $\Delta\phi'_r$ is the maximum angle difference and $p_{N,r}$ is the median angle normal stress. Furthermore, he suggested that if three or more residual test results are available, at the widest range of normal stresses possible, the full residual strength envelope can be defined and the previous equation can be rewritten in the following manner:

$$a + b\sigma'_n = \sigma'_n / (\phi'_0 + \phi') \qquad (2.6)$$

where $a = p_N / \Delta\phi'$, $b = 1 / \Delta\phi'$, $\Delta\phi' = \phi'_0 - \phi'_{B,r}$, and ϕ'_0 is the residual angle at zero normal stress.

Moore (1991) reported on the importance of mineralogy and pore water chemistry on the residual strength of natural clays. He suggested that weathering decreases the residual strength of soils, and that the use of distilled water for remoulding

samples in the laboratory leads to higher residual strengths than when natural water is used.

Yagi *et al* (1991) presented ring shear test results which suggest that the reason the residual strength estimated by the back analysis of an existing landslide is often greater than the residual strength measured in the laboratory may be that shear box tests promote the development of slickensides which are not always present in the field.

2.3 Ring Shear Apparatus

The ring shear apparatus has for long been regarded as the most effective way of measuring the residual soil strength because of its ability to permit the large unidirectional shear displacements necessary to establish complete particle orientation.

The original idea for the use of a ring shearing apparatus to investigate *plastic flow of soils* was introduced by Hvorslev (1936). He presented a simplified drawing of his then newly developed *load controlled* apparatus [Fig. 2.3.1] which was constructed so that the confining rings could be either *stationary throughout the test* (*"fixed rings"*), or free to rotate (*"free rings"*), or fixed to the top piston which transmitted the vertical load on the sample (*"restrained rings"*). Furthermore, for calculation purposes he made the following assumptions: the stresses on the vertical and horizontal sides of the sample are uniformly distributed, there is no distortion of any horizontal plane throughout the test, and the friction between the sample and the confining rings is negligible. Hvorslev concluded that the *ring shearing apparatus* should be adopted upon investigating *plastic flow of soils* and that the minimum soil resistance associated with large shear displacements could be correctly estimated only with this type of apparatus. However, the particular significance of the ring shear apparatus as the only device able to shear the sample for large unidirectional displacements was recognised after Skempton (1964) reintroduced the concept of the residual strength of soils.

La Gatta (1970) presented a detailed description of the first of a new generation of *strain-controlled* ring shear devices [Fig. 2.3.2]. This *rotation shear machine* comprised a *shearing unit*, a *vertical loading system*, a *torque application system*, and a *moment measuring system*, and was able to shear either disc-shaped specimens with 7.11 cm diameter, or annular specimens with 7.11 cm outer diameter and 5.08 cm inner

diameter, while the specimen height could vary from 0.1 cm to 2.5 cm. The shearing unit was designed to allow the confining ring to be fixed to the turn-table (*fixed ring configuration*), or be independent (*floating ring configuration*). The normal load was applied by means of a loading yoke and a counter-balanced lever system. Rotation of the lower part of the shearing unit was governed by means of a gear motor and a worm gear speed reducer. The shear forces were measured by a set of force transducers located on the top plate. In order to force failure at the centre of the specimen, La Gatta initially used a split confining ring. However, a fixed clearance of 0.005 in. led to excessive soil loss at large shear displacements and it was suggested that the clearance should be reduced to zero. Related problems such as side friction between the confining ring and the sample, and shear friction developed on the mating face of the two halves of the confining ring were addressed but not eliminated. It was observed that the shear surface was not always in the middle of the sample and that the measured residual strength of a sample which failed in the middle did not deviate by much from that of a identically prepared sample which failed near one of the porous platens. Therefore, in order to simplify the design and the testing procedure, a solid confining ring was used. La Gatta investigated the sources of error associated with the rotation shear apparatus and recognised that the sample extrusion from the confining ring upon shearing was the most significant one. He indicated, however, that the non-uniform distribution of the normal stress resulting from soil extrusion leads to under-estimation of the residual strength, and that with disc-shaped specimens the error could be as large as 12%, whereas in cases of annular specimens the error could be insignificant. Furthermore, La Gatta estimated that errors introduced by factors such as failure occurring near the loading platen, inaccuracy of the lever system, side friction, and friction between the rotating and the stationary parts may lead to an over-estimation of the residual strength by 0.1 % to 1.4 % which was considered insignificant.

Bishop *et al* (1971) reviewed and presented the various types of ring shear devices which had been developed previously [Fig. 2.3.3]. They suggested that the main reason why the ring shear had not enjoyed any popularity, in spite of its profound advantage in measuring soil strength at large shear displacements, was that it was a complicated item of equipment in its construction and operation. Also, they argued that the high local strain concentrations, the uncertainty about the direction of principal

stresses, and the additional friction which develops between the upper and the lower confining rings add to the complexity of the system. Furthermore, they introduced a new design [Fig. 2.3.4] which they claimed provided a better control of the gap between the upper and lower confining rings, and had the ability to measure the *side friction* which develops between the sample and the inner walls of the rings. Remoulded or undisturbed annular samples of 6.0 in. outer diameter, 4.0 in. inner diameter, and 0.75 in. thickness could be loaded to a maximum *nominal* normal stress of 142 lb/in^2 and sheared at rates of displacement which varied between 0.00001 mm/min and 1000 mm/min. Even though their basic assumption that the proving ring, which is part of the gap controlling mechanism, is stiff enough to keep the gap size constant upon shearing may be subject to criticism, the fact remains that the variation in the net normal load acting on the sample during shearing can be considered, for all practical purposes, negligible. Furthermore, for the purpose of calculating the test results, Bishop *et al* evaluated all the previously proposed normal and shear stress distributions [Fig. 2.3.5] and their effect on the estimated residual strength. They concluded that the assumption that the normal and shear stresses are distributed uniformly *across the plane of relative rotary motion* is the most realistic choice and leads to deviations less than $\pm 2^{\circ}$ when compared with the most extreme non-uniform distributions. Comparing their *'new'* ring shear apparatus with the multiple reversal shear box in estimating the residual strength of soils, they postulated that the shear box fails to simulate field conditions of large unidirectional shear movements and leads to large scatter in the results. Finally, after comparing various results, they concluded that this apparatus enables the *"correct measurement of the residual strength"*.

Mandl *et al* (1977) developed another ring shear apparatus [Fig. 2.3.6.a] for the purpose of investigating the structure of shear zones in soils. The annular sample in this equipment had an inner diameter of 35 cm, an outer diameter of 45 cm and a height of 4.6 cm. The normal load was applied by gas pressure and the lower part was rotated by means of a constant speed motor. The normal and shear stresses were measured by means of "Cambridge load cells" and photoelastic meters [Fig. 2.3.6.b]. The gap between the rotating and stationary parts of the apparatus was maintained closed at all times by means of springs and adjustment screws in order to prevent soil extrusion.

Bromhead (1979) presented a design for a ring shear apparatus which he described as *"simple, robust and inexpensive"* with potential to be used in commercial practice [Fig. 2.3.7]. An annular soil sample of 7 cm and 10 cm inner and outer diameters, respectively, and 0.5 cm height was sheared using a variable speed motor which was connected to the base plate and lower platens of the apparatus. The shear surface was formed near the top platen which was artificially roughened in order to prevent slipping to occur along the soil/metal interface. Similar sample preparation and testing techniques to those described by Bishop *et al* (1971) were used, but the lesser height of the sample reduced the consolidation time and accelerated the testing procedure. Bromhead noted that the ring shear apparatus causes progressive failure on the sample and thus, leads to an unreliable estimate of the peak strength. As far as the sources of error associated with his apparatus *i.e.* inaccuracies in load measurements, friction caused by soil squeezed out of the annular chamber and becomes trapped between the confining rings and the upper platen, side friction between the sample and the confining rings, as well as the possibility of tilting of the top platen causing non-uniform normal stress distribution, and excessive soil loss, Bromhead suggested that they have minor effect on the results and, therefore, may be ignored.

Anayi *et al* (1989) indicated that the failure surface of a specimen tested with the Bromhead-ring shear apparatus forms very close to or at the interface between the soil and the loading platen and that the application of shear stress depends on the adhesion between the porous bronze rings and the specimen. Furthermore, upon testing Lias clay samples they observed that as shearing progressed an unbalance of the torque measuring mechanism developed which often led to *"friction on the centre pivot and tilting of the loading platen"* and undulation of the shear surface. In order to minimise the problem, they suggested that uniformly distributed vanes should be introduced on the top and bottom platens, the sample height should be increased from 5 mm to 10 mm and the sample should be placed in the confining chamber in two parts of 5 mm height, so that a flat shear surface may be encouraged. Finally, they suggested that this modification minimises soil extrusion and the errors associated with it.

Stark and Vettel (1992) presented a critical review of the Bromhead-ring shear apparatus. They suggested that the wall friction between the specimen and the confining rings affects greatly the estimated residual strength, and increases as soil is extruded and

as the loading platen settles during shearing. Furthermore, they argued that this may lead to over-estimating the residual strength by 25 %. They proposed a testing procedure which involves the addition of remoulded soil, on the top of the sample, and the re-consolidation of the added and existing soil, at the desired normal stress before shearing resumes. With this proposed procedure they concluded that the top platen will remain "flush" with the top of the sample container. This technique, however, requires only one test per specimen and is thus time consuming. Finally, Stark and Vettel indicated that the installation of vanes on the loading platen of the Bromhead-ring shear apparatus will result in increased wall friction and over-estimation of the residual strength.

Stark and Eid (1993) introduced a new specimen container for the Bromhead-ring shear apparatus which enables the sample to be over-consolidated and pre-cut before being sheared. Furthermore, they presented a rather complicated and test operator dependent *"modified ring shear test procedure"* which, they argued, reduces substantially the test duration and the discrepancies of experimental results.

Hungr and Morgenstern (1984) presented another ring shear design [Fig. 2.3.8] which was developed to test granular materials. This apparatus consists of a *lower box* which rotates by means of an electric motor and an *upper box* which is fixed to a stiff load cell. The gap between the upper and lower boxes is controlled by the load cell which also measures the normal load and the side friction. The normal load is applied by means of an annular-shaped, inflatable rubber membrane. The volume changes of the sample upon shearing are accommodated by the flexibility of the membrane. The shear stress is transmitted and measured by means of a strain-gauged torque arm which is fixed to the upper box. The annular sample has 30 cm and 22 cm outer and inner diameters, respectively, is 2 cm in height and it is not submerged in water. Hungr and Morgenstern suggested that this apparatus is able to shear granular samples at rates of displacement as high as 200 cm/sec, but the associated vibrations which were observed at rates above 90 cm/sec forced them to abandon fast testing for fear of damaging the equipment.

Sassa (1985) presented a high speed ring shear apparatus [Fig. 2.3.9] which has the same general characteristics with the one presented by Hungr and Morgenstern (1984). The normal load is applied by means of an inflated rubber membrane, the normal stress is measured by means of a load cell which is located below the rotating part of the

apparatus, and the shear stress is transmitted through a torque arm and measured by a load cell which is mounted on a vertical arm on the side of the shear box. Sassa suggested that this apparatus enables constant volume shearing at rates of displacement as high as 90 cm/sec. The volume changes were monitored by means of a dial gauge located at the top of the apparatus while the edges of the rings were individually machined in order to ensure a perfect fit with minimum friction and equipped with silicon rubber rings in order to prevent soil loss.

Nieuwenhuis (1991) introduced a ring shear apparatus which was developed at the University of Utrecht [Fig. 2.3.10]. Apparently, this apparatus can function either as a strain or stress-controlled system. The normal stress is applied, by means of an axial motor, and measured at three equispaced supports along the periphery of the top platen, while the lower part of the apparatus is rotated by means of a radial motor, and the shear stress measured through the vertical supports. Both motors are controlled and all stresses are monitored by a PC, while the gap is adjusted and controlled through a system of three clamps attached to the lower part of the apparatus and equipped with wheels to facilitate rotation. The sealing of the gaps between the upper and the lower confining rings is achieved by means of compressible hollow and lubricated O-rings which are kept in place by two lubricated steel rings [Fig. 2.3.11] while the friction which is introduced by this sealing system is measured in dummy sample tests and the necessary corrections are applied on soil test results. Nieuwenhuis suggested that this ring shear apparatus (*Utrecht device*) enables the application of either positive or negative pore pressures on the sample through the central column, inside the confining rings. However, a comprehensive presentation of this latter part of the apparatus is not provided.

Yagi *et al* (1991) introduced two types of ring shear devices which were developed for the investigation of stability analyses on landslides. Although a brief description of the new equipment was presented, it was indicated that the difference between the two types is concentrated on the shearing conditions under which the sample is subjected. The *B-type* ring shear apparatus [Fig. 2.3.12] comprises of an annular shear box which encourages shearing along a plane, while the *S-type* ring shear apparatus [Fig. 2.3.13] imposes uniform shearing conditions throughout the thickness of the sample. Furthermore, in the *B-type* apparatus, the normal load is imposed by means of an air cylinder, the sample has a height of 20 mm and is not submerged in water, the shear

stress is transmitted through a torque arm and measured by a single load cell which is mounted on a vertical arm, pore pressures are measured at the bottom of the specimen, and the bottom confining rings have a tooth-like edge in order to minimise soil extrusion. The *S-type* ring shear apparatus was developed because field evidence suggested that slickenside is not evident in all landslides. It appears from the diagram presented by Yagi *et al* that while the main features of the *S-type* apparatus are similar to those of the *B-type*, the sample is 10 mm high, it is submerged in water, it is supplied with water from the top, and is not confined. Finally, Yagi *et al* suggested that the pore pressure generated during shearing the samples at rates of displacement below 0.21 mm/min was insignificant, and therefore the aforementioned rate was adopted in their testing procedure.

Sassa *et al* (1991) introduced another *"high-speed high-stress ring shear apparatus"* [Fig. 2.3.14] which was developed for the purpose of investigating *the apparent friction angle during rapid loading*. The normal load is applied by means of six air pistons and the normal stress acting on the sample is measured by a load cell which is located below the apparatus table. The gap between the upper and lower confining rings is controlled with a precession of 0.001 mm by means of a servo-motor. Furthermore, the lower part of the apparatus is rotated by means of another servo-motor while the normal stress is controlled at a constant level by an air-servo valve through signals from the load cell. The shear stress is measured by a single load cell which is mounted to a vertical arm. All motors and measuring devices are connected to and controlled by a PC. A closer look at the cross section of the confining rings which was presented by Sassa *et al* indicates that a rubber edge is mounted on the upper confining ring and is always pressed onto the lower ring in order to prevent soil extrusion, while the outer rings are manufactured of transparent acrylic glass in order to facilitate the visual observation of particle movements upon shearing. The sample is 33 cm and 21 cm in outer and inner diameter, respectively, is 6 cm in height, and is submerged in water while drainage is allowed from the top and bottom by means of porous metals.

Sassa (1992) introduced yet another improved *"cyclic loading high-speed ring shear apparatus"* [Fig. 2.3.15] which is able to combine the application of cyclic shear and normal stress loading conditions. The vertical load is imposed on the top platen by an air-piston which is mounted at the top of the frame and it is measured by a load cell.

The pressure in the piston is controlled by an electric air-pressure converter which is regulated by a PC. The side friction between the sample and the upper confining rings is measured by a load cell which is located below the rotating table while the shear stress is transmitted from the sample through a torque arm and it is measured by a single load cell. The lower confining rings are rotated by means of a servo-motor and the shear displacement is measured by a *rotary transducer*. The gap between the upper and lower confining rings is controlled by means of another servo-motor while a teflon-sprayed rubber ring, 9 mm thick, is glued on the upper confining rings and constantly pressed on the lower confining rings in order to prevent sample extrusion. The sample is 31 cm and 21 cm in outer and inner diameter, respectively, its height may vary between 6 cm and 9 cm, and it is submerged in water. Drainage is allowed at the top and bottom platens. Furthermore, three needles connected to pressure cells are inserted around the shear zone of the sample in order to monitor any changes in pore pressure upon rapid loading. Sassa suggested that this apparatus is able to shear soil samples at rates of displacement which may vary between 0.0002 cm/min and 9,000 cm/sec.

2.4 Rate of Displacement Effects on Residual Strength of Soils

The effect of strain and/or displacement rate on the measured soil strength has been the focus of research mainly in an effort to accelerate commercial testing procedures and reduce laboratory costs. Therefore, the triaxial and shear box devices have been employed for most of these works. Furthermore, extremely low strain rates have been employed in order to study the thixotropic characteristic of soils or to investigate creep. However, research works which concentrated on the effect of high rates of displacement and are associated with large shear displacements are of interest in this investigation.

Horn and Deere (1962) investigated the frictional characteristics of minerals which are present in rocks and soils and are often associated with landslides. Direct shear tests were conducted on dry and saturated, smooth and roughened quartz, and phlogopite mica at rates of sliding which varied from 0.7 in/min to 6.0 in/min. They concluded that the rate of sliding has no effect on the frictional characteristics of either smooth or roughened quartz. However, they indicated that the shear strength of

phlogopite increases by 12 % in dry conditions and by 28 % in saturated conditions when the rate of sliding increases from 0.7 to 6.0 in/min.

De Beer (1967) presented ring shear test results for Boom Clay. He investigated the effect of the rate of deformation on the residual strength and he concluded that the latter decreases by 20 % when the former increases from 0.395 to 2.1 mm/min. He speculated that the apparent increase in residual strength with decreasing rate of displacement is caused by the reforming of bonds between particles within the shear matrix. Furthermore, he suggested that samples with a pre-existing shear surface exhibit higher residual strength than those which are being sheared for the first time.

La Gatta (1970) indicated that the residual strength of the soils which he tested (London clay, and Pepper and Cucaracha shales) is insignificantly affected by a decrease in the rate of displacement from 0.056 mm/min to 0.0056 mm/min. Furthermore, he added that, in the case of the Cucaracha shale, when the rate of displacement was increased from 0.0056 mm/min to 5.6 mm/min, the measured residual strength increased only by 3.5 %.

Idriss (1981) investigated the effects of large shear displacements on the undrained soil strength by evaluating the results of miniature vane, direct shear, and cone penetration tests on Bootlegger Cove clay. He postulated that large deformations often associated with earthquakes may result in complete reorientation of clay particles and changes in pore pressure, thus, establishing the residual strength under undrained conditions. He indicated that there is large scatter associated with the amount of drop in strength from peak to residual, but, nevertheless, for shear displacements less than 0.5 ft, the undrained residual strength would be around 70 % of the peak strength whereas for larger shear displacements it would be 30 %. Finally, he proposed the use of a step-function for computational purposes when calculating the undrained shear strength [Fig. 2.4.1].

Lupini (1981) completed a thorough investigation on the residual strength of soils, the results of which have also been presented by Lupini *et al* (1981). The effects of fast rates of shearing, which were as high as 177 mm/min, on the residual strength of natural soils were also investigated. He suggested that the behaviour of soils during fast shearing is controlled by their shearing mode under drained conditions. He concluded that the residual strength of soils with *sliding* shear mode would increase with increasing

rates of shearing, whereas the residual strength of soils with *turbulent* shear mode would tent to decrease with increasing rates of shearing. Lupini postulated that the residual strength of soils with sliding shear mode is a function of the interparticle friction (ϕ_μ) and therefore it increases with increasing rates of shearing because of *viscous effects*. He argued that with increasing rates of shearing the orientation of platy particles is destroyed, the shear mode changes from sliding to turbulent and the strength may reach its *constant volume* value (ϕ'_{cv}). Furthermore, he suggested that the reduction in residual strength during fast shearing of soils with turbulent shear mode may be attributed to increased void ratio of the soil matrix within the shear zone which results in a reduction of the particle interlocking.

Savage (1982) presented tests on polystyrene beads at high shear rates by means of an annular shear cell apparatus in which the whole sample was involved in shearing. He concluded that the *dynamic friction angle* (ϕ_d) depends on *the solids concentration* (ν) and the *shear-rate*. At low values of ν, ϕ_d increases with increasing shear-rate, whereas, at high values of ν, ϕ_d is relatively independent of the shear-rate. Furthermore, Savage suggested that shear strength can be divided in two parts and expressed with the following equation:

$$\tau_{total} = \tau(\nu)_{Coulomb} + \mu(\nu)\rho_p \sigma^2 \left(\frac{du_1}{dx_2}\right)^2 \qquad (2.7)$$

where: $\tau_{Coulomb}$ and μ are functions of ν, $\frac{du_1}{dx_2}$ is the mean shear rate, σ is the particle diameter and ρ_p is the individual particle mass density. The first part of the sum is the rate independent part which results from dry Coulomb friction and particle overriding, and the second is the rate dependent part. Finally, in analysing the case of simple shear flow, he concluded that the normal and shear stresses are proportional to the square of the shear rate.

Martins (1983) investigated the immediate effects of rapid loading on pre-existing slip surfaces. For the purpose of that investigation ring shear tests on weathered Panama Tuff were conducted at rates of shearing as high as 100 mm/min. He recognised the following effects of fast shearing on soil strength [Fig. 2.4.2],

i. *threshold effect* associated with zero shear displacement,

ii. *increase in strength* due to restructuring of particles within the shear matrix.

iii. *pore pressure effect* developed after slippage is initiated, and

iv. ***non-transient increase in strength*** appearing when fast shearing is followed by slow shearing and is probably due to restructuring of the soil matrix within the shear zone on fast shearing.

Hungr and Morgenstern (1984) presented a series of high velocity ring shear tests on a medium, uniform, quartz sand with rounded grains, on sand and rock flour, and on polystyrene beads. Dry, or submerged in water, samples were consolidated under normal stresses which varied between 15 and 200 kPa and they were sheared at rates of displacement which varied between 0.1 and 98 cm/sec. Hungr and Morgenstern concluded that the measured shear strength of granular soils does not depend on the rate of displacement at which they are sheared or the normal load under which they are subjected.

Bernander *et al* (1985) indicated the inability of conventional *plastic failure analysis* to explain dramatic landslides which have occurred in the past. They postulated that the *large strain strength* of soils is OCR and strain rate dependent. Furthermore, they suggested that undrained parameters and relative strain rates ought to be used when analysing rapid failure mechanisms. The shear box test results which were presented suggested that the residual strength of soils depends highly on the strain rate due to the excess pore pressure which develops in the slip surface. They concluded that experimental evidence suggests that the residual strength decreases with increasing strain rate and OCR.

Sassa (1985) suggested that, in order to investigate the steady flow of soils, constant normal stress tests ought to be conducted. He presented results from ring shear tests on glass beads which indicate that granular materials are independent of the rate of shearing, and that, the concept of *flow structure*, suggested by Casagrande as an explanation for the decreasing shear resistance during rapid loading, may not be valid. Furthermore, Sassa postulated that, as the rate of displacement increases, a greater percent of the particles which are associated with shearing floats in water, increases the specific gravity of the pore fluid, and results in the decrease of the effective stresses and the soil strength.

Lemos *et al* (1985) indicated that fast shearing increases the residual strength of soils and speculated that the decrease in measured shear strength below its static residual value which was observed in one case during their investigation was not caused by

increasing pore pressures. Having adopted the residual shearing modes introduced by Lupini *et al* (1981), Lemos *et al* presented typical results from tests which were conducted on the ring shear apparatus. The general testing procedure which was followed for this series of tests involved remoulding the sample at low water contents, consolidating under the desired normal load, shearing under drained conditions to establish the residual structure, unloading in shear and allowing for pore pressure dissipation, shearing at fast rates of displacement, unloading in shear and allowing for pore pressure dissipation, and, finally, shearing under drained conditions. The following five strengths [Fig. 2.4.3] were recognised from these tests:

(a) the *drained residual strength*,

(b) the *threshold strength* which is mobilised at zero shear displacement when undrained loading follows drained loading,

(c) the *peak strength* which is mobilised after the threshold strength during undrained loading,

(d) the *ultimate fast strength* which is the steady value of shear resistance reached during undrained loading, and

(e) the *peak strength after fast shearing* which is mobilised at zero shear displacement when drained loading follows undrained loading and pore pressure dissipation.

Lemos *et al* suggested that soils which are associated with the sliding shear mode exhibit an increase in residual strength with increasing rates of displacement, whereas, soils which are associated with the turbulent shear mode exhibit the opposite effects over the slower rates which have been used in their investigation. Furthermore, they argued that the gain in strength may be attributed to disordering of clay particles which is often facilitated by the presence of rotund particles within the clay matrix. Finally, they concluded that the possibility of increased pore pressures during rapid loading will result in a dramatic reduction of the ultimate fast strength and catastrophic movements.

Lemos (1986) presented a thorough investigation on the effects of the rate of shearing on the residual strength of pre-established shear zones in natural soils. The ring shear apparatus was employed, and rates of displacement as high as 6000 mm/min were imposed onto the soil samples. The following observations and conclusions summarise the results of this investigation:

a. The residual strength of soils which have low clay fraction and exhibit turbulent shear mode during drained shearing is not affected by increased rates of displacement [Fig. 2.4.4.a],
b. The *fast residual strength* of soils which have higher clay fraction and exhibit turbulent or transitional shear mode during drained shearing is lower than their *slow residual strength* [Fig. 2.4.4.b],
c. The fast residual strength of soils which exhibit sliding shear mode during drained shearing is higher than their *slow residual strength* [Fig. 2.4.4.c],
d. The *slow peak strength* which is measured when fast shearing is followed by slow shearing is attributed to the *disordering effect* of the former on the soil matrix of the shear zone. In cases of soils with turbulent shear mode this effect becomes insignificant,
e. The threshold strength should be considered as an intercept to the failure envelope rather than an increase of the friction angle, and
f. The increase of the *fast peak strength* above the threshold strength may be attributed to the disordering effect on the soil matrix of the shear zone.

Tika (1989) summarised and extended the works which had been presented earlier by Lupini (1981), Martins (1983), and Lemos (1986). Furthermore, she conducted additional ring shear tests on natural and artificial soils in order to validate Lemos's conclusions and develop a comprehensive theory concerning the variation of the residual strength upon increasing rates of displacement. She identified five types of that variation [Fig. 2.4.5]:

Type I involves insignificant changes of the residual strength with increasing rate of displacement and is characteristic of soils with turbulent shear mode, plasticity index less than 9 %, sand content higher than 20 %, and clay content lower than 10 %.

Type II involves an insignificant, or moderate, increase or decrease of the residual strength with increasing rates of displacement followed by a substantial decrease in strength as the rates increase further. This type of behaviour is characteristic, mainly, of soils with transitional or turbulent shear mode, and plasticity index between 9 and 21 %. However, deviations from this behaviour have been observed at high normal stresses.

Type III involves a decrease of the residual strength with increasing rates .of displacement, followed by an increase and a substantial drop of strength below its slow

residual value as the rate of displacement increases further. This behaviour is characteristic of silty clays with transitional or sliding shear mode, and plasticity index between 24 and 26 %.

Type IV involves a decrease of the residual strength with increasing rates of displacement below its slow residual level, followed by an increase above the former as the rate of displacement increases further. This behaviour is characteristic of clays with turbulent or transitional shear mode and plasticity index between 10 and 21 %, and silty clays with transitional to sliding shear mode and plasticity index between 26 and 36 %.

Type V involves an increase of the residual strength with increasing rates of displacement and is characteristic of clays with sliding shear mode, plasticity index between 36 and 51 %, and clay content above 48 %.

Tika argued that the increase in soil strength above its slow residual level with increasing rates of displacement, which was observed in behaviour Types II, III, IV, and V, may be attributed to viscous effects in the cases where soils with transitional and/or turbulent shear mode are involved, and to disorientation of particles in cases where soils with sliding or transitional shear mode are involved. Furthermore, she suggested that the substantial decreases in strength below its slow residual level, which were observed in behaviour Types II, and III, should be attributed either to increased porosity of the shear zone or decreased viscosity and increased ability to flow. Ring shear tests against specially equipped glass interfaces indicated that pore pressure and temperature increases during fast shearing are insignificant. Finally, Tika added that once the soil strength drops significantly below its residual level it becomes insensitive to any further changes of the rate of displacement.

Lemos (1991) presented further arguments on the behaviour of soils on fast shearing. He defined the effect of fast shearing as *neutral*, *positive* or *negative* depending on whether the soil strength remains equal to, increases above, or decreases below its slow residual level, respectively. He suggested that soils with clay fraction below 5 % exhibit neutral rate effects, soils with clay fraction above 50 % exhibit positive rate effects and soils with clay fraction between 5 and 40 % will exhibit negative rate effects. However, he presented examples of soils which occasionally deviate from these general rules. Lemos, suggested that the immediate and maximum increase in strength with increasing rate of shearing should be attributed to viscous effects and that the threshold

strength increases with increasing rate of shearing. Furthermore, he postulated that the positive rate effect is caused by disturbance of the shear zone, is more profound in clayey soils which contain massive particles, and reaches its maximum at rates of displacement around 400 mm/min. Even though some of the evidence which were presented may suggest the development of excess pore pressures upon fast shearing [Figs. 2.4.6 and 2.4.7], Lemos concluded that the negative rate effect should be attributed to increased porosity and water content of the soil matrix within the shear zone. The presence of water in the bath of the ring shear apparatus appeared to change the behaviour of a sample from positive to negative [Fig. 2.4.8], whereas, the temperature rise which was measured during shearing at 400 mm/min was considered insignificant to effect the measured strength.

Yagi *et al* (1991) presented ring shear tests in which the pore pressures generated during shearing were measured [Fig. 2.4.9]. The results suggest that while excess pore pressures are insignificant when the rate of displacement is below 0.35 mm/min and they do not affect the estimated soil strength, the situation reverses once the rate of displacement increases further. It appears that the excess pore pressure increases to a maximum value within the first 8 cm of shearing, decreases almost by 80 % after another 8 cm, and remains at that level until it decreases further to 10 % of its maximum value after the sample has been sheared for 40 cm.

Sassa *et al* (1991) and Sassa (1992) presented results from ring shear tests conducted in a *fast-loading, high-speed* apparatus which enabled the continuous monitoring of pore pressures generated in the sample during loading and shearing. In order to simulate earthquake conditions, the testing procedure which was adopted involved cyclic loading and shearing. The measured pore pressures were considered insignificant, and it was argued that they would have no effect on the behaviour of the sample. However, it was suggested that cyclic loading in both the normal and shear directions may have induced opposing dilating and contracting mechanisms which prevented the generation of excess pore pressures. In the case were the total normal stress decreased and shear velocity increased, the pore pressure appeared to increase considerably. Finally, it was observed that the measured sample strength increased above its slow residual value upon dynamic loading.

Steenfelt (1992) presented results from several field sliding, and large shear box tests on a remoulded clay till at various rates of displacement. He suggested that these results [Fig. 2.4.10] indicate that around 120 mm/hr of displacement rate there is a transition from *elastic* to *elastoplastic* behaviour. He argued that the measured soil strength increases with decreasing rate of displacement below 120 mm/hr while it increases with increasing rate of displacement above 120 mm/hr. Furthermore, Steenfelt presented comparative results between load and displacement controlled tests. He concluded that, even though the load controlled tests resulted in lower estimates of strength, both test types indicate similar relationship between soil strength and rate of displacement [Fig. 2.4.11].

2.5 Theories on the Loss of Soil Strength Upon Fast Loading

The apparent loss of soil strength associated with a number of catastrophic landslides has been the focus of a several investigations.

Kerr and Drew (1968) studied landslides which had occurred during the 1964 Alaska earthquake and involved quick-clay. They speculated that the observed loss of soil strength must be attributed to a *thixotropic breakdown* of the clay structure to *flowing fluid*.

Hutchinson and Bhandari (1971), and Hutchinson (1986), upon studying the mechanisms of mudflows and flow slides, postulated that the apparent decrease in frictional resistance, below its residual level, associated with such events is brought about by undrained loading which gives rise to excess pore pressures along the slip surfaces.

Erismann (1979) indicated that large landslides are associated with "high concentrations of energy" and increased temperatures along the sliding surface which result in *self-lubrication*, through fusion or rock dissociation, and consequently loss of strength.

Voight and Faust (1982) suggested that the rapid loss of rock strength during sliding may be associated, under specific circumstances, with frictional heat generated within the shear zone of a landslide. They argued that this increase in temperature may result in increased pore pressures, vaporisation, or even rock melting. However,

vaporisation is considered rare because is associated with thick slides and soft slip zones, whereas, rock melting is possible only under temperatures in excess of 1000°C.

Sassa (1984) presented a mechanism for landslides and debris flows. He suggested that the decrease is soil strength is a result of an increase of the specific gravity of the pore fluid. He postulated that rapid loading causes an increase in the percentage of fine soil particles which are floating in pore water, an increase in the specific gravity of pore fluid, and, consequently, a decrease in the effective stresses and the soil strength.

Sassa (1992) presented further experimental evidence which indicate that the apparent loss of soil strength during rapid loading may be attributed to increased pore pressures.

2.6 Summary

Previous research works relevant to this investigation have been reviewed in this chapter. Literature on the subject of residual soil strength, which forms the basis of comparison for this work has been presented followed by literature highlighting the advantages of the ring shear apparatus, in its various forms, to establish large unidirectional displacements. Subsequently, literature associated with the rate effects on soil strength has been presented. It is suggested by previous findings that the behaviour of soils and the variations in residual strength during fast shearing depends on the mode of their interparticle movements during slow shearing, their plasticity index, and their clay content. The role of increased pore pressures and temperatures on residual strength has not been properly evaluated mainly because of limitations imposed by the existing laboratory equipment. Finally, some of the theories which have been advanced to explain the loss of soil strength during fast shearing were presented.

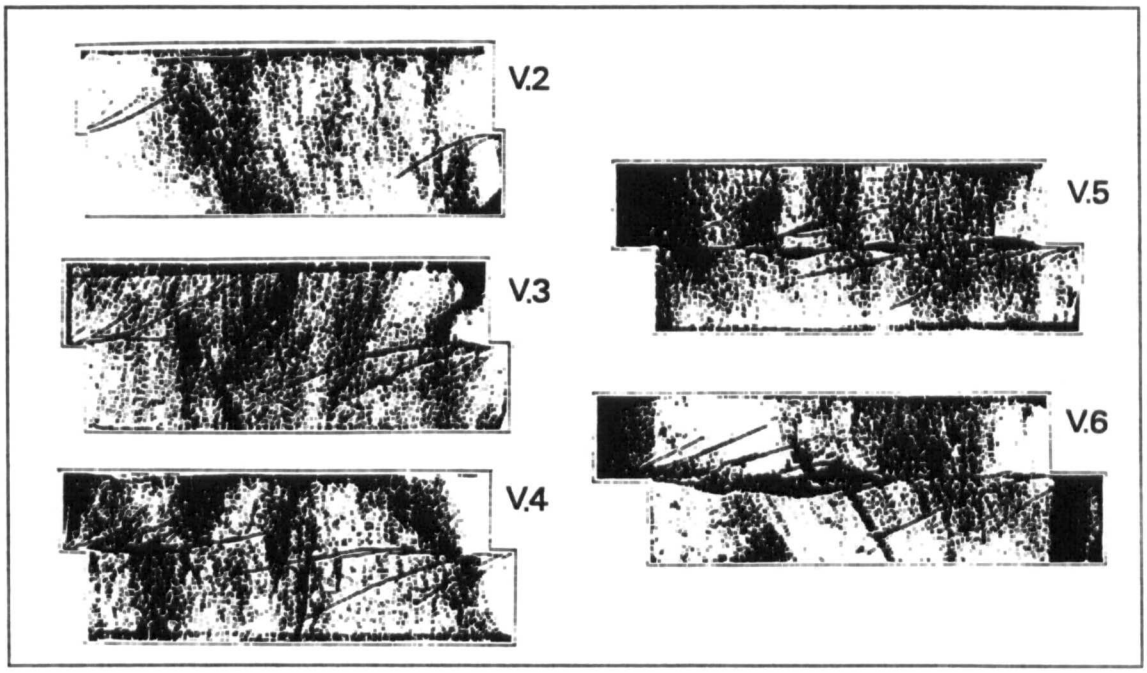


Figure 2.2.1 Sequence of shear induced fabrics (Morgenstern & Tchalenko, 1967)

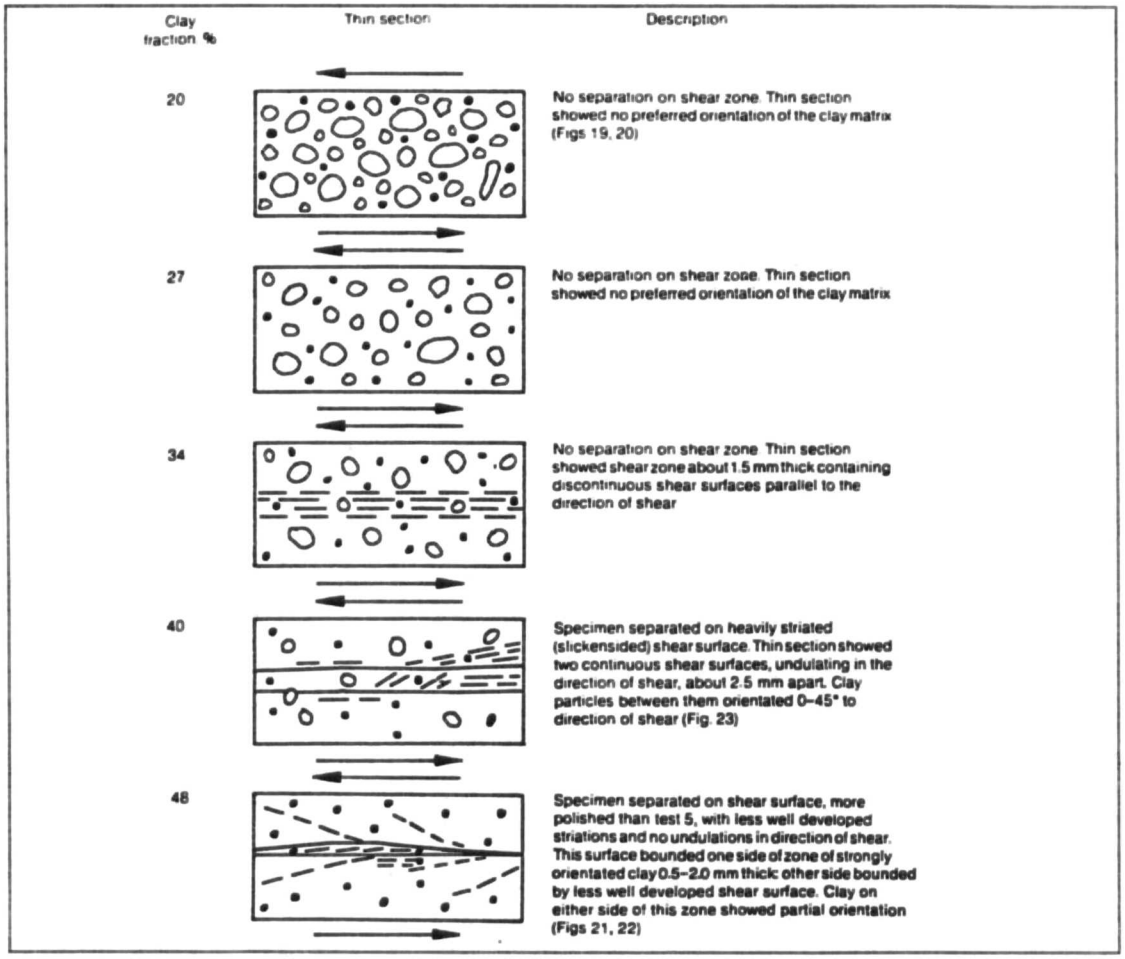


Figure 2.2.2 Summary of post-failure structures (Lupini et al, 1981)

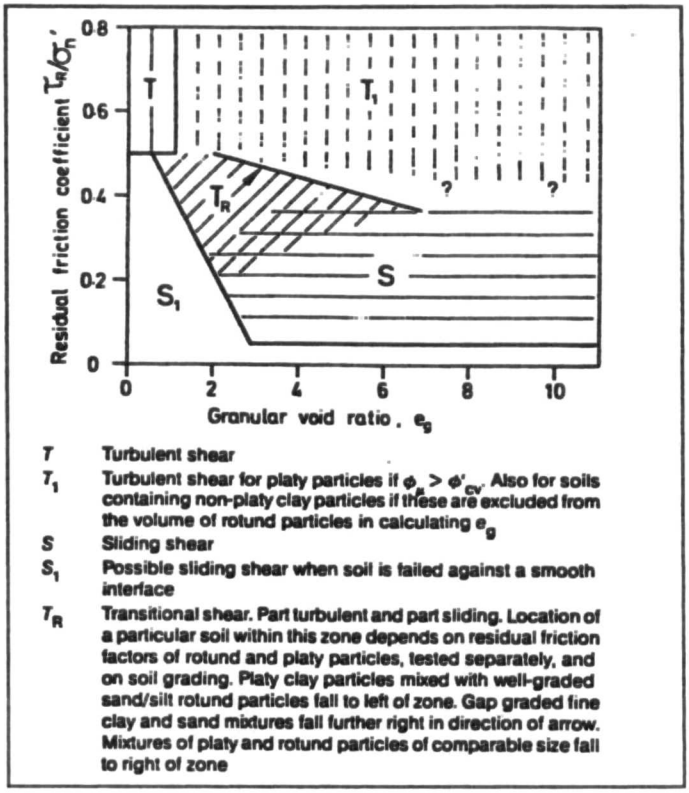


Figure 2.2.3 Residual shear mechanisms as a function of particle packing (Lupini et al, 1981)

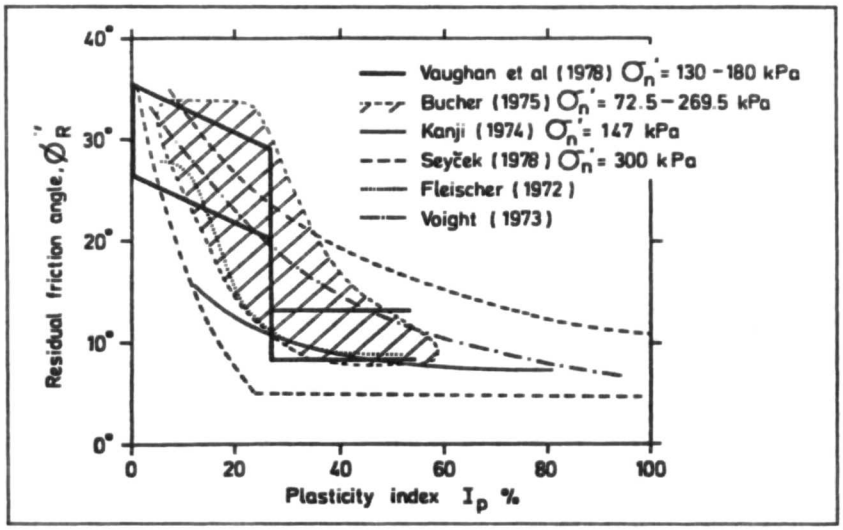


Figure 2.2.4 Correlations of residual strength with plasticity index (Lupini et al, 1981)

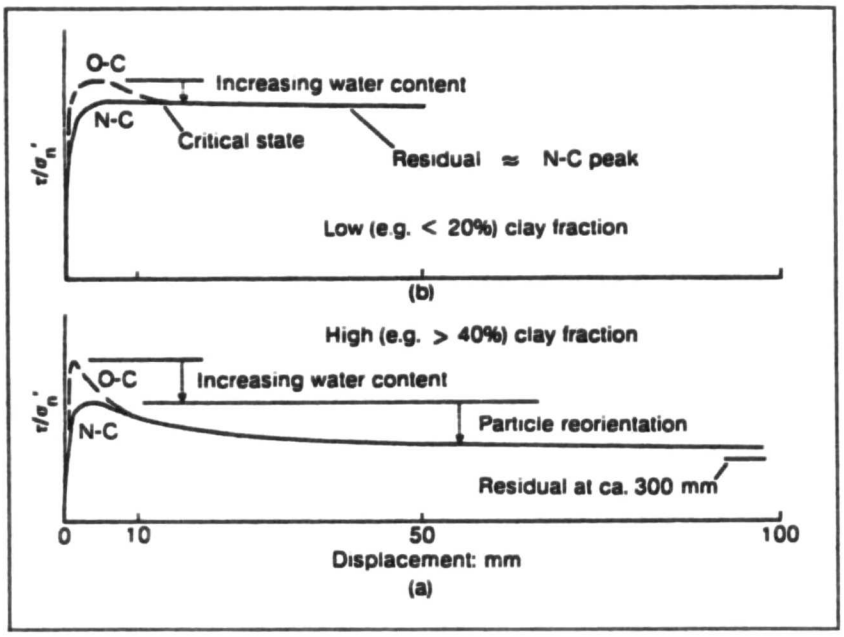


Figure 2.2.5 Stress-displacement curves at constant σ_n' (Skempton, 1985)

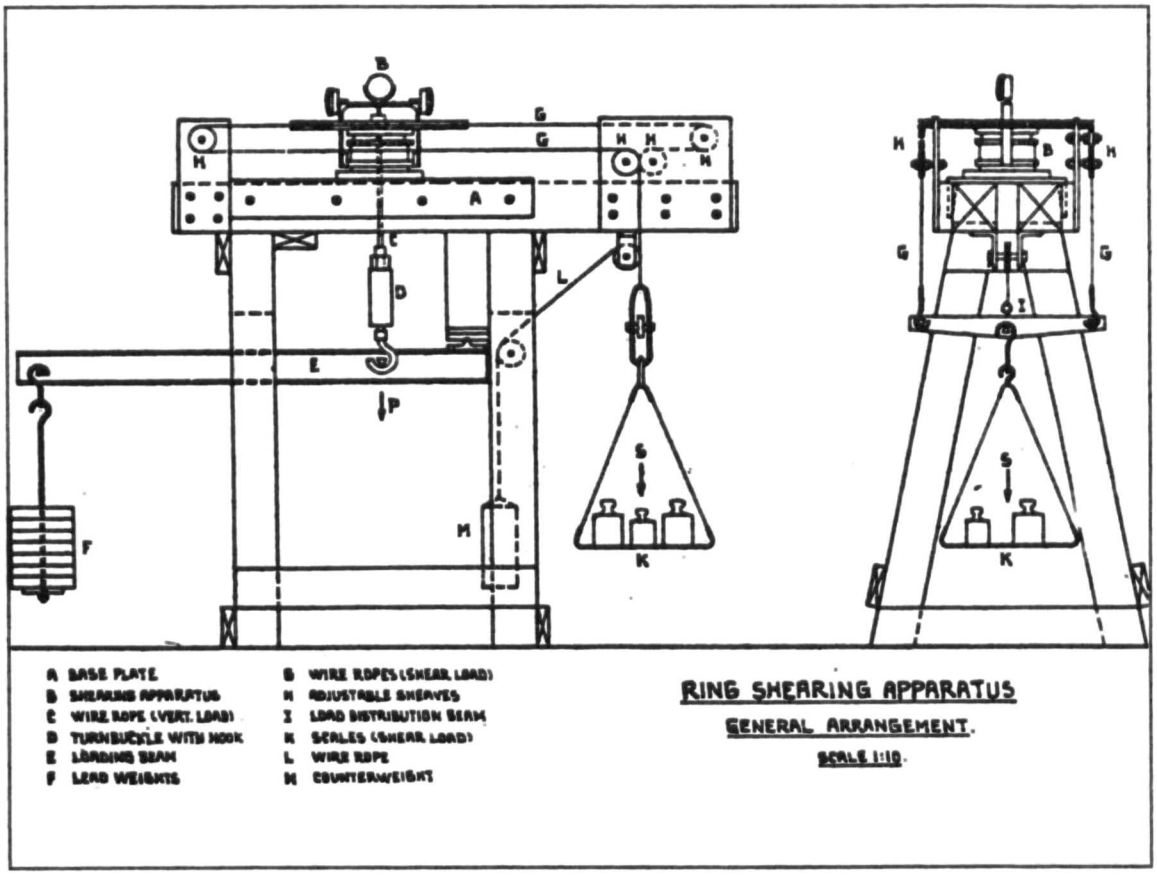


Figure 2.3.1 Hvorslev's ring shearing apparatus (Hvorslev, 1936)

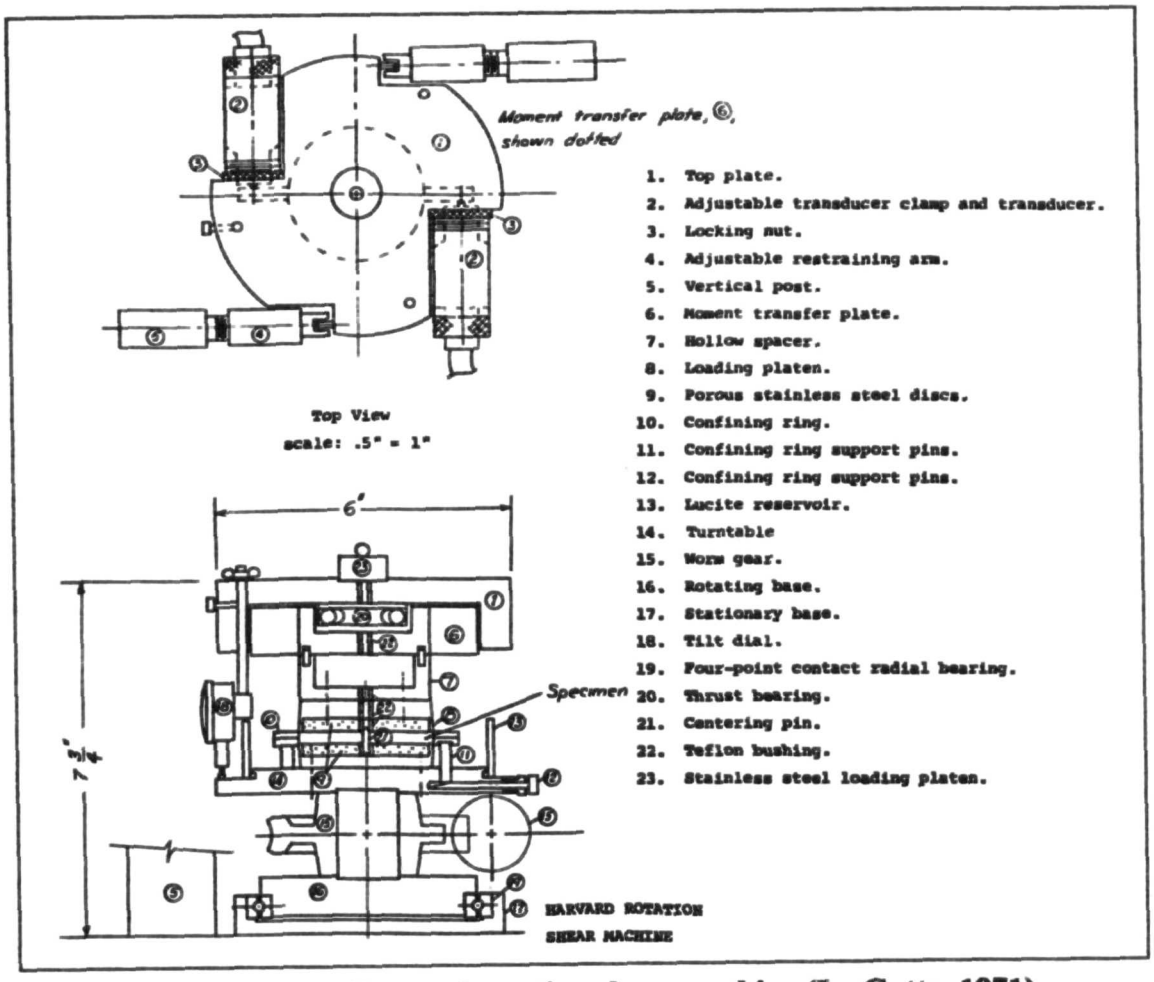


Figure 2.3.2 Harvard rotation shear machine (La Gatta, 1971)

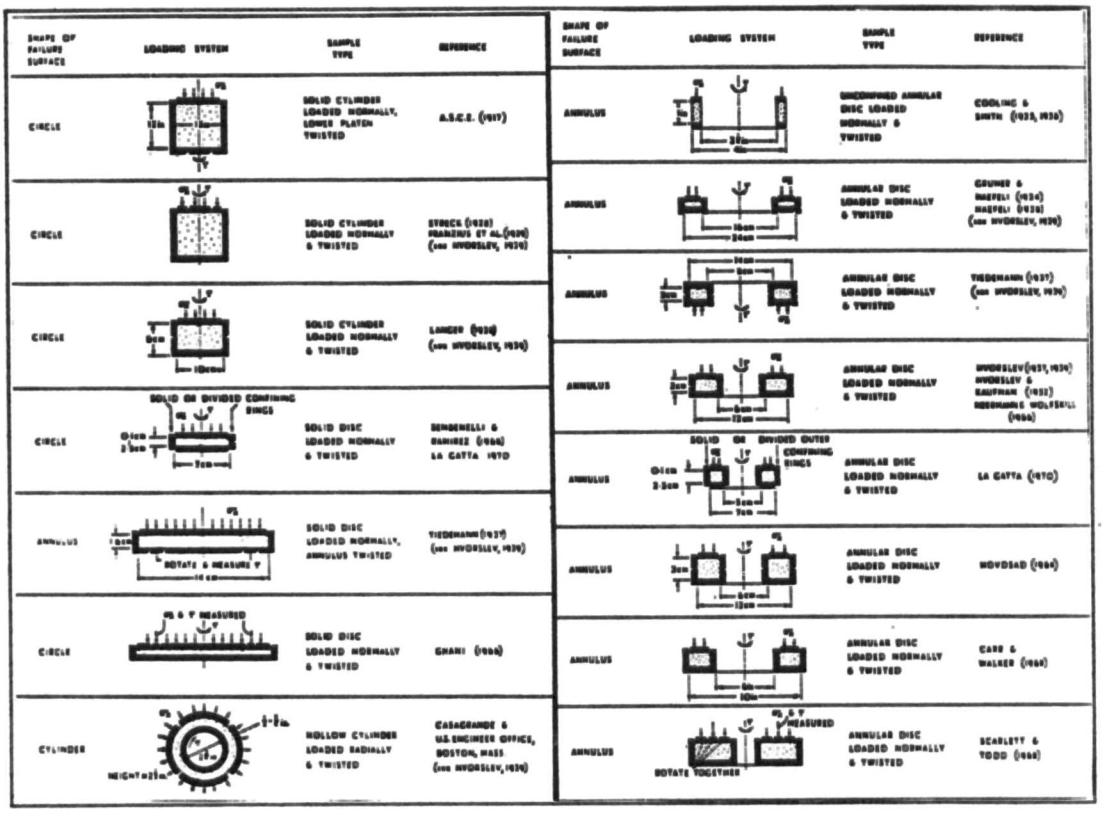


Figure 2.3.3 Principal features of various forms of torsion and ring shear apparatuses (Bishop et al, 1971)

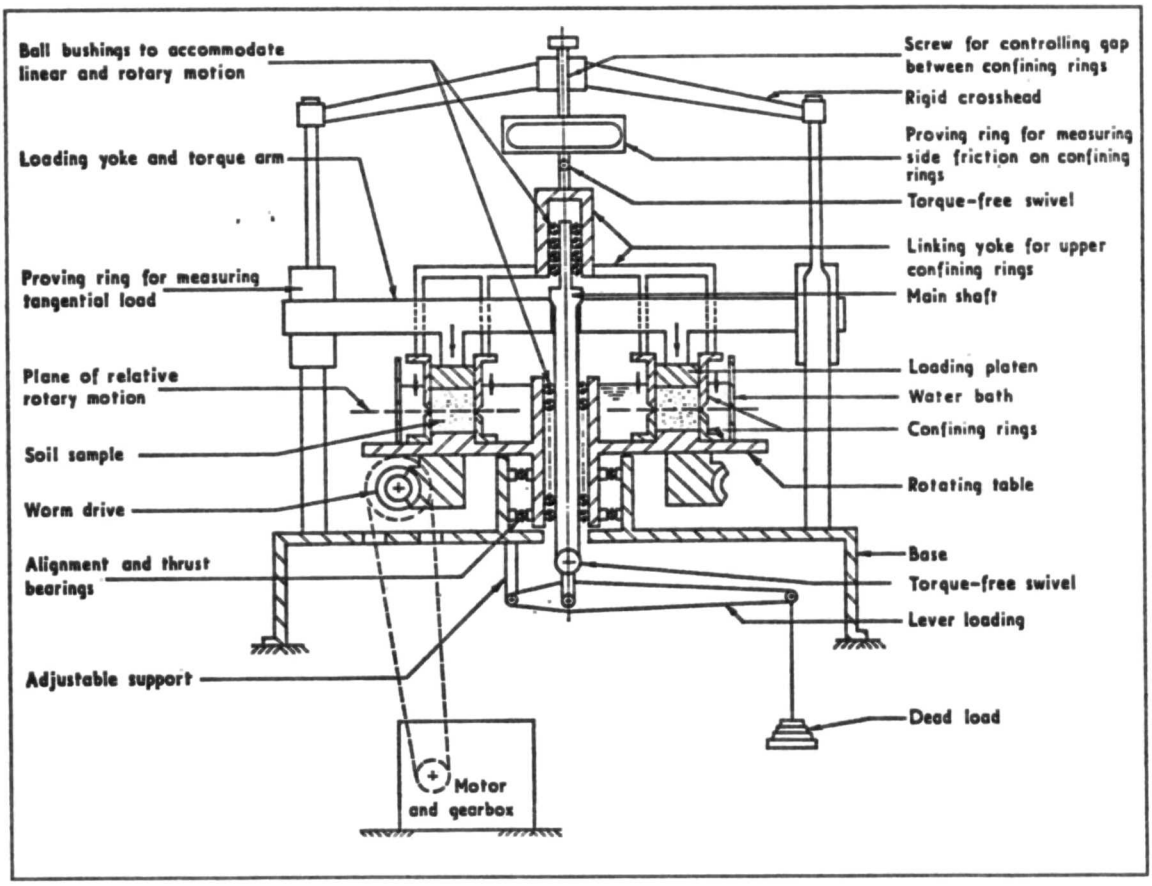


Figure 2.3.4 The IC/NGI ring shear apparatus (Bishop et al, 1971)

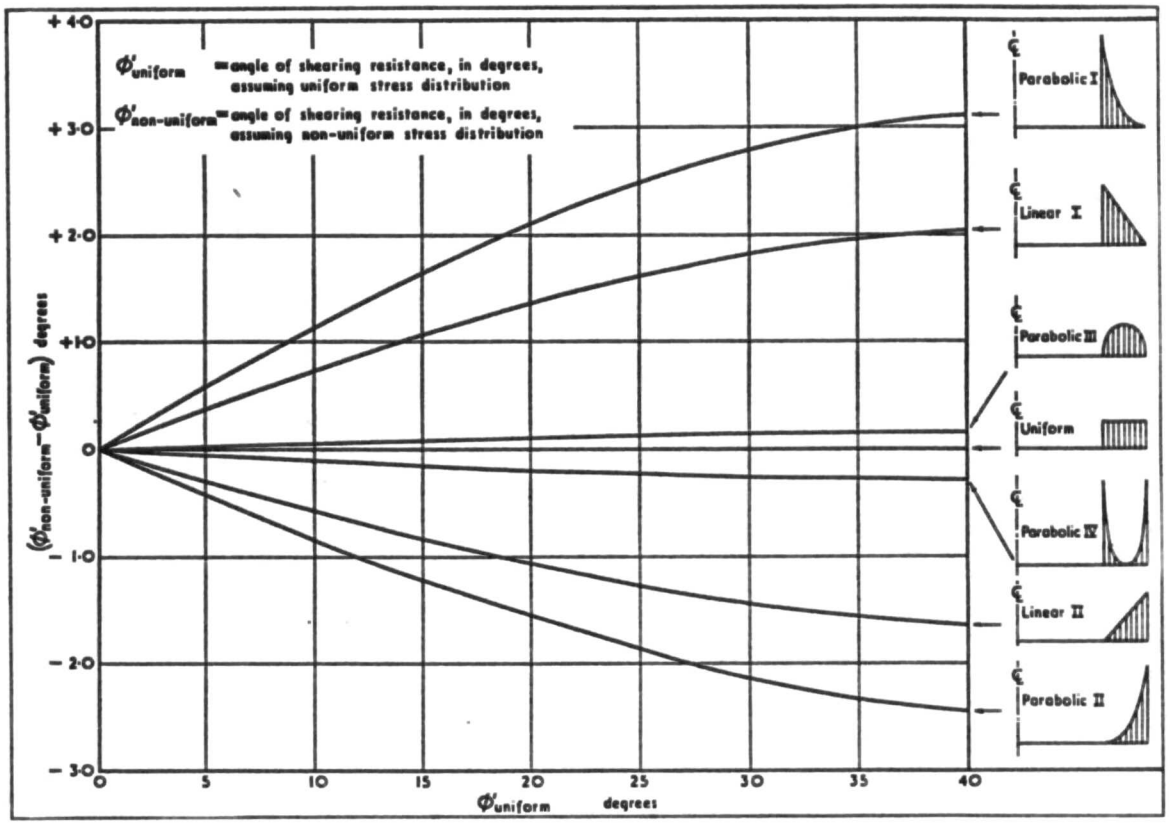


Figure 2.3.5 Influence of assumed stress distribution on value of angle of shearing resistance (Bishop et al, 1971)

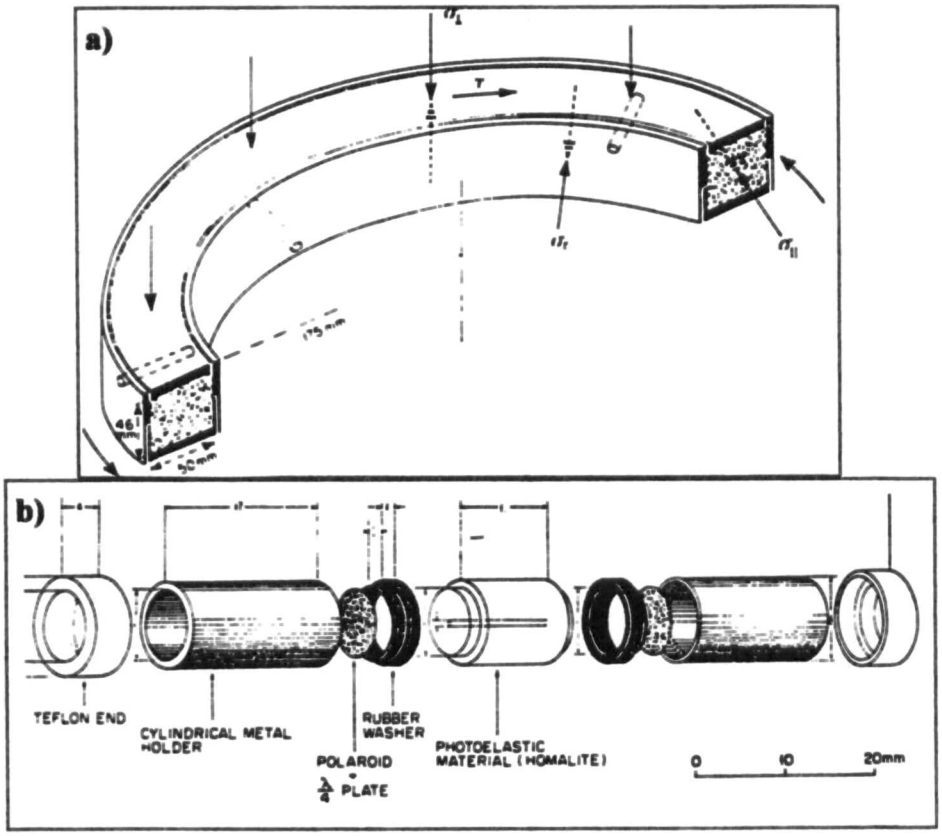


Figure 2.3.6 Cambridge load cells in ring shear apparatus (Mandl et al, 1977)

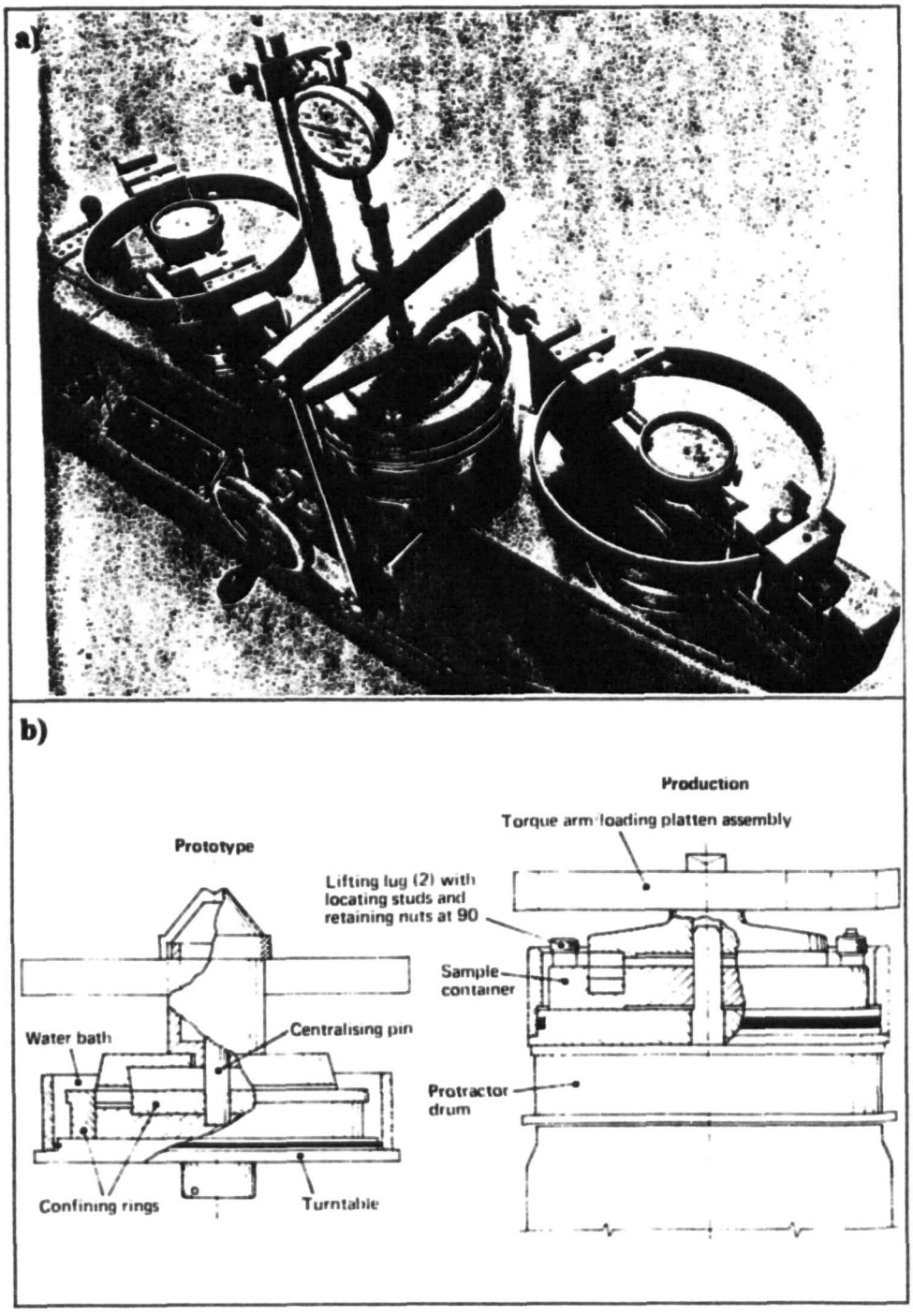


Figure 2.3.7 Bromhead-type ring shear apparatus (Bromhead, 1979)

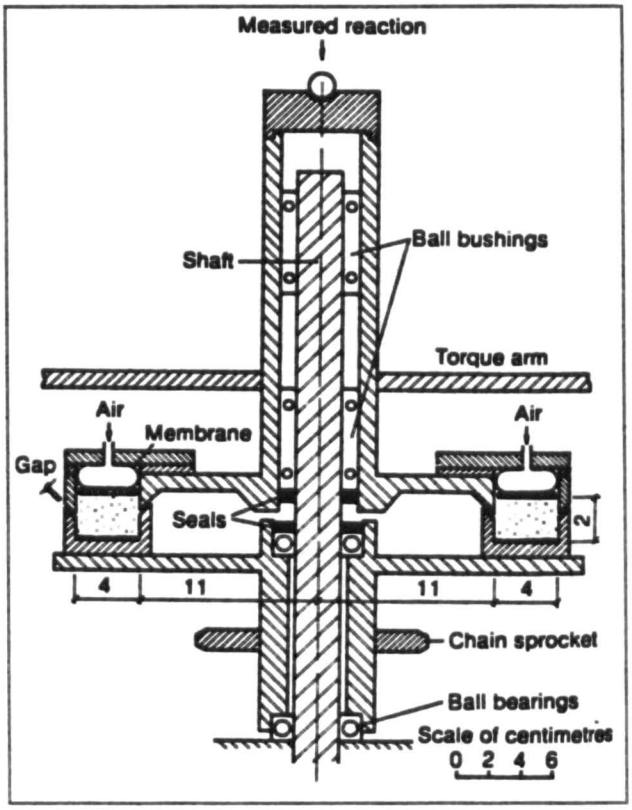


Figure 2.3.8 Cross section of ring shear apparatus used by Hungr & Morgenstern (1984)

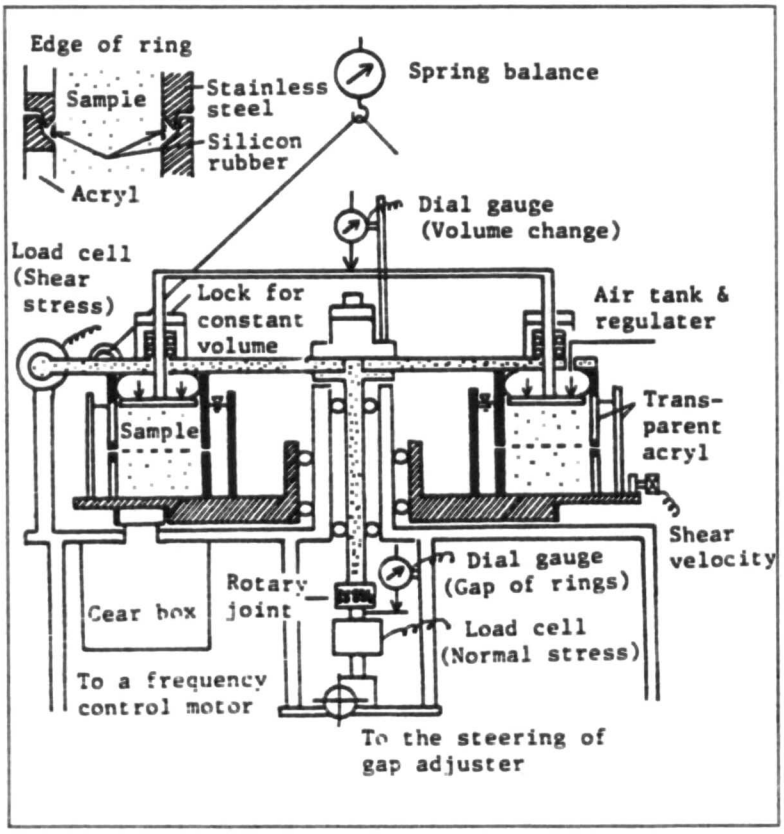


Figure 2.3.9 Ring shear apparatus presented by Sassa (1985)

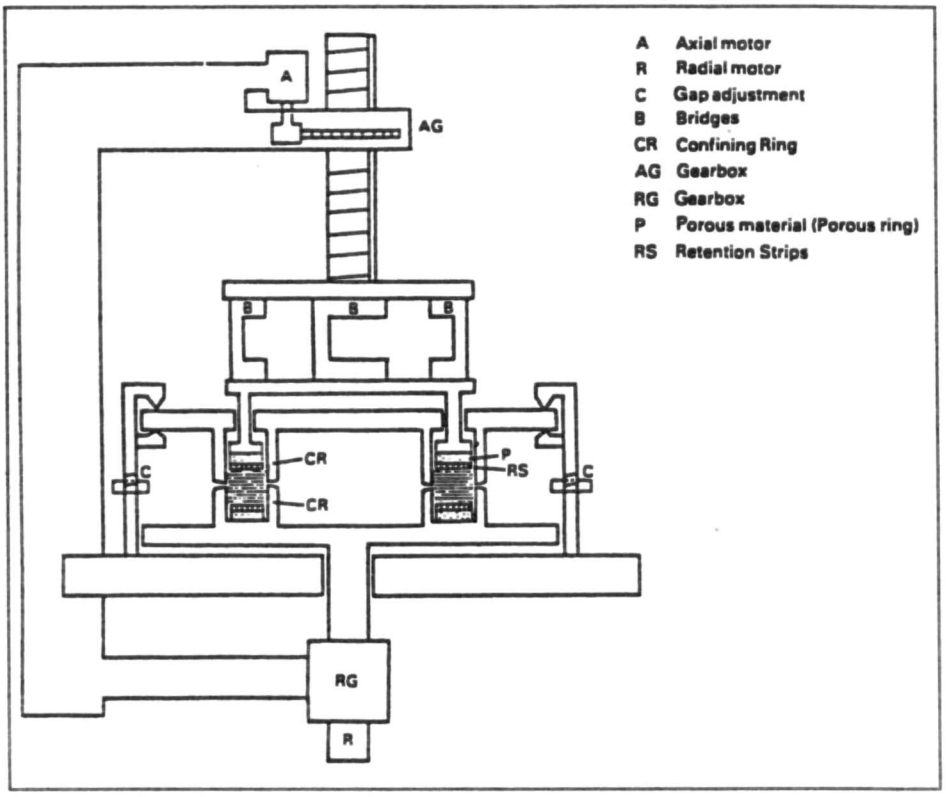


Figure 2.3.10 Utrecht University ring shear apparatus (Nieuwenhuis, 1991)

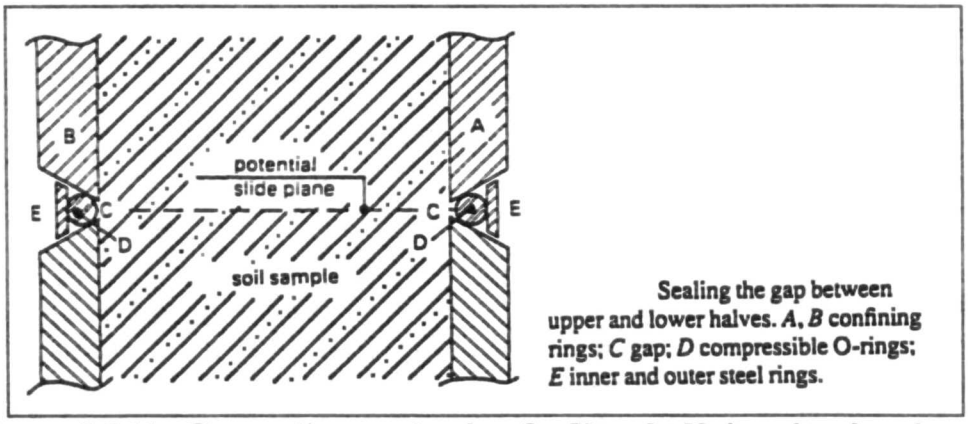


Figure 2.3.11 Gap sealing mechanism for Utrecht University ring shear apparatus (Nieuwenhuis, 1991)

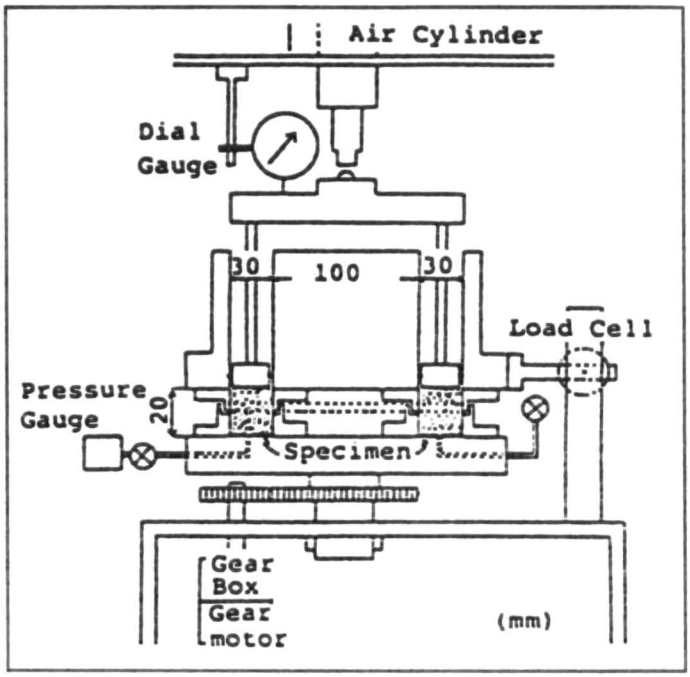


Figure 2.3.12 B-type ring shear apparatus (Yagi et al, 1991)

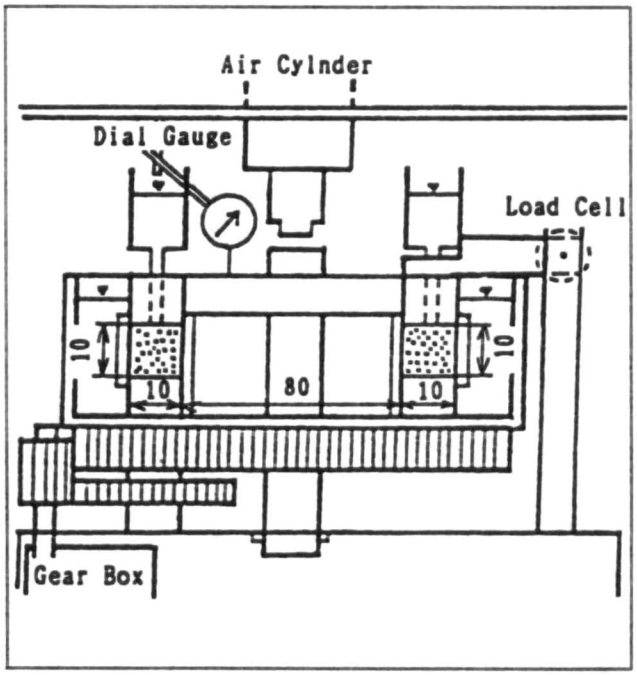


Figure 2.3.13 S-type ring shear apparatus (Yagi et al, 1991)

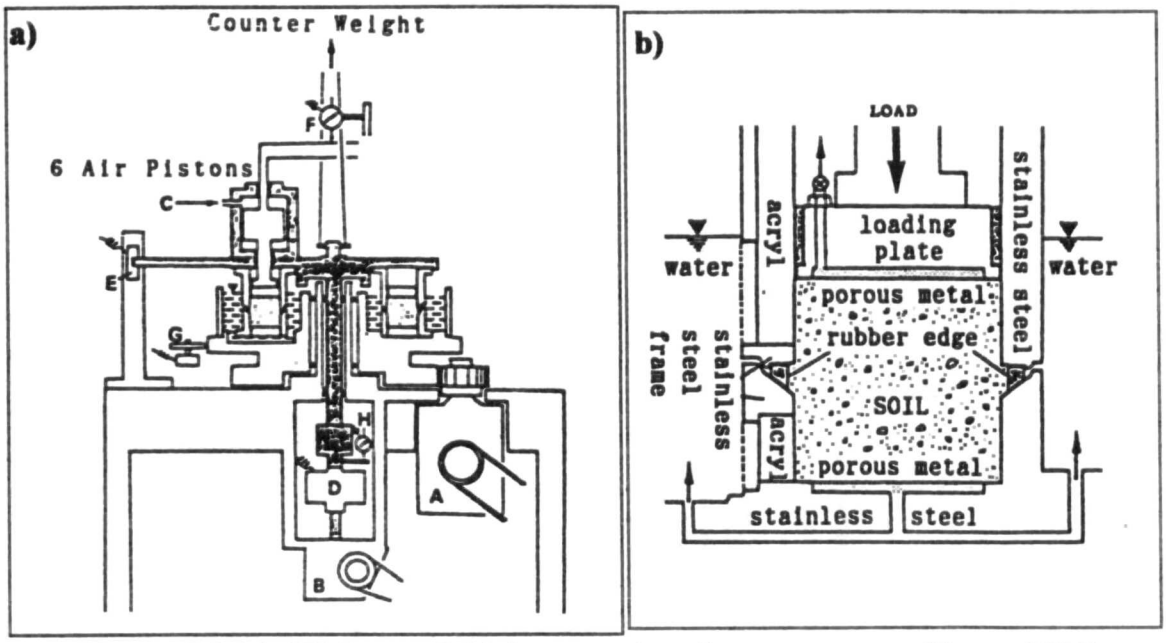


Figure 2.3.14 High-speed, high-stress ring shear apparatus (Sassa, 1991)
a) General diagram of the apparatus b) Cross section of the shear box

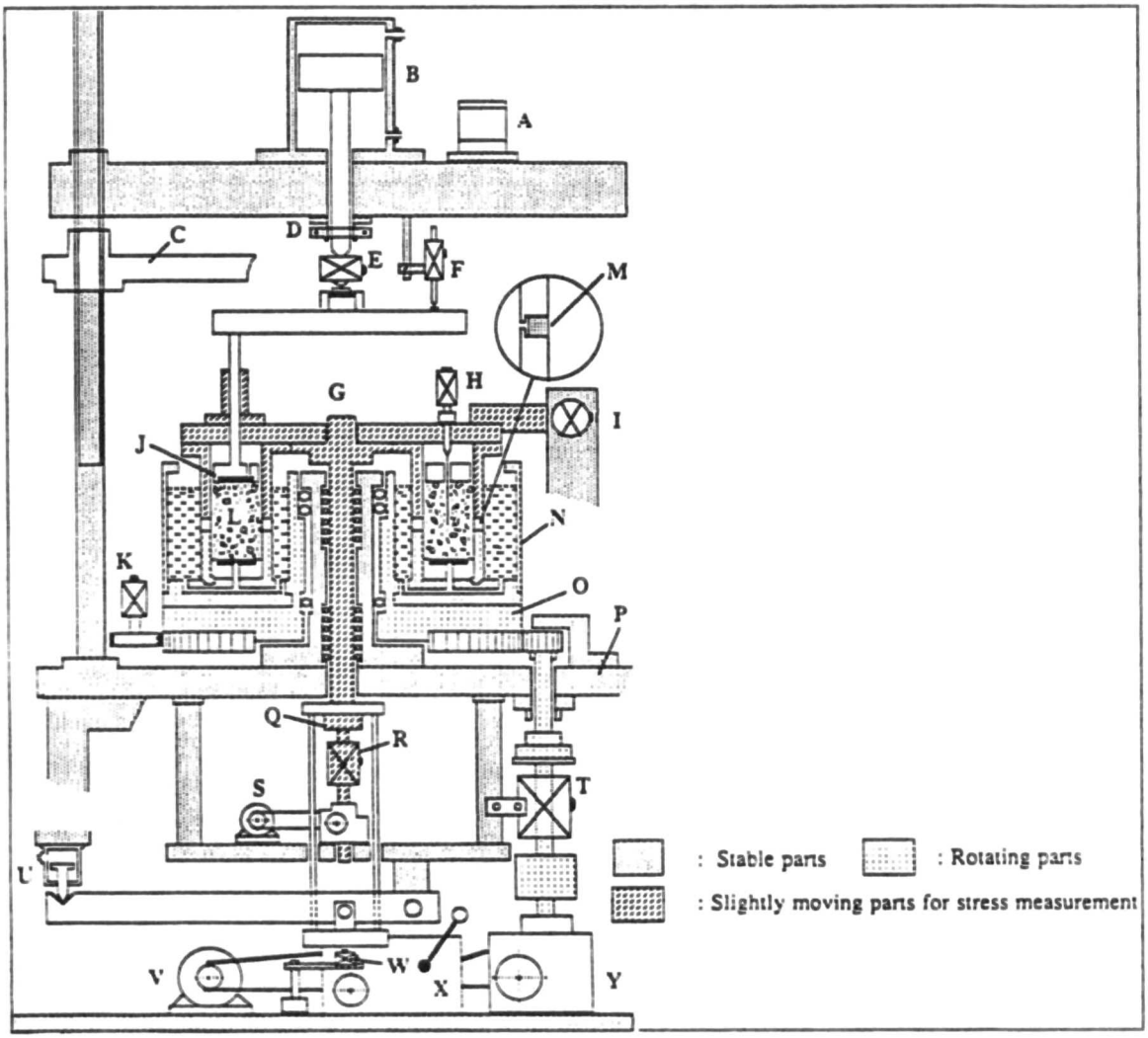


Figure 2.3.15 Cyclic loading, high-speed ring shear apparatus (Sassa, 1992)

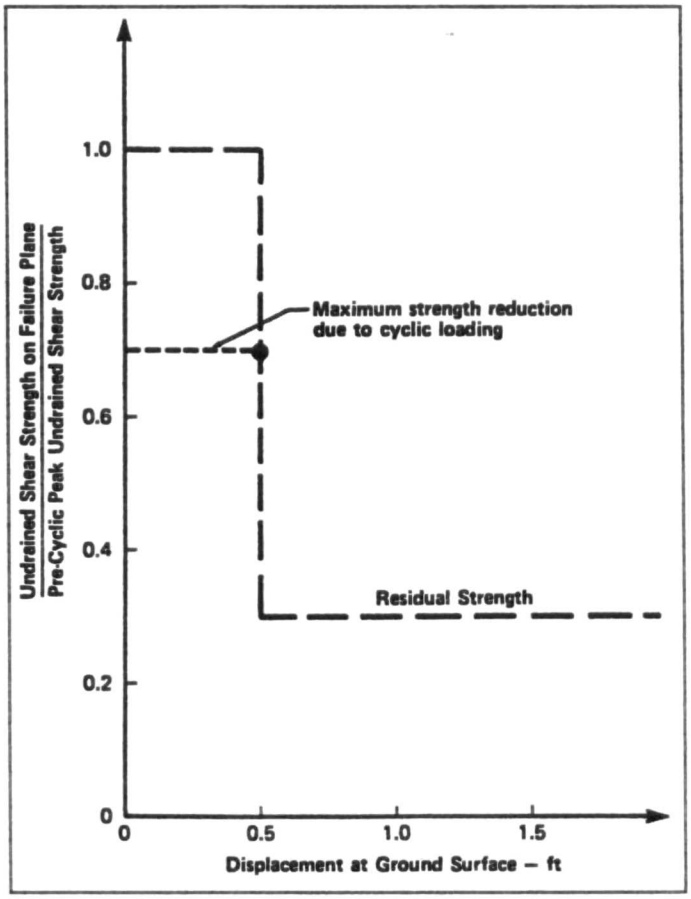


Figure 2.4.1 Variation of undrained shear strength on potential failure surface (Idriss, 1981)

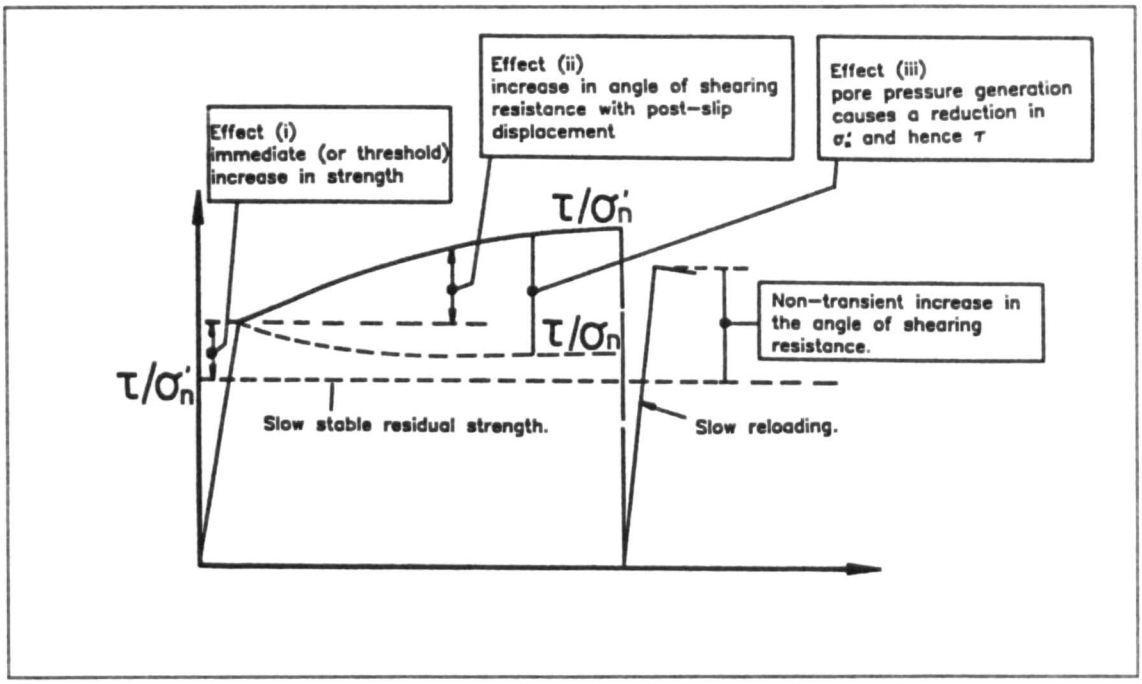


Figure 2.4.2 Summary of rate-dependent phenomena (Martins, 1983)

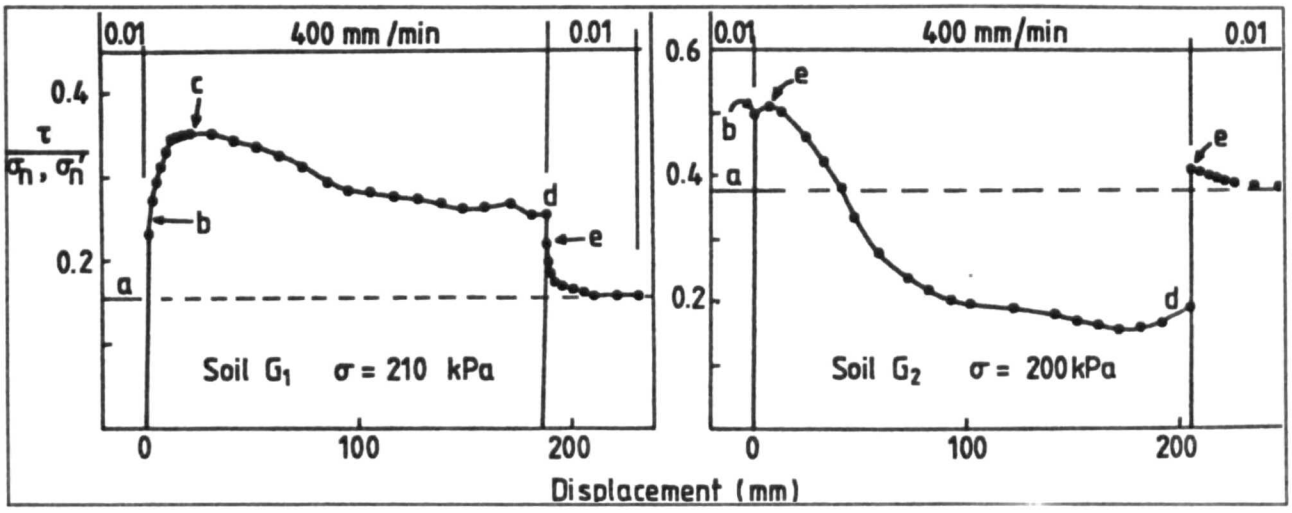


Figure 2.4.3 Typical behaviour during fast shear tests (Lemos et al, 1985)

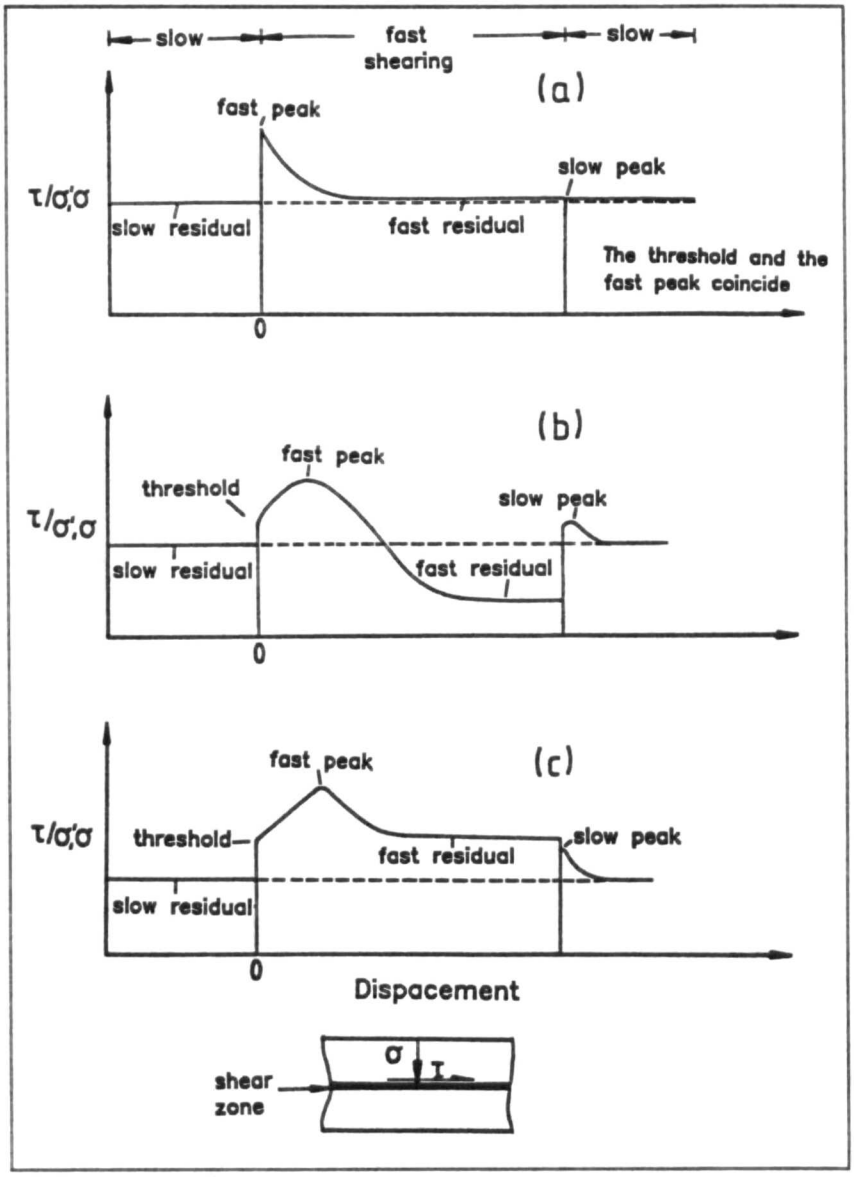


Figure 2.4.5 Typical behaviours of pre-sheared samples during fast shearing (Lemos, 1986)

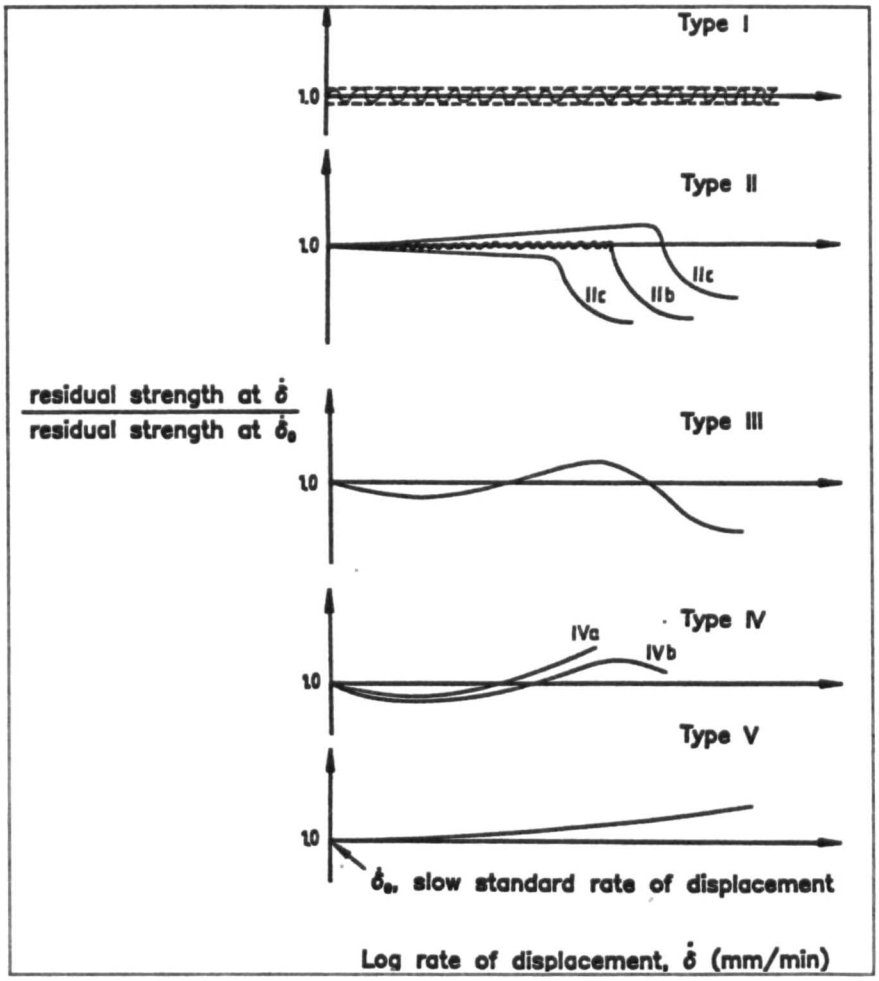


Figure 2.4.5 Variation of residual strength during fast shearing (Tika, 1989)

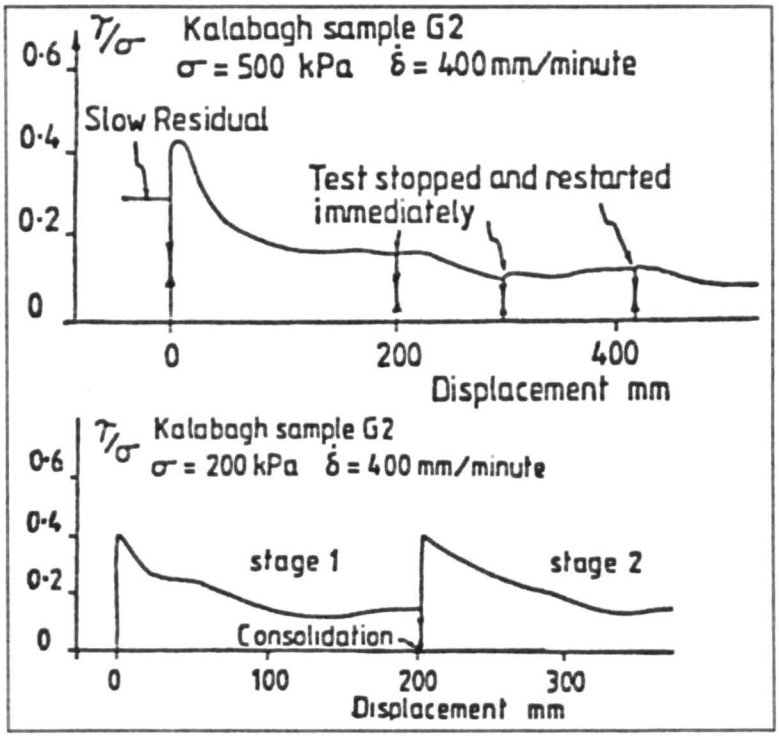


Figure 2.3.6 Strength recovery after consolidation (Lemos, 1991)

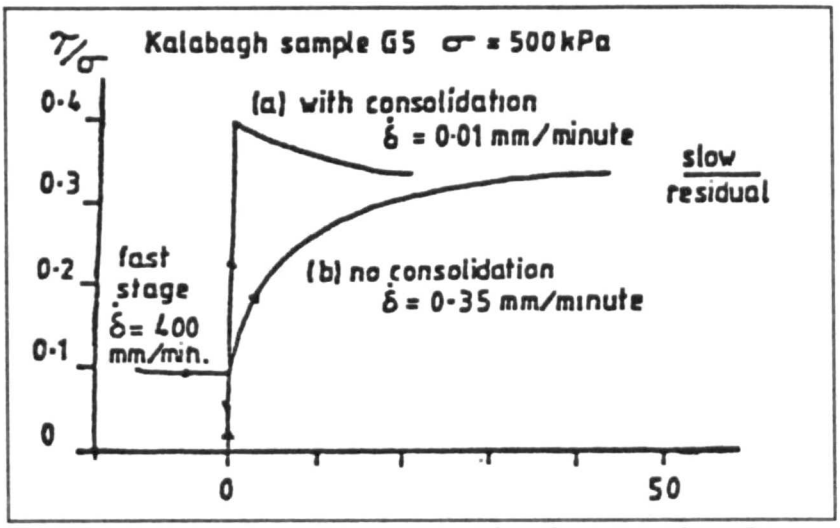


Figure 2.4.7 Pore pressure dissipation after fast shearing (Lemos, 1991)

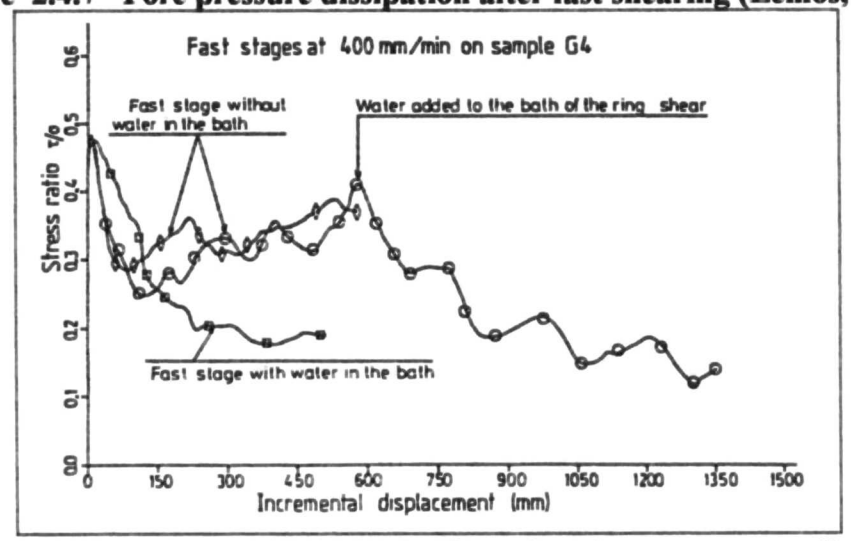


Figure 2.4.8 Effect of water in the bath of the ring shear apparatus (Lemos, 1991)

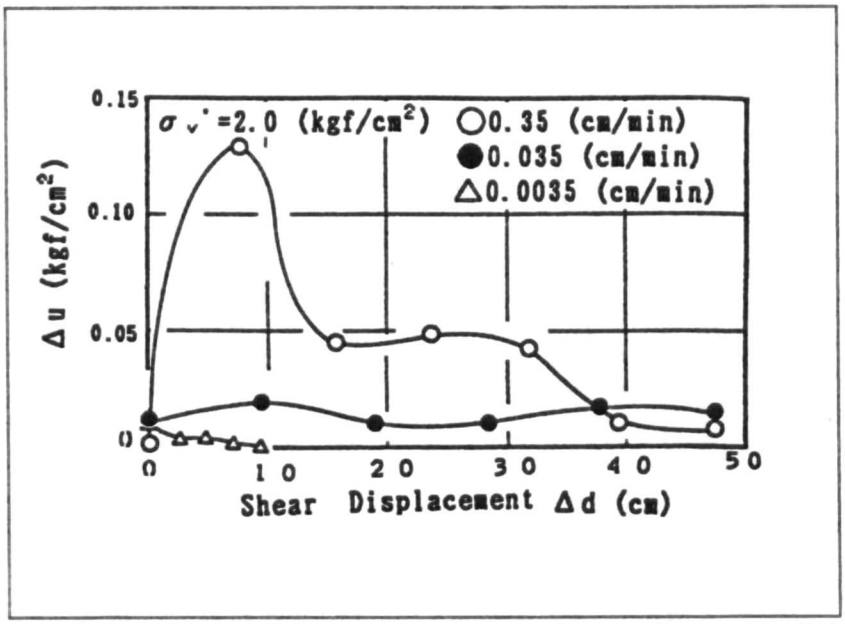


Figure 2.4.9 Changes in pore pressure at increasing rates of shear (Yagi et al, 1991)

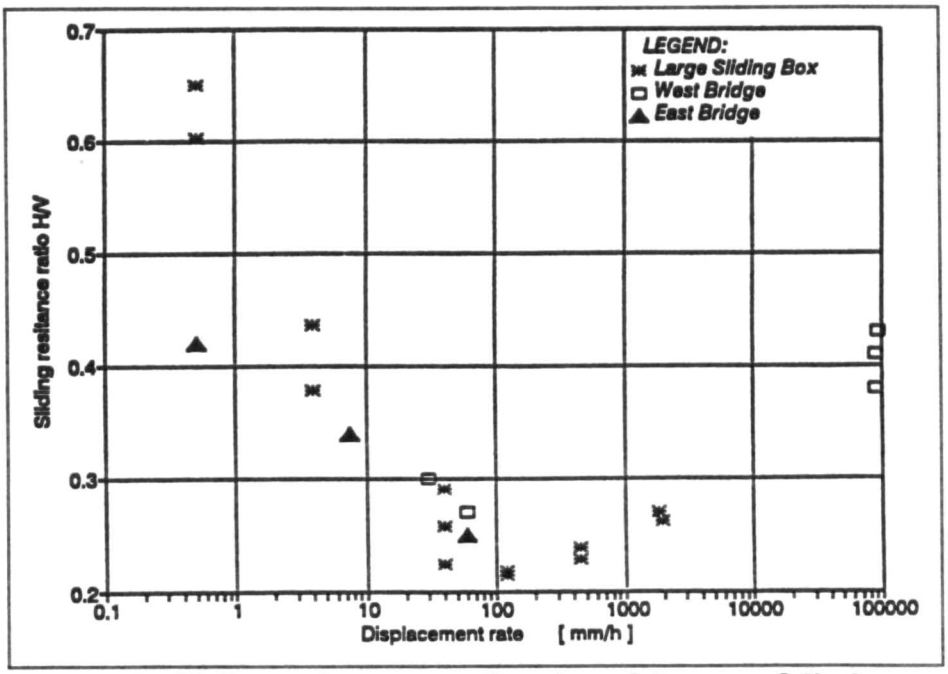


Figure 2.4.10 Sliding resistance as a function of the rate of displacement (Steenfelt, 1992)

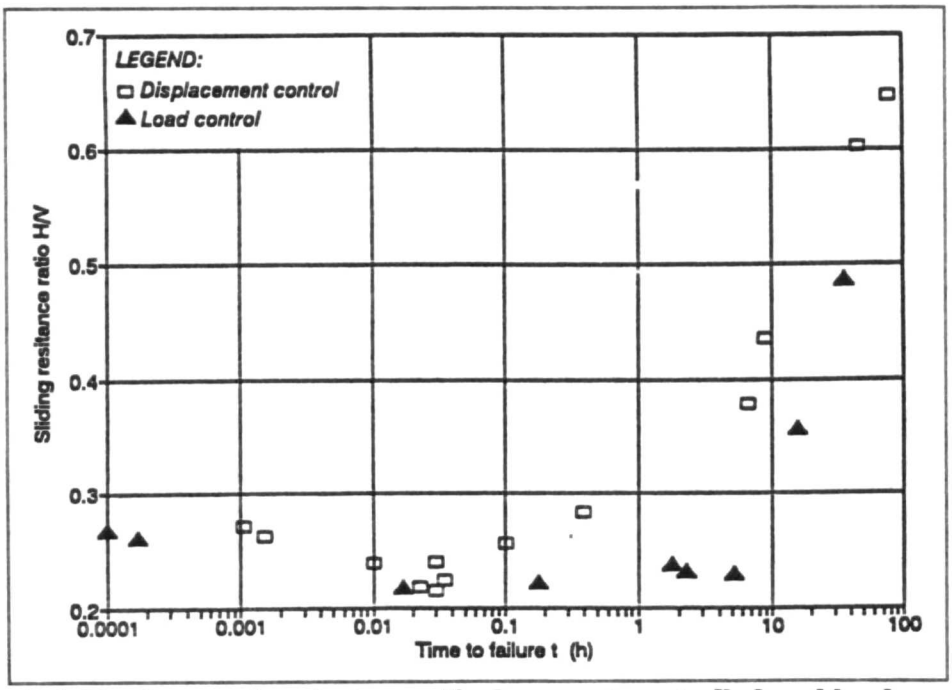


Figure 2.4.11 Comparison between displacement controlled and load controlled tests (Steenfelt, 1992)

CHAPTER THREE

DESCRIPTION OF APPARATUS, TESTING METHODS AND PROCEDURES

3.1 Introduction

This investigation on the effects of the rate of displacement on the residual strength of soils has been conducted entirely by means of the IC/NGI ring shear apparatus. The soil types which have been tested are of a wide range of plasticity and include four fine-grained soils, of which two are natural and one is an artificial mixture, and a coarse-grained sand. In this chapter, a brief description of the apparatus and its peripheral system is presented. The preparation techniques and the testing procedures are also described, along with the procedures used for the classification tests.

3.2 The IC/NGI Ring Shear Apparatus

3.2.1 System Description

A detailed description of the design of the ring shear apparatus which has been used for the purpose of this investigation was presented by Garga (1970) and Bishop *et al.* (1971) [Figs 2.3.4, 3.2.1]. Two devices of the same general design [Fig 3.2.2] were used, and they will be referred to as *Ring 1* and *Ring 2* according to the terminology which has been used by previous researchers who employed the same equipment (Lupini 1981, Lemos, 1986, and Tika, 1989). The principal difference between the two devices in their present configuration is the motor-drive units which are of different specifications.

Briefly, the IC/NGI ring shear apparatus, which is a *controlled rate of displacement* device, consists of a rotating base and a stationary frame, and houses an annular sample which is confined between a set of rings on the sides and two porous

stones, one at the top and one at the base [Fig 3.2.3]. The annular sample is 152.4 mm and 101.6 mm in outer and inner diameters respectively, and 19 mm in initial thickness. The confining rings consist of an upper part which is connected to the stationary frame, and a lower part which is connected to the rotating base. Rotation is provided by a *semi-automatic control* motor and a set of gears [Figs 3.2.4, 3.2.5]. The normal load is applied by a *dead load lever system* and is transmitted to the top porous stone, and the sample, through a *loading yoke* which also transmits the shearing resistance from the shear surface to the load cells. The loading yoke is locked into position with the upper porous stone by a set of safety pins. The upper and lower parts of the confining rings may be separated, and their vertical position is controlled by means of a differential screw and a proving ring which are located at the top of the stationary frame. The proving ring enables the measurement of the side friction, which develops between the sample and the inner walls of the confining rings when the sample dilates or contracts during shearing. When the base begins to rotate, a shear force is imposed on the sample, at or around its mid-height, and the shearing resistance which develops between the upper/stationary and the lower/rotating parts of the sample is transmitted through the loading yoke to the load cells and measured.

3.2.2 Instrumentation and Measuring Devices

The various quantities which are required for the calculation and presentation of the test results are measured by a set of electronic devices, transducers, a pair of dial gauges, and a angular vernier scale.

The shearing resistance is transmitted from the shear zone through the safety pins and the loading yoke, and is measured by a pair of *Imperial College load cells* which are mounted on the vertical columns of the stationary frame. The design principles of the Imperial College load cells have been presented by El-Ruwayih (1975). With the initiation of the shearing procedure, the loading yoke becomes tangential to the load cells, compresses (or extends) them, and causes their horizontal deflection. The latter is measured by means of *inductive displacement transducers*, with a maximum travel of 2 mm, which are equipped with a pair of *linear variable differential transformers* (L.V.D.T) connected to modulator/demodulator cards with a DC output. The load cells

are calibrated and their deflection is associated with the load which results in it. Ideally, during shearing, both load cells should measure the same or similar quantities of shearing resistance. However, it has been observed occasionally that, due to non-uniformities of the shear zone and loss of balance of the loading yoke, the load which is being measured deviates substantially between the two cells. This non-uniform stress distribution is an inherent problem of this apparatus which cannot easily be corrected, but which is not considered to affect the test results significantly.

The side friction is measured by a *strain-gauged* proving ring which is mounted at the top of the stationery frame. The dilation, or contraction, of the sample during shearing induces a deflection of the proving ring which is converted to load in a similar manner with the load cells which have been described previously. The problems which are associated with the flexibility of the proving ring will be discussed in Chapter 4.

The vertical displacement of the sample is measured by a pair of *linear strain conversion (LSC) displacement transducers* with a maximum travel of 25 mm, which produce a proportional voltage output in relation to their spindle displacement. The spindles of the two transducers contact the top porous stone at diametrically opposite points. These transducers provide a continuous monitoring of the changes of the sample height during consolidation and shearing. However, volume change calculations based on the sample height changes should not be considered quantitatively since they are affected by the loss of soil through the gap which separates the upper and lower confining rings.

The horizontal displacement of the sample is measured by means of an *angular vernier scale*, when the rate of displacement allows its visual monitoring, and by means of a *conductive plastics rotary potentiometer*, when the vernier cannot be monitored. This particular potentiometer houses a conductive plastic track which is a thin film with a multiple conduct wiper configuration, and allows high speeds to be maintained over 360° continuous rotation. The transducer has a virtually infinite resolution, a sine/cosine output and its rotating arm is attached to a brass disk, 2.5 cm in diameter, which is kept, by means of a spring, continuously tangential to the rotating base of the apparatus [Fig 3.2.6]. Due to the fact that continuous monitoring of the transducer could result in erroneous displacement measurements when the output passes from one quarter of the sinusoidal curve to another, it was decided that the monitoring of the transducer would involve counting the changes of the sign of the output. Considering that the sign of the

output changes every half rotation of the brass disk, the horizontal displacement was estimated by multiplying the number of output sign changes with the half of the perimeter of the disk. This would lead to an underestimation of the horizontal displacement at the beginning of the test, but the overall error was considered insignificant for the purposes of this investigation.

Finally, the vertical position of the upper confining rings and, therefore, the size of the gap, is monitored by a pair of dial gauges which are located diametrically opposite the front and back of the apparatus, and can be visually checked at any given time during the course of a test.

3.2.3 Data Recording System

Thus, each ring shear apparatus is equipped with six electronic measuring devices. The possibility of installing a pore pressure transducer and a thermocouple to the apparatus has also been considered. Previous results (Lemos, 1986, and Tika, 1989) suggest that the measured pore pressure and temperature increases are not significant enough to alter the test results. However, allowances were made for the system to accept one pore pressure transducer and one thermocouple for each ring shear apparatus, if this is later decided to be necessary. The signals from all the transducers of each ring shear are monitored by a Datascan 7220 data logging unit. The latter, is a 16-channel measurement processor with direct sensor connection for DC voltage, thermocouples, strain gauges, and 4-20 mA converters; it has a 16 bit resolution and a measurement speed of 40 readings per second. For the purpose of this experimental programme the data collection device enabled the simultaneous monitoring of the output of 8 transducers from each ring shear apparatus. In the present configuration only 6 transducers from each apparatus are connected to the data logging unit. The data logging unit also provides the interface between the transducers and a PC equipped with a 386 DX co-processor [Fig. 3.2.7], which collects the signals, converts them to measurements, and stores them according to commands given by a computer program which was written for the purpose of this research. The incompatibility between the speeds of the data logging unit and the PC has occasionally caused the system to collapse when monitoring was attempted continually. Subsequently, it was decided that a

maximum monitoring frequency of 12 readings per second would be employed, and this was considered to give results of satisfactory precision.

3.2.4 Computer Program to Record, Analyse and Present Data

The computer program utilised in this research enables the PC to distinguish the signals transmitted by the transducers of the two devices, converts these signals to measurements, calculates and plots on the screen the results, and stores them into the hard disk. The program comprises six subroutines and enables the operator to control the shearing, or consolidation, procedures in both devices simultaneously, calibrate all transducers, direct the monitoring of the data, retrieve and plot past test results while another test is in progress, and interrupt each test at any given time. Due to the limitations of the existing infrastructure, the automatic control by the PC of the rate of displacement or the normal load which is imposed on the sample during testing could not be established. Further information regarding this computer program is presented in Appendix II.

3.2.5 Gear Boxes and Motor Drive Units

A complete description of the gear boxes and motor drives of the IC/NGI ring shear devices has been given by Lupini (1981) and Tika (1989). Apart from the constant speed AC motor which drives Ring 1, which had to be replaced by a similar one, the same driving system, as Lupini and Tika used, was employed in this research. The calibration of the driving system is presented in Appendix I.

3.3 Sample Preparation

The preparation techniques which were used varied for the different soil types which were tested. The natural soils, i.e. brown London clay and Cowden till, were air-dried, mechanically crashed, sieved through the 425 μm sieve, remoulded with distilled water to a water content around their plastic limit, and left overnight to hydrate before they were placed into the confining rings. The commercially available soils, i.e.

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kaolin and pure silica, were just remoulded with distilled water, and left to hydrate. The artificial sandy silty was prepared by mixing thoroughly in a dry state three parts of pure silica with one part of clean fine sand, remoulding the mixture with distilled water, and leaving it to hydrate overnight. The initial water contents of each sample are presented in Table 5.2.1. After hydration, the samples were placed into the confining rings by hand and compacted to avoid trapped air voids and reduce the decrease in sample height during the consolidation stage. Furthermore, the techniques which were presented by Bishop *et al* (1971) for the placement of remoulded samples in the ring shear apparatus were followed. Finally, the sandy samples were sieved through the required aperture sieves, submerged in water, stirred thoroughly to release possible trapped air, and scooped into the confining rings.

3.4 Testing Procedures

In order to minimise the variables under consideration when analysing the test results, the same general testing procedure was followed in all tests apart from those which were conducted on sand.

Unless otherwise stated, the *multistage testing technique* (e.g. Anderson and Hammoud, 1988) was adopted in order to investigate the effect of increasing normal stress on the behaviour of the same soil sample. According to this technique the sample is consolidated to the lowest normal stress at which it is to be tested, and sheared. After the shearing stage is terminated, the sample is unloaded in shear, the normal stress is increased to the next level, and the procedure is repeated until the shearing stage of the highest normal stress at which the sample is to be tested. In this investigation each normally consolidated sample was tested at 50, 100, 200 and 400 kPa. In some cases, where the soil loss led to a substantial decrease in sample height, the last stage was conducted at 300 or even 250 kPa in order to avoid damaging the load cells. The over-consolidated samples were consolidated at 400 kPa, unloaded to 100 kPa, and tested at 100, 200 and 300 kPa, unless otherwise stated. Furthermore, for each soil type, two multistage tests were conducted on separate, but identically prepared, normally consolidated samples in order to investigate the effect of the presence of water in the environment of the sample. Hence, one sample was tested with the water bath of the ring

shear apparatus empty (***non-submerged***), and another with the water bath full (***submerged***). Preliminary tests indicated that the apparatus was developing excessive vibrations when over-consolidated samples were tested with the water bath empty and this type of tests had to be abandoned.

The shearing stage of each test consisted of two parts. The first part (***slow*** or ***drained***) involved shearing the sample at a constant rate of displacement of 0.09 mm/min. Preliminary tests on London clay and Cowden till indicated that the measured *slow residual strength* of soils is, for all practical purposes, the same when samples are sheared at rates of displacement which range between 0.005 mm/min and 0.09 mm/min. Consequently, the higher end of that range was adopted in order to accelerate the testing procedure and produce large shear displacements. The second part (***fast*** or ***undrained***) of each test involved shearing the sample at rates of displacement which were continuously varied between 0.01 cm/sec (6 mm/min) and 10 cm/sec (6000 mm/min). The range of variation of the rate of displacement was controlled by limitations imposed by the motors of the ring shear devices. The two parts of the shearing procedure were separated by a pause in shearing which was also imposed by the limitations of the driving system. In order to increase the rate of displacement from 0.09 mm/min to the desired higher levels, the gear had to be changed. Thus, shearing was interrupted, the sample was unloaded in shear (i.e. the shear stress was zeroed), the "slow" gear was disengaged, the "fast" gear was engaged, and shearing was restarted. Also, when the rate of displacement was to be increased above 1.9 cm/sec (1,100 mm/min) in Ring 2, the same procedure had to be repeated in order for the "faster" (upper) motor to be engaged directly to the rotating base of the apparatus [Fig. 3.4.1].

Deviations from the standard testing procedure involved ***three-stage***, ***"intact" sample***, and ***reversed direction*** testing. The three-stage testing procedure involved an additional slow shearing part on the standard procedure. In this procedure, after the fast shearing part was terminated, the "fast" gear was disengaged, the "slow" gear engaged, and shearing restarted at a rate of displacement of 0.09 mm/min until the measured strength reached a steady value. The "intact" sample testing procedure was similar to the standard procedure with the exclusion of the slow shearing part. In this procedure the sample was sheared fast immediately after the end of the consolidation stage. This procedure was also followed in the tests on sand. Finally, the reversed direction testing

procedure was identical to the standard procedure apart from the fact that, during the fast shearing part, the direction of shearing was repeatedly reversed before the test was terminated.

3.5 Calculations of Results and Errors

The equations for the calculation of the ring shear test results have been summarised by Tika (1989), and they are based on the assumption that the normal and shear stresses are distributed uniformly across the shear plane. It has been suggested (Bishop *et al* 1971) that this oversimplification of the stress distribution on the shear plane introduces an error which could be considered insignificant, and may be preferred over more complex stress distributions which do not reduce significantly the error in the calculation of results. Therefore, the average shear and normal stresses are given by the following equations:

$$\sigma_n = \frac{W_n}{\pi \cdot (r_2^2 - r_1^2)} \tag{3.1}$$

$$\tau = \frac{3M}{2\pi \cdot (r_2^3 - r_1^3)} \tag{3.2}$$

$$\tan\phi = \frac{\tau}{\sigma_n} = \frac{3M \cdot (r_1 - r_2)}{2W_n \cdot (r_1^2 + r_1 r_2 + r_2^2)} \tag{3.3}$$

$$W_n = 10L + P \pm \Delta W \tag{3.4}$$

$$M = \frac{(F_1 + F_2)}{2} \cdot l \tag{3.5}$$

where W_n is the net normal load on the sample,

M is the torsional moment,

r_1 and r_2 are the internal and external sample radii respectively,

L is the load on the hanger of the lever arm,

P is the weight of the upper loading platen,

ΔW is the side friction between the sample and the upper confining rings,

F_1 and F_2 are the measurements of the load cells, and

l is the horizontal distance between the two load cells.

Therefore, if one considers that $r_1 = 50.8$ mm., $r_2 = 76.2$ mm. and $l = 480 \pm 0.2$ mm, and using equations (3.4) and (3.5), equation (3.3) can be written as follows:

$$\tan\phi = \frac{\tau}{\sigma_n} = 3.7298\frac{F_1 + F_2}{10L + P \pm \Delta W} \tag{3.6}$$

Furthermore, the average displacement at the mean radius of the sample may be given by the following equation:

$$\delta = \frac{(r_1 + r_2)}{2} \cdot \theta \tag{3.7}$$

or $$\delta = 1.1083 \cdot \theta^{o} \tag{3.8}$$

where θ and θ^{o} are the angular displacement in radians and decrees respectively.

The most important errors involved in the calculation of the results of the ring shear test are associated with: (a) the non-uniform distribution of stresses, (b) the additional friction which is developed in the gap between the upper and lower confining rings, and (c) the side friction which is developed between the sample and the upper confining rings. Item (a) is discussed in Appendix I, whereas (b) and (c) are considered in Chapter 4 where the *"new" confining rings* are being described.

3.6 Soil Classification Tests

3.6.1 Particle Size Distribution

The particle size distribution of each soil type which was used in this investigation was determined by wet sieving, when necessary (Test 7(A), BS 1377:1975), and by the hydrometer method (Test 7(D), BS 1377:1975).

3.6.2 Index Properties

The index properties were determined according to BS 1377:1990. The plastic limit was determined according to BS 1377:2:1990, and the liquid limit by means of the cone penetrometer (Test 2(A), BS 1377:1990).

3.6.3 Specific Gravity

The specific gravity of the fine grained soils was determined according to BS 1377: 1990 (Test 6(B), Density bottle method).

3.7 Thin Sections

The thin sections from samples which had been tested for the purposes of this investigation were prepared and examined in the manner described by Tchalenko (1967). After the multistage test on a particular sample was terminated, the confining rings with the soil sample and the porous stones were separated from the apparatus. The sample was pushed gently out of the confining rings, and the porous stones were separated from the sample with the help of a pair of sharp blades. The procedure was completed in the shortest possible time in order to prevent loss of moisture. Subsequently, the sample was submerged in melted Carbowax 6000 at 65°C, and was left into an oven set at that temperature for three weeks. After being removed from the Carbowax bath, the sample was allowed to cool, and was then sent for the thin sections to be prepared.

3.8 Summary

The ring shear apparatus, the instrumentation, and data recording system which have been used for the purposes of this investigation has been presented in this chapter. Moreover, the sample preparation and testing procedures, and the equations for the calculation of experimental results which have been employed have also been described. Finally, the classification tests conducted have been listed and the method of preparation of thin sections has been briefly described.

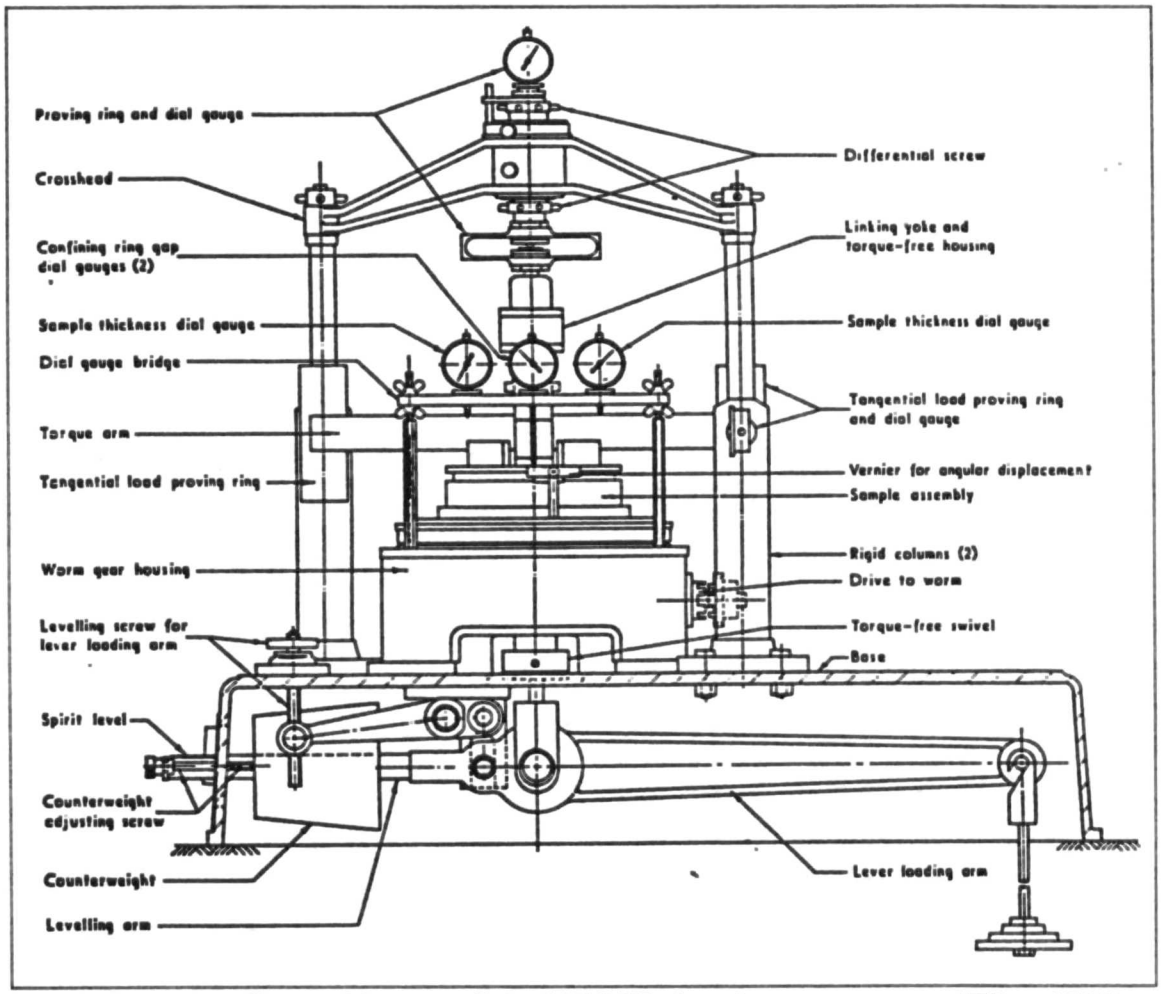


Figure 3.2.1 General layout of the IC/NGI ring shear apparatus (Tika, 1989)

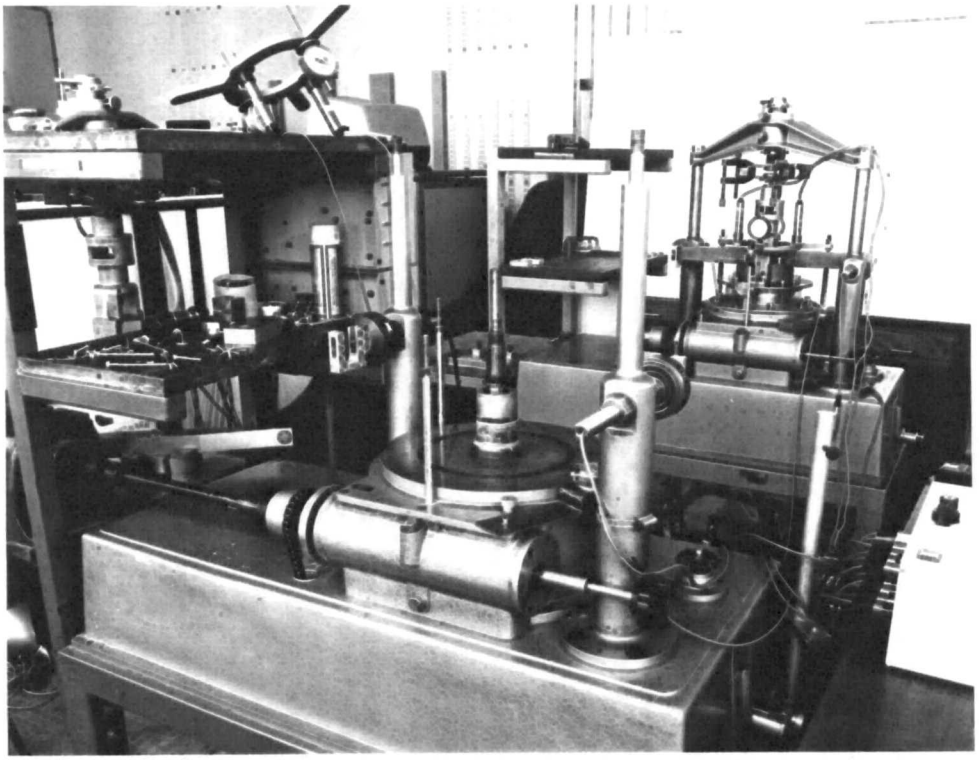


Figure 3.2.2 Ring 1 (background) and Ring 2 (front)

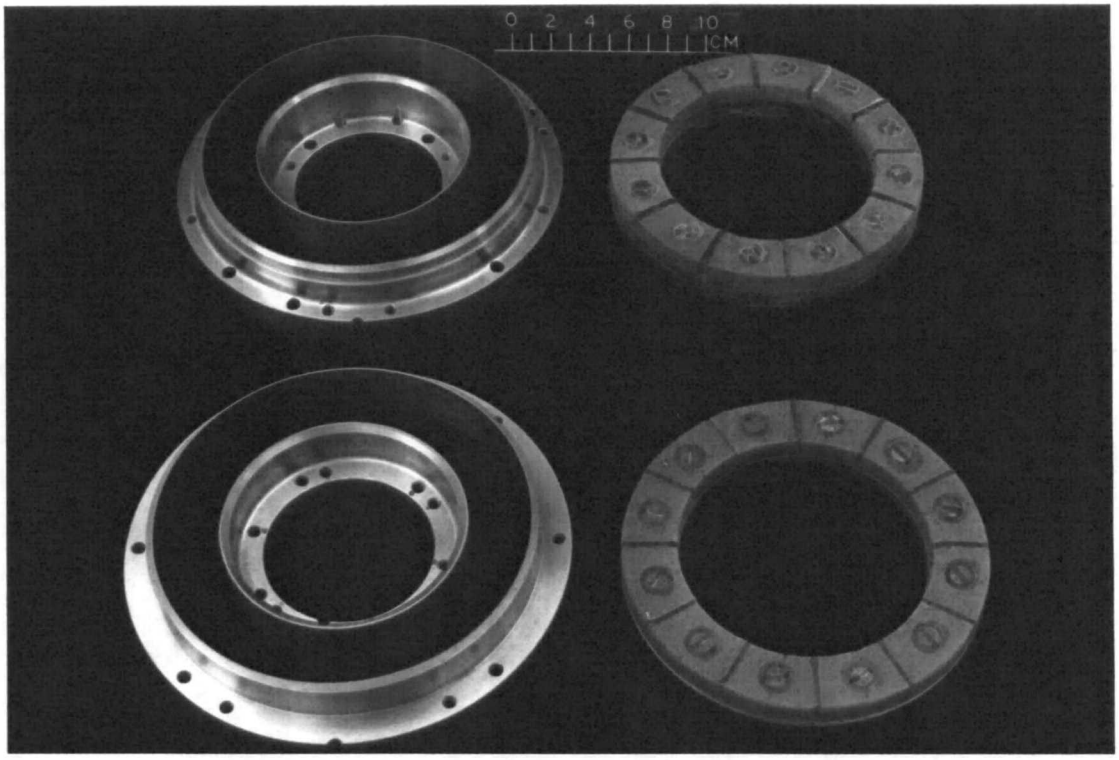


Figure 3.2.3 Original confining rings and porous stones

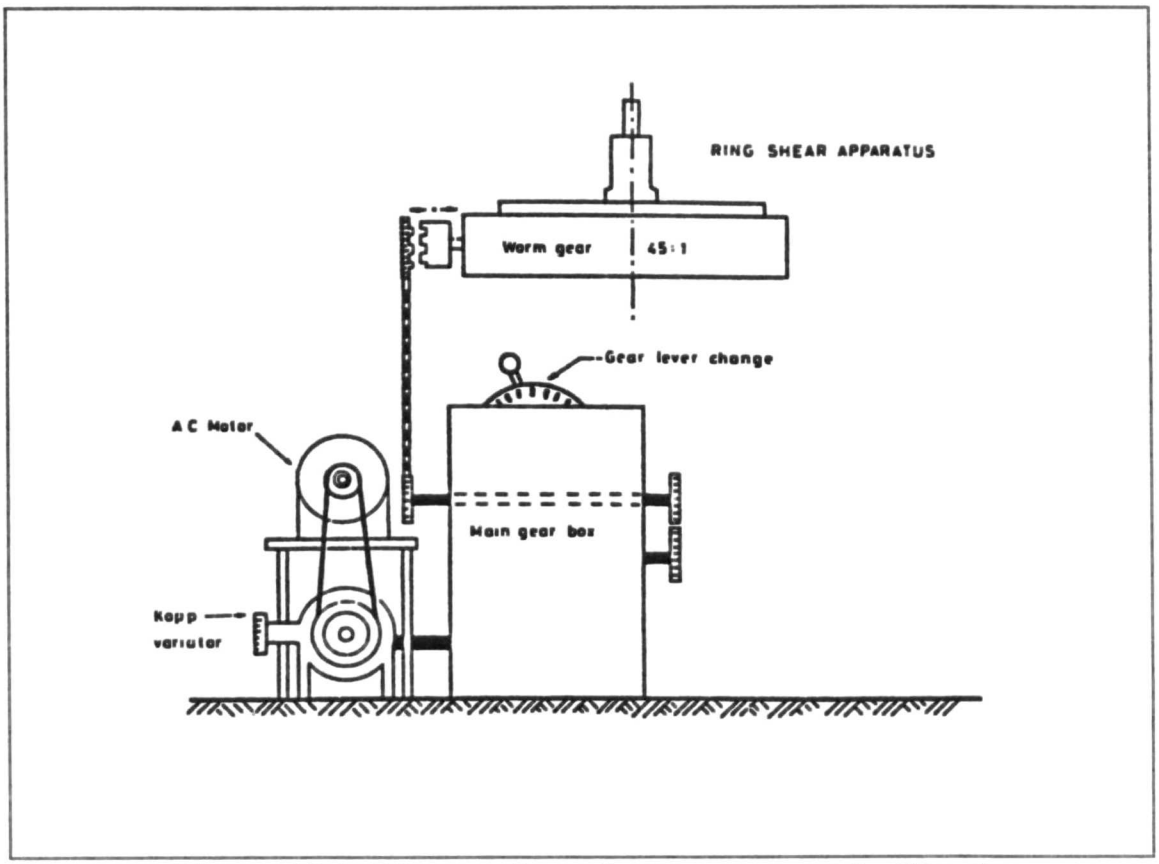


Figure 3.2.4 Motor and gear box arrangement for Ring 1

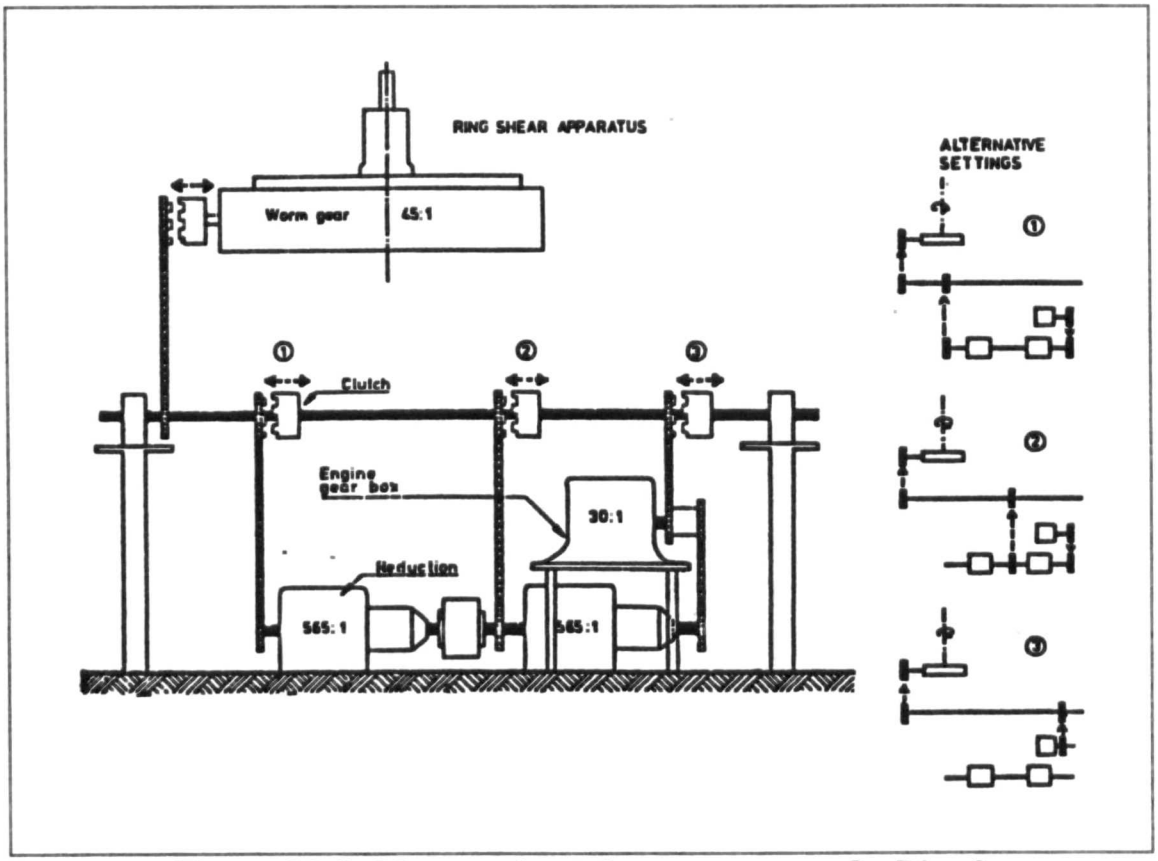


Figure 3.2.5 Motor and gear box arrangement for Ring 2

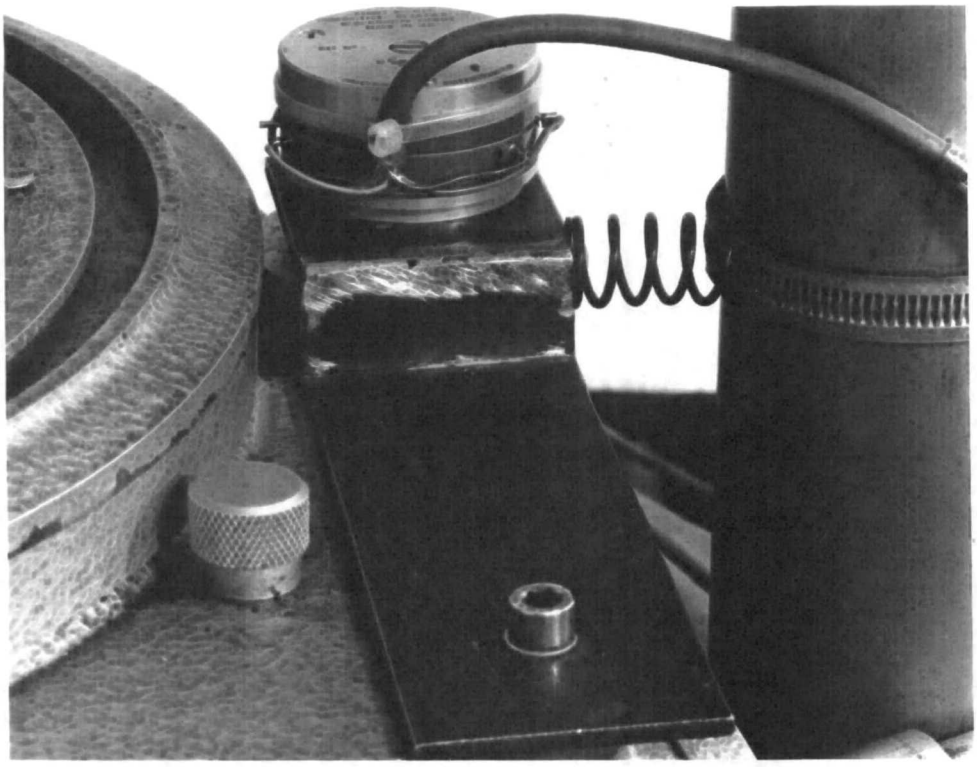


Figure 3.2.6 Horizontal displacement transducer



Figure 3.2.7 Overview of the complete system

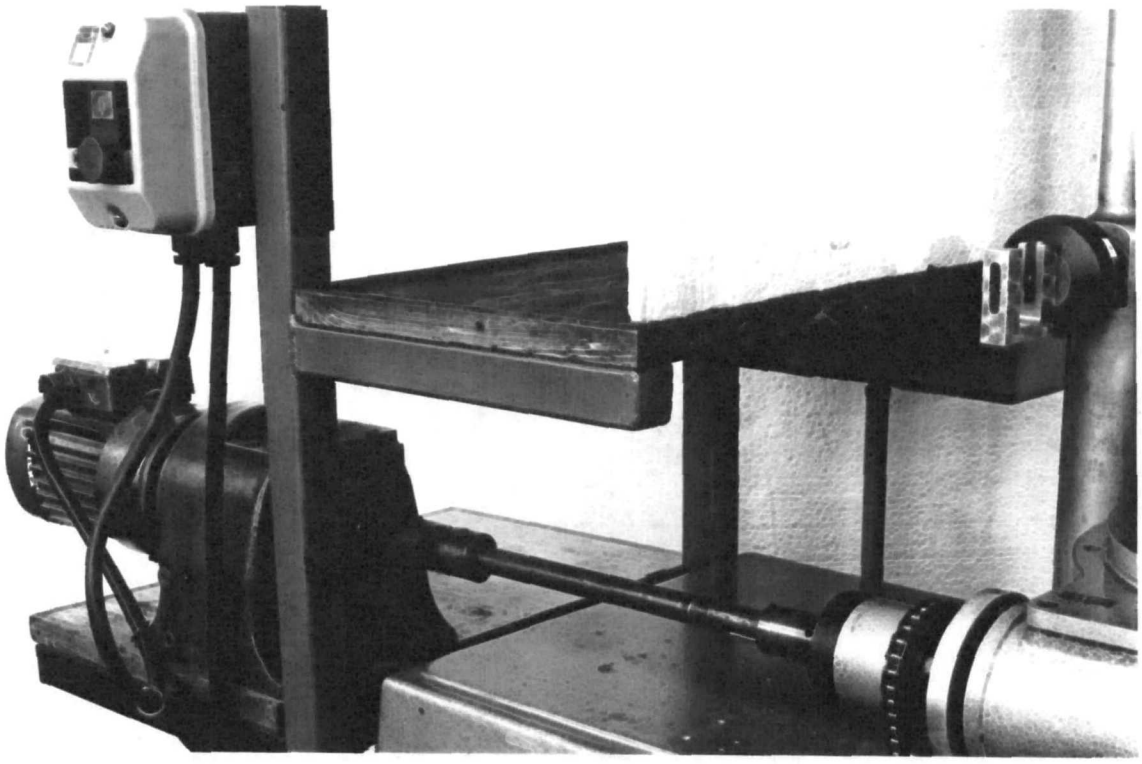


Figure 3.4.1 *"Fast"* motor of Ring 2

CHAPTER FOUR

A NEW DEVELOPMENT ON THE IC/NGI RING SHEAR APPARATUS

4.1 Introduction

The IC/NGI ring shear apparatus has been developed for the assessment of the residual strength of soils. Therefore, its use was intended to be associated with drained rates of displacement (0.001-1.0 mm/min), and total shear displacements within a range between 1 and 10 cm. Under these conditions, the behaviour of the apparatus was considered satisfactory, and no indications of significant soil loss through the gap between the upper and lower confining rings were encountered. Furthermore, at such slow rates of displacement, and for such relatively small shear displacements, the decrease in sample height is minimal, and any side friction which develops between the inner walls of the upper confining rings and the sample is limited. Because of its ability to provide, theoretically, limitless shear displacements at varying rates of displacement, the IC/NGI ring shear apparatus has been employed in investigations of the behaviour of soils at earthquake induced rates of shearing, which may be of the order of metres per second, for large shear displacement, which may be tens of metres.

Extensive use of the IC/NGI ring shear apparatus by previous researchers at Imperial College (Lemos, 1986 and Tika, 1989) has led to the conclusion that the duration of fast shearing tests is severely limited by the excessive amount of soil which is lost through the gap between the upper and lower confining rings. In this chapter the efforts to minimise the soil loss will be presented, and the modifications to the design of the confining rings of the ring shear apparatus which allow prolonged testing at earthquake induced rates of shearing for large shear displacements will be introduced. Furthermore, the evaluation procedure for the compatibility and efficiency of the new design will be described.

4.2 Critical Review of the IC/NGI Ring Shear Apparatus

The basic advantage of the IC/NGI ring shear apparatus is that it minimises additional soil-metal friction which develops between its rotating and stationary parts, and could influence the test results, and moreover it measures the side friction which develops between the dilating or contracting sample and the upper confining rings. As far as slow shearing (0.001-1 mm/min) tests to evaluate the residual strength of soils are concerned, the gap can be successfully controlled to a constant width, and the soil loss minimised. Even though there has been criticism of the mechanics of shearing, mainly due to the high local concentrations of stresses at the fins attached to the porous stones, the uncertainty concerning the directions of principle stresses, and the non-uniform shear and normal stress distributions over the sample, it has been suggested (Lupini *et al.*, 1981) that, at large shear displacements and low strain rates, these sources of error do not affect significantly the estimated residual strength. It is, in fact, at high rates of displacement and prolonged stages of fast shearing that the IC/NGI ring shear apparatus is not performing properly.

The basic problem lies in the fact that the mechanism which controls the gap between the upper and lower confining rings, namely the proving ring and the differential screw, is fixed at one point at the top of the frame of the apparatus [Fig. 4.2.1], and that no restrain is imposed on the horizontal movements of the upper confining rings. Furthermore, the stiffness of the proving ring is controlled by its purpose of measuring the side friction. Therefore, one could argue that the system is flexible enough to allow for both vertical and horizontal movements of the upper confining rings. It should be stressed that the controlling mechanism seems to be adequate for tests to measure the residual strength of soils, because the horizontal and vertical movements which arise during such tests may be considered negligible. However, when the rate of displacement is increased during a test, the matter becomes more complicated. As the sample dilates upon fast shearing, the size of the gap increases, and soil exits at rates which increase with increasing normal stress and shear displacement. Furthermore, the absence of a lateral restraint on the horizontal movements of the controlling frame leads to misalignment of the upper confining rings with respect to the lower ones, causing more sample loss. It has been observed during testing that a combination of the previously

described situations may cause a non-uniform reduction of the sample height, which in turn may lead to a non-uniform distribution of the normal stress acting on the shear zone.

The problem of soil loss is exaggerated when testing fine-grained soil samples. As soil flows out of the gap, the material of the shear zone is continually altered, influencing greatly the measured shear strength, and controlling the duration of the fast stage of shearing. Therefore, in order to avoid losing all the sample, the duration of the test and the shear displacement have to be minimised. When the ***multistage ring-test technique*** (e.g. Anderson and Hammoud, 1988) is employed in order to accelerate the production of results, and fast stages of shearing are introduced at each normal stress level in order to investigate rate effects, the amount of sample that is being lost through the gap limits the cumulative shear displacement and the number of normal stress levels for which quality tests can be conducted. Obviously, solving the problem of soil loss is inevitably connected with being able to conduct long duration, high rate of displacement tests on any soil type. Several attempts by previous researchers to tackle the problem enjoyed limited success.

Hungr and Morgenstern (1984) presented a set of high rate of displacement tests on medium to coarse-sized sands on their *'new'* apparatus design. A gap of about 0.5 mm was adequate to prevent sample loss when clean sands were tested, but when rock flour and sand mixtures were tested, sample extrusion was experienced. Lemos's (1986) effort to solve the problem by clamping the proving ring and thus stiffening the gap controlling mechanism was not successful either and it was abandoned. In the recent years the sophisticated, fully automated, ring shear apparatus presented by Sassa (1992) apparently offers better control over the size of the gap by means of a servomotor. However, in view of the fact that the existing apparatus had to be employed for the purpose of this investigation, the possibility of severely altering its design was excluded from the early planning stages. Having recognised the problem, the effort of this researcher focused on producing a modification on the original design that would minimise soil loss and allow multistage, long duration, fast shearing tests on fine grained soils.

4.3 Preliminary Efforts to Minimise Soil Loss

The idea of Lemos (1986) to clamp the proving ring in order to restrict the movement of the upper confining rings was first implemented. However, after several trial tests it was noted that, while clamping restricted the expansion of the proving ring and, thus, the downward movement of the upper confining rings, it did not prevent the proving ring from contracting, thus, allowing the confining rings to move upward and the gap size to increase. In order to restrict any upward movement, it was decided that two cubes of cast iron would be placed inside the opening of the proving ring. Furthermore, two sets of toolmaker's clamps were used to tighten the proving ring with the cast iron cubes [Fig. 4.3.1] and also restrict any downward movement. The problem that was associated with this approach in controlling the size of the gap was that the proving ring was not functioning the way it was intended, and other methods for measuring the side friction would have to be employed. However, after a series of trial tests on clayey samples it became evident that the size of the gap could not be successfully controlled, and that the lack of lateral support allowed horizontal movements of the upper confining rings which caused additional sample loss. Thus, this approach for solving the problem of soil loss was abandoned, and it was decided to design a set of confining rings (referred to, hereon, as **"modified rings"**) which would restrict horizontal movements, and contain the soil sample during fast shearing. This approach was thought to provide the most feasible solution to the problem since it would entail the minimum modification of the apparatus at the minimum cost.

Upon investigating the possible cross sections for the modified rings it was suggested that the same gap controlling mechanism, *i.e.* proving ring and differential screw, would be employed. Furthermore, it was decided that, since in the original design the gap between the confining rings allows the sample to flow freely into the water bath, the modified design ought to provide a more restrictive path that the sample would need to travel in order to escape. Also, a lateral support would have to be provided in order to minimise the horizontal movements of the upper confining rings. Accordingly, three sets of cadmium-plated steel confining rings [Fig. 4.3.2] were manufactured. The basic idea was to fit the upper rings onto the lower rings so that they could rotate relative to one another without being necessary for the operator of the apparatus to have to open the

gap during shearing. The internal metal-metal friction which would develop along the interfaces of the rings in contact, would have be measured and subtracted from the shear loads recorded during testing.

The first set of modified rings [Fig. 4.3.2.a] provided the lateral support necessary to restrict horizontal movements of the upper rings, and an extended path which the soil would have to follow in order to escape. It was hoped that, because of the restrictions imposed on the sample, the soil loss would be greatly decreased. However, because the plane of relative movement between the upper and lower confining rings was at the surface of the lower porous stone, the shear surface was not expected to be in the middle of the sample, as it has been the case when the original rings were used, but at some point above the fins of the lower porous stone. That meant that the mechanics of shearing would not be the same with the original design, and that a modified analysis of the test results would have to be considered. However, the first point of concern was to examine the efficiency of the modified rings and their ability to minimise sample loss.

An artificial silty sand sample was remoulded at a water content around its plastic limit, consolidated under 100 kPa, and sheared at 0.1 mm/min. In order to ensure that the gap would not easily open during testing allowing soil to escape, the proving ring was pre-stressed and the controlling frame compressed the upper confining rings. The system appeared to behave properly during the first stages of the test, but when the strength of the sample had past its peak value and had started to decrease, the measured shear stress began to increase continuously until the motor became unable to shear the sample further. On dismantling the rings, it became apparent that soil had entered the gap from points A and B [Fig. 4.3.2.a], rending the modified rings assembly completely rigid.

The second set of modified rings was similar to the original design, but it included two floating rings, one inner and one outer [Fig. 4.3.2.b]. It was hopped that, upon shearing, the floating rings would be able to contain the soil sample and rotate independently of the confining rings. Furthermore, no gap would need to be created before shearing commenced, and the shear surface could develop at any point along the height of the sample. Again an artificial silty sand was used to evaluate the performance of this set of modified rings. The sample was remoulded around its plastic limit, consolidated to 100 kPa, and sheared at 0.1 mm/min. However, as was the case with the first set of modified rings, after the first stages of the test, the measured shear stress

began to increase, and the test was terminated. After dismantling the apparatus it became apparent that soil had managed to escape from the sides of the porous stones and stiffen the assembly of the modified rings at the interfaces between the floating rings and the porous stones.

The third set of modified rings were similar in cross-sectional design to the first set, with the only difference that the "tooth" of the upper confining rings which fitted into the lower ones extended all the way to the bottom of the lower porous stone [Fig. 4.3.2.c]. This set was able to operate under compression from the upper frame of the apparatus, thus making it difficult for the gap to open during shearing. Because of the experience from the two previous designs, it was decided that a special adhesive sealant would be applied along the periphery of the lower porous stone at points A and B [Fig. 4.3.2.c]. The ability of the sealant to adhere to the porous stone, but not to the polished metal, allowed the rotation of the upper confining rings relative to the lower porous stone, and it was hoped that it would prevent soil from escaping. The same material and testing procedure that had been used for the evaluation of the first two designs were again adopted. As it had been the case previously, the measured shear strength started to increase unexpectedly after the first stages of the evaluation test. The test was terminated, in fear of damaging the apparatus, and the rings were dismantled. It was observed that upon shearing the soil had managed to break the sealant, enter the gap between the upper confining rings and the lower porous stone, and stiffen the whole assembly. Therefore, it was concluded that none of the three modified ring designs was successful in minimising soil loss, and it became clear that a new system had to be designed which would enable the control of the gap while bypassing the existing mechanism.

4.4 The "New Rings"

4.4.1 Development of the New Design

After further consideration, a fourth design was completed, and a set of what will be referred to from now on as the **"new rings"**, was manufactured [Fig. 4.4.1]. Previous experience suggested that the size of the gap had to be controlled by means other than the existing mechanism. Thorough investigation of the available possibilities led to the

decision that the gap should be controlled from the rotating base of the confining rings, and, in order to achieve that, the concept of ball bearings was adopted. The original idea was that the upper rings should be clamped onto the rotating base, and rotation was to be facilitated by a set of steel balls running in a V-shaped groove. However, in reality the matter proved to be more complicated, since the dimensions of the new rings were restricted by the set-up of the apparatus. The target of the new design was to control the gap at a uniform and constant width of 20 μm, and provide sufficient lateral support in order to prevent any horizontal movement of the upper frame of the apparatus. Furthermore, the additional soil-metal friction had to be minimised or even eliminated.

Upon developing the new rings, several factors had to be considered. Retaining the uniform and constant gap size of 20 μm was the first point of concern, since there was fear that the forces which arise during a test, as soil dilates or contracts, added to the permanent weight of the frame which keeps the upper confining rings stationary, would cause with time indentation of the groves, non-uniform reduction of the gap size, and eventually reduce the effectiveness of the system. A second point of concern was the fact that the rings had to operate submerged in water, so rust prevention was also essential. The use of stainless steel alloys as materials for the manufacturing of the new rings had to be excluded from the beginning, because it was essential to insure maximum hardness in order to prevent the gap size from altering. It was suggested by the technicians of "Tsiricos S.A.", were the new rings were manufactured, that it would be difficult to achieve the required hardness by chemically treating any stainless steel alloy, and even if it was managed, the deformations due to the hardening procedure of the individual pieces which formed the new rings would be too severe to be corrected by machining each individual piece, separately, after hardening. Therefore, it was decided that the individual pieces which constitute the new rings [Fig. 4.4.2] would be manufactured of steel, hardened to achieve a maximum hardness of 62 Roc, and machined so that any deformation induced by the hardening procedure could be corrected. Also, it was decided that the groves within which the steel balls would be running should have a semi-circular shape in order to avoid high stress concentrations on the points of contact. Furthermore, the new rings had to be chromium plated in order to avoid rusting. The minute size of the gap, and the clearances that were achieved between the individual pieces, prohibited the chromium plating of any of the horizontal faces or the groves, as it

was thought that the thickness of the plating would alter the dimensions of the gap, and the concentrated stresses in the groves would eventually break the plating and induce friction. Therefore, all vertical faces of the rings were chromium plated, and it was decided to take the rings apart after each test and thoroughly clean them in order to prevent rapid deterioration.

4.4.2 Description and Setting-up of the New Rings

Each one of the inner and outer new rings comprise of four individual parts [Fig. 4.4.2], namely, the *base rings* (front left), the *intermediate rotating discs* (front right), the *safety discs* (back left), the *upper rings* (back right), and two sets of steel balls which facilitate rotation. The base rings [Fig. 4.4.3] are bolded onto the rotating table of the ring shear apparatus. A set of steel balls (5 mm in diameter for the outer ring and 4 mm for the inner ring) are placed in the lubricated grove of the base ring [Fig. 4.4.4], and the intermediate rotating disc is fitted on top of it [Fig. 4.4.5]. In order to avoid additional soil to metal friction as shearing progressed, the base rings included a 4x10 mm *depositing chamber* along their periphery [Fig. 4.4.6] into which soil can be deposited without making contact with the intermediate rotating disc. The gap between the intermediate rotating disc and the rim of the depositing chamber [Fig. 4.4.7] is controlled at 20 μm. That rim was made 4 mm thick in order to further restrain the exiting soil even if that had to result in increased soil-metal friction along its width. A second set of steel balls is placed in the lubricated upper grove of the intermediate rotating disc [Fig. 4.4.8], and the safety disc is fitted on top of it and locked by a set a screws at the three equidistant 40x8x13 mm columns of the base ring [Fig. 4.4.9]. Finally, the upper ring is screwed onto the intermediate rotating disc [Fig. 4.4.10], a nickel-platted steel tube is fitted onto the rotating base in order to contain the water bath, and the whole assembly is placed onto the ring shear apparatus [Fig. 4.4.11]. The upper rings (inner and outer) are locked into position by the existing frame [Fig. 4.4.12], and the base rings along with the safety discs can rotate without any additional friction.

It is important to note at this point that each individual part of the rings was machined separately in order to provide sufficient clearances and ensure no metal-metal friction. Furthermore, it should be emphasised that the forces which are imposed on the

rings by the frame of the apparatus are not uniformly distributed along their periphery, and the dilation or contraction of the sample during testing also produces non-uniform tensional or compressive stresses on the inner walls of the rings. Thus, it was essential to ensure during manufacturing that the rings could operate without friction under any stress combination which might be applied externally.

4.4.3 Gap Friction and Side Friction Corrections

Before any testing could be conducted on the new rings, two further factors had to be considered, namely *gap friction* and *side friction*. The gap friction is the soil-metal friction which develops over the width of the rim of the depositing chamber when soil enters the 20 μm gap between the base rings and the intermediate rotating discs, while shearing progresses. Since all soil which is squeezed through the gap falls into the depositing chamber and causes no further friction, it is considered that, once the gap is contaminated with soil, the gap friction remains relatively constant throughout the duration of a test. Therefore, it can be measured and subtracted from the results after each test is completed.

Measuring the gap friction after the end of each test is done by means of a frictionless dummy sample. This dummy sample comprises two brass rings [Fig. 4.4.13] which are manufactured so that the top ring can be rotating on a set of steel balls running in a V-shaped grove indented on the bottom ring [Fig. 4.4.14], with no friction regardless of the normal load acting on the top ring. After each test is completed, the sample is unloaded in both shear and normal directions, and the rotating table, confining rings, soil sample and porous stones assembly is removed from the apparatus. The confining rings are detached from the rotating table, and the sample together with the porous stones is pushed gently out of the rings. The vertical faces of the confining rings which come into contact with the sample are carefully cleaned in order to avoid any contamination of the dummy sample upon measuring the gap friction. The bottom ring of the dummy sample is screwed onto the rotating table [Fig. 4.4.15], and is then followed by the confining rings. A set of 8 mm diameter steel balls is placed in the lubricated V-groove of the bottom ring of the dummy sample, and the top ring is placed on them

[Fig. 4.4.16]. The whole assembly is repositioned on the ring shear apparatus along with the loading yoke and the upper frame.

Each gap friction correction test is multistage. The dummy sample is loaded to the minimum normal load under which the sample has been tested, and sheared first at 0.09 mm/min, in order to simulate the slow part of the shearing stage of soil tests, and then at varying fast rates, in order to simulate the fast part of the shearing stage of soil tests. The same procedure is repeated at each normal load under which the soil sample has been tested. After several trial tests, it was concluded that the amount of gap friction is independent of the normal stress level under which the test is conducted, and thus, the dummy sample was sheared only under two normal loads. The shear stresses which are measured during the gap friction correction test are attributed to the soil-metal friction in the gap, and to any further internal friction due to ring contamination, and are subtracted from the shear stresses measured during the soil tests upon calculating the shear strength of each sample.

The second factor that had to be assessed and corrected for is the *side friction i.e.* the friction which is developed between the dilating or contracting sample and the inner walls of the confining rings, and which influences the normal stress acting on the shear surface. When the soil sample dilates, it tents to move upwards relative to the upper confining rings, creating a soil-metal friction (side friction) which tends to increase the normal stress on the shear surface [Fig. 4.4.17.a]. Likewise, when the sample contracts, the side friction acts in the opposite direction and tends to decrease the normal stress on the shear surface of the sample [Fig. 4.4.17.b]. In the IC/NGI ring shear apparatus the measurement of the side friction is accomplished by the proving ring which is connected to the upper confining rings [Fig. 4.2.1]. The main criticism of this side friction measuring mechanism is that the proving ring is compressed further because of the amount of soil which is being extruded through the gap (by squeezing between the lower and upper rings) during testing. Since the lower confining rings cannot move in the vertical direction, the extruding soil displaces the upper rings which are connected to the proving ring and, therefore, the load which is being measured by the proving ring cannot be attributed purely to side friction. It has been suggested (Tika, 1989) that the difference between the measured and the actual side friction is not significant. In the modified design, however, the new rings allow no vertical movement and the proving

ring cannot function in the way it was intended. Therefore, it was essential to develop a correlation between the change in sample height (dilation or contraction) and the side friction measured by the proving ring for each soil type that was to be used in this experimental programme. For that purpose, the original confining rings were employed, and identically prepared and consolidated samples with those tested in the course of this investigation were sheared at rates of displacement that varied between 100 mm/min and 1,000 mm/min. While shearing these samples, the changes in sample height and the side friction which was being measured by the proving ring were recorded. Having established a database of side friction measurements and the corresponding changes in sample height which produced them, it was possible to correct the test results as follows: as the change in sample height was monitored continually during soil testing with the new rings, the corresponding side friction, taken from the side friction correction tests, was subtracted from (if the sample was contracting) or added to (if the sample was dilating) the normal load on the hanger and the corrected normal stress which acted on the shear surface of soil sample was estimated.

4.4.4 Additional Modifications of the New Rings

Having accumulated a substantial amount of experience in the performance of the new confining rings, another effort was made to contain further the soil during prolonged fast shearing stages. The basic principal behind the second version of the new confining ring design was that, in order to absolutely minimise soil loss, the gap had to be kept closed during testing by means of an elastic system which would not induce a large amount of internal friction.

Several mechanisms and materials were considered before it was decided that a carbon flange [middle section, Fig. 4.4.18] would be manufactured, placed inside the depositing chamber and pushed toward the intermediate rotating disc through a set of springs. The advantages of using carbon as the material for the proposed flanges are that when it is being sheared against a harder, smooth material, it develops a polished surface which induces minimum friction and operates similarly under water. The main disadvantages are that this material is very brittle and wears easily. In order to achieve continuous contact between the carbon flange and the intermediate rotating disc, one set

of springs were embedded at the bottom of the depositing chamber [Fig. 4.4.19]. A thin brass flange was placed onto the springs [Fig. 4.4.20], before the carbon flange [Fig. 4.4.21], in order to protect it from the sharp edges of the springs. The purpose of these springs was to push constantly the carbon flange toward the intermediate rotating disc, thus keeping the gap essentially closed at all times [Fig. 4.4.22]. Because of the brittleness of carbon, the width of the flange had to be a minimum 7 mm for the outer ring and 6 mm for the inner ring, and the width of the depositing chamber had to be increased accordingly while the width of its rim was decreased to 1 mm in both cases. However, this second version of the new confining rings had not been fully developed in time for its employment in this experimental programme, and had to be used without the carbon flanges.

4.5 Tests to Evaluate the Efficiency and Compatibility of the New Rings

Before they were tested for their efficiency in minimising soil loss and their compatibility with the original confining rings in measuring the slow residual strength of identically prepared samples, the new rings were subjected to a series of trial tests at normal stresses which varied between 50 and 500 kPa in order to examine whether they were able to shear a sample with prolonged fast shearing stages without exhibiting additional internal friction, and without contamination from extruded soil.

These tests were conducted on samples of blue London clay (available in the teaching laboratory of the Soil Mechanics Section of Imperial College) remoulded to a water content around its plastic limit. The samples were consolidated at 50, 100, 200, and 400 kPa and tested in each normal stress level at rates of displacement which varied between 0.09 mm/min and 6000 mm/min for shear displacements of 2 to 3 m. It was observed that the amount of soil loss, *i.e.* the decrease in sample height, during fast shearing was significantly reduced, while no additional friction due to contamination of the rings appeared to develop as the tests progressed. Therefore, the main evaluation procedure, where the performance of the new rings would be compared with that of the original rings, could follow.

It was recognised from the beginning that the behaviour of the new rings during prolonged fast shearing stages would be inevitably different from that of the original

rings, since the amount of soil which is extruded through the gap alters the material within the shear zone and consequently affects the measured shear stress. Thus, it was decided that the evaluation of the new rings would focus on the comparison between the decrease in sample height upon fast shearing and the measured slow residual strength when both ring types are used in identical conditions.

Three fine-grained soils were used in the main evaluation procedure and one multistage test was conducted on a medium-coarse clean sand in order to assess the consistency of results from the new rings when coarse-grained soils are being tested. The sample preparation procedure which was adopted for the fine grained soils was as follows: the sample was air-dried, mechanically broken down, passed through the 420 μm test sieve, remoulded to a water content around its plastic limit, and left to hydrate overnight. The multistage testing technique was employed in this evaluation programme. The sample, being consolidated at the lower normal stress level at which it was to be tested, was sheared at 0.09 mm/min until the measured strength had reached its slow residual level. Subsequently, the rate of displacement was increased to 900 mm/min in order to observe the response of the sample and measure the decrease in sample height *i.e.* soil loss. After the fast shearing stage was completed, the sample was unloaded in shear, the normal stress was increased to the next level, and the same procedure was repeated either until the sample had been sheared fast under 50, 100, 200, and 400 kPa or until the decrease in sample height was so large that there was fear of damaging the apparatus. Gap friction and side friction corrections were taken into account when the results were calculated and compared.

The first material to be tested was blue London clay, normally consolidated, not submerged in water. A near perfect agreement between the slow residual strength values which were measured using the new rings and those measured using the original rings was observed [Table 4.5.1]. The expected non-linear relationship between slow residual strength and normal stress which resulted from both ring types gave further evidence on the performance of the new rings [Fig 4.5.1]. In this particular set of tests with the new rings, the rate of displacement ranged up to 1.5 cm/sec, producing a cumulative shear displacement of 20.3 m on the same sample. On the other hand, excessive soil loss limited the number of normal stress levels that could be applied to an identically prepared and tested sample when the original rings were used. Therefore, only two fast shearing

stages were completed using the original rings, leading to a cumulative shear displacement of 8.1 m, before the test had to be terminated because there was a danger of damaging the apparatus. The rate at which the soil was exiting from the gap was estimated by the decrease in sample height per unit shear displacement. It can be seen in Table 4.5.2 that the rate at which the soil is being extruded through the gap increases with increasing normal stress for both ring types for this particular soil. In average, when the original rings were used, the sample height was decreasing by 0.7 mm per metre of shear displacement while it was decreasing by 0.17 mm per metre of shear displacement when the new rings were employed.

The second material to be tested was Cowden till normally consolidated, not submerged in water. Unfortunately, excessive soil loss through the gap of the original rings prevented the continuation of the multistage testing procedure beyond the second normal stress level and, thus, comparisons between the slow residual values measured with the two ring types could only be made for the first two normal stress levels (50 and 100 kPa). As it has been the case with the blue London clay samples, there was good agreement between the slow residual values measured with the two ring types. This fact was added to the consistency which was observed in the test results when the new rings were employed to further support their efficiency and compatibility [Table 4.5.1, Fig 4.5.2]. The improvement provided by the new rings is highlighted in this set of tests. When the new rings were used, all four fast shearing stages were completed, producing a cumulative shear displacement of 32.8 m, while there was only 1.7 mm of decrease in sample height attributable to soil lost through the gap. In contrast, only two fast shearing stages were completed with the original rings, producing a cumulative shear displacement of 10.2 m before the test had to be terminated to avoid damaging the load cells. When the new rings were used, the average rate at which the soil was being extruded through the gap was estimated to be 0.05 mm per metre of shear displacement as opposed to 0.35 mm/m when the original rings were used on identically prepared and tested samples [Table 4.5.3]. Overall, it can be concluded that the new rings lose 4 to 7 times less soil than the original rings, while the behaviour of both ring types upon slow shearing is similar. Furthermore, it is important to note that the reproducible shape of the stress-strain curves during slow shearing at each normal stress level when the new rings are employed, along with the consistency of the measured slow residual strength,

highlight the fact that soil contamination of the new rings does not affect their performance.

Two more soils were tested using only the new rings in order to further evaluate the consistency of the results. The results from a multistage test on normally consolidated brown London clay, submerged in water, again showed good consistency [Table 4.5.4, Fig 4.5.3]. The cumulative shear displacement during all the fast shearing stages was just under 20 m and the soil which was lost through the gap was restricted to a minimum, thus, allowing all four stages of the test to be completed. Finally, a medium-coarse clean sand was tested, normally consolidated, not submerged in water. The slow residual strength values showed satisfactory consistency [Table 4.5.4 Fig 4.5.4] and after a cumulative shear displacement of 28.4 m, under fast shearing rates, no sample appeared to have been lost through the gap. However, due to the nature of the sample, excessive vibrations were observed during the fast shearing stages and testing of coarse sands had to be postponed in fear of damaging the equipment.

4.6 Summary

The disadvantages of the IC/NGI ring shear apparatus and the problems associated with it have been presented in this chapter. Preliminary efforts to minimise the amount of soil loss through the gap of the original confining rings which introduces severe limitations on the use of the apparatus have also been described. The factors which led to the development of the new confining rings were discussed and the manufacturing process and description of these rings were presented. Finally, the data correction and evaluation procedures concerning the new confining rings were introduced.

The satisfactory agreement between the slow residual strength values measured with the two ring types and the reproducible behaviour of soil samples upon fast shearing makes evident the quality of the new rings. It has been shown that, typically, the residual strength estimated by means of the new confining rings may vary by 0 to 2 % from that estimated by means of the original rings. Also, it has been demonstrated that the new confining rings lose up to 7 times less soil than the original rings and allow prolonged stages of fast shearing.

It is concluded that the new ring design provides the most efficient means of minimising soil loss and testing samples in prolonged stages at earthquake-induced rates of displacements.

Normal Load (kPa)	Blue London Clay (New Rings)	Blue London Clay (Old Rings)	Cowden Till (New Rings)	Cowden Till (Old Rings)
50	12.4	12.4	39.4	39.7
100	10.2	10.2	34.9	34.6
200	9.1	8.5	31.8	INC[1]
400	9.6	INC[1]	33.4	INC[1]
[1] Incomplete test because of excessive soil loss				

Table 4.5.1 Residual Angles (in degrees) for Blue London Clay & Cowden Till Samples

Ring Type	Normal Stress (kPa)	Shear Displacement (m)	Height Decrease (mm)	Rate of Soil Loss (mm/m)
New	50	5.3	0.6	0.11
New	100	4.8	0.8	0.16
New	200	5.9	1.2	0.2
New	400	4.3	0.9	0.21
Original	50	4.3	2.6	0.6
Original	100	3.8	2.6	0.7

Table 4.5.2 Comparisons using Results from Tests on Blue London Clay

Ring Type	Normal Stress (kPa)	Shear Displacement (m)	Height Decrease (mm)	Rate of Soil Loss (mm/m)
New	50	7.7	0.6	0.07
New	100	9.3	0.5	0.05
New	200	8.6	0.4	0.05
New	400	11.7	0.3	0.03
Original	50	7.2	1.8	0.25
Original	100	3	1.9	0.6

Table 4.5.3 Comparisons using Results from Tests on Cowden Till

Normal Load (kPa)	Brown London Clay (New Rings)	Medium-Coarse Clean Sand (New Rings)
50	14	35
100	11.3	39.4
200	12.3	39.7
400	11.3	40.7

Table 4.5.4 Residual angles (in degrees) for Brown London Clay & Medium-Coarse Clean Sand samples

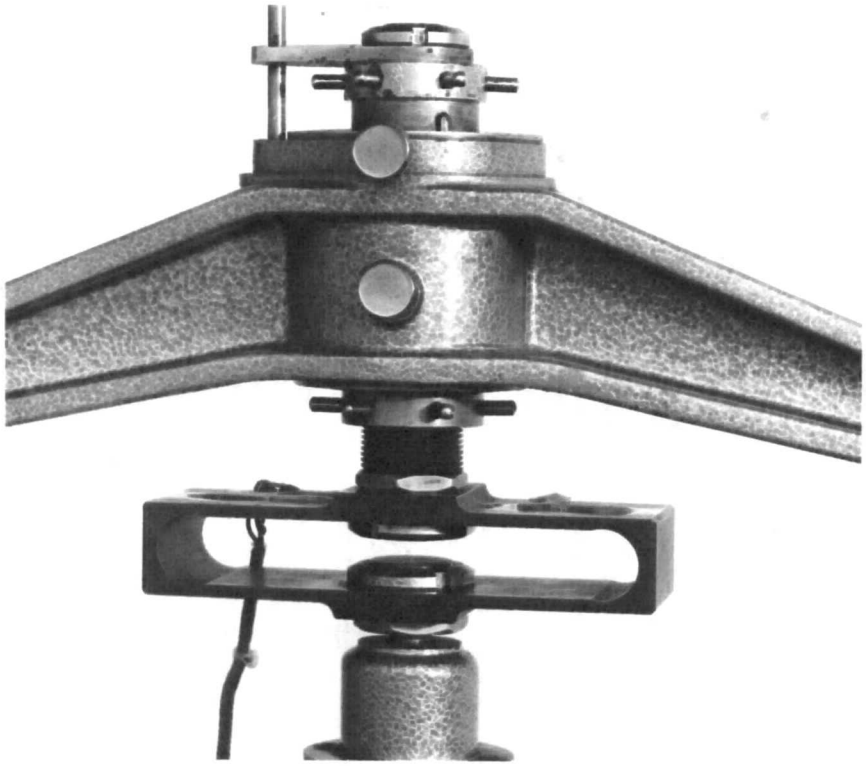


Fig. 4.2.1 Original gap controlling mechanism - Proving ring and differential screw

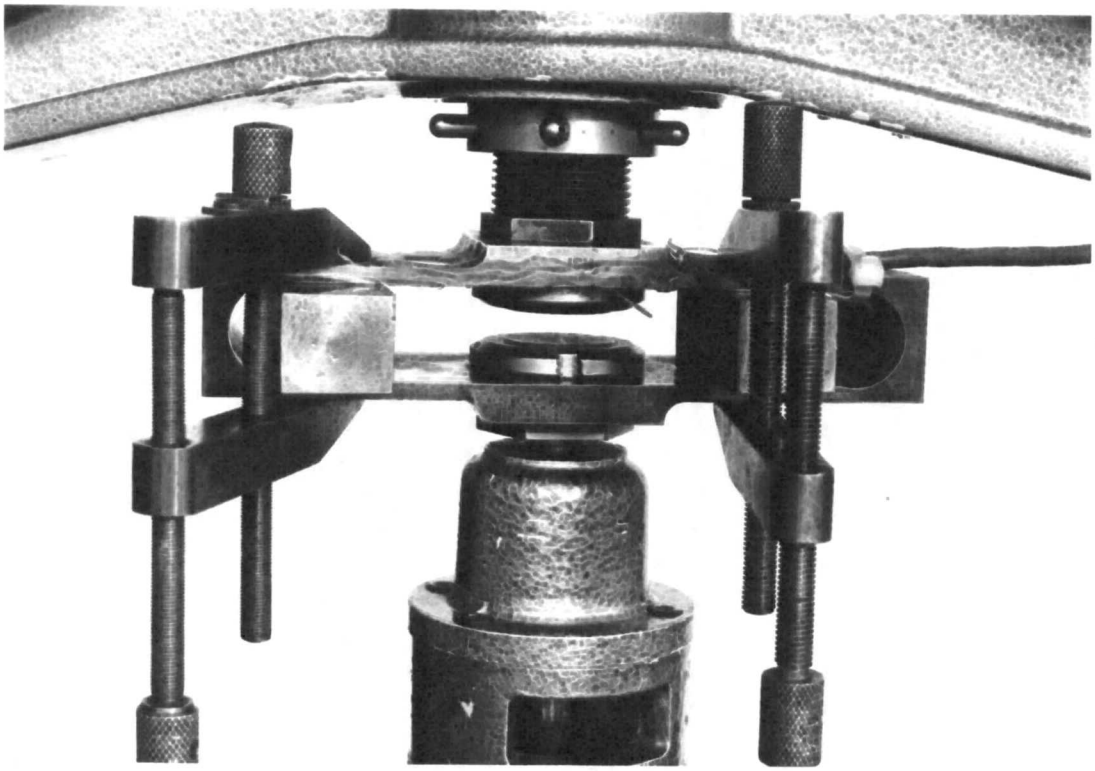


Fig. 4.3.1 Unsuccessful attempt to control gap by stiffening the proving ring

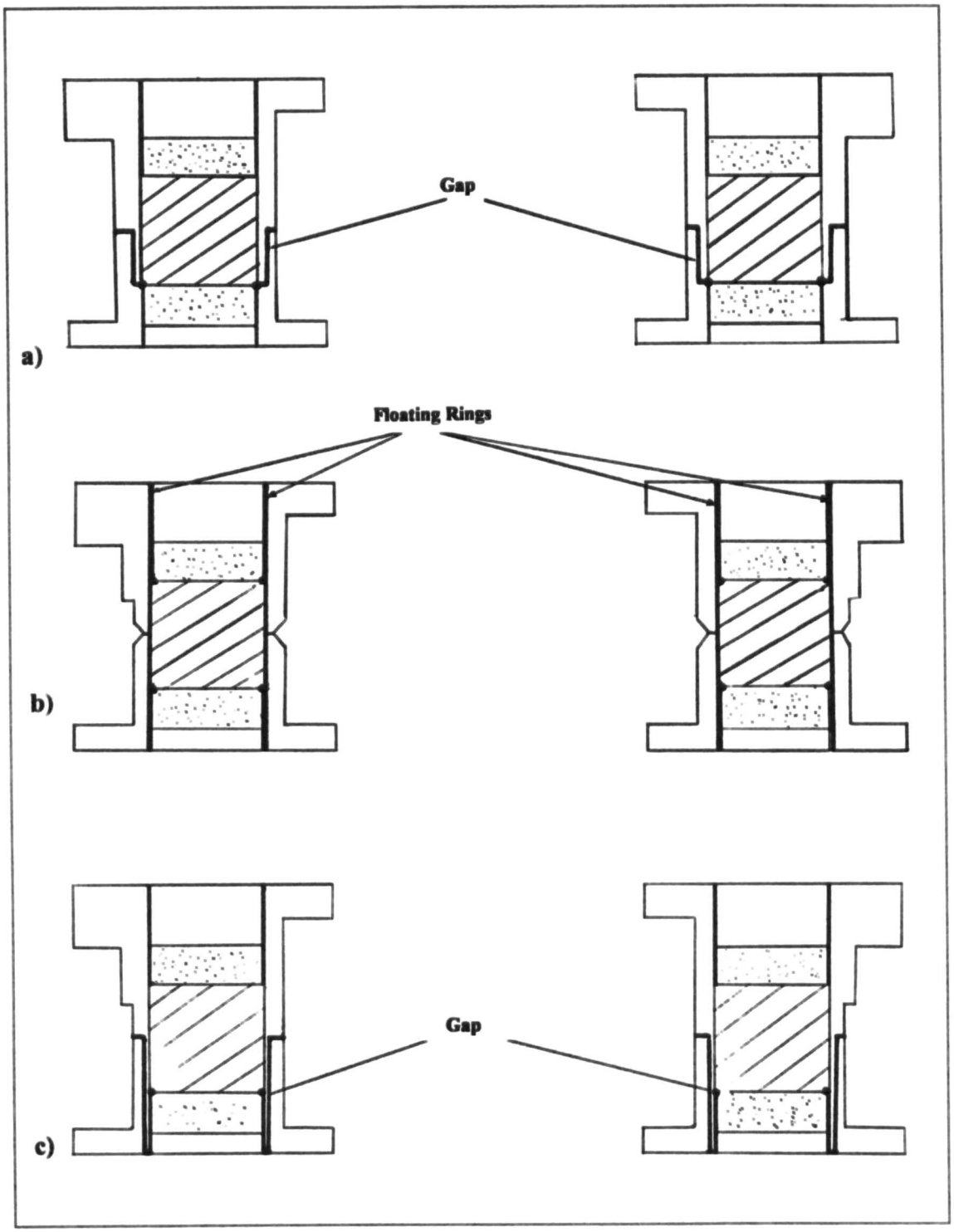


Fig. 4.3.2 Sets of modified confining rings

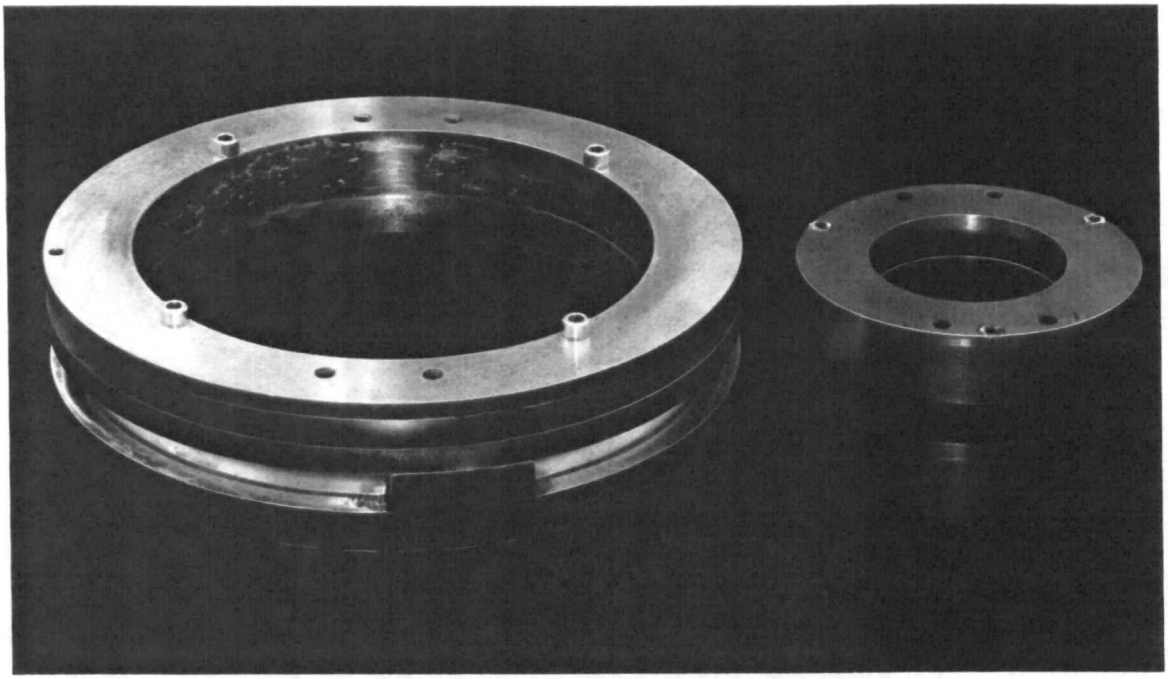


Fig. 4.4.1 New confining rings (assembled)

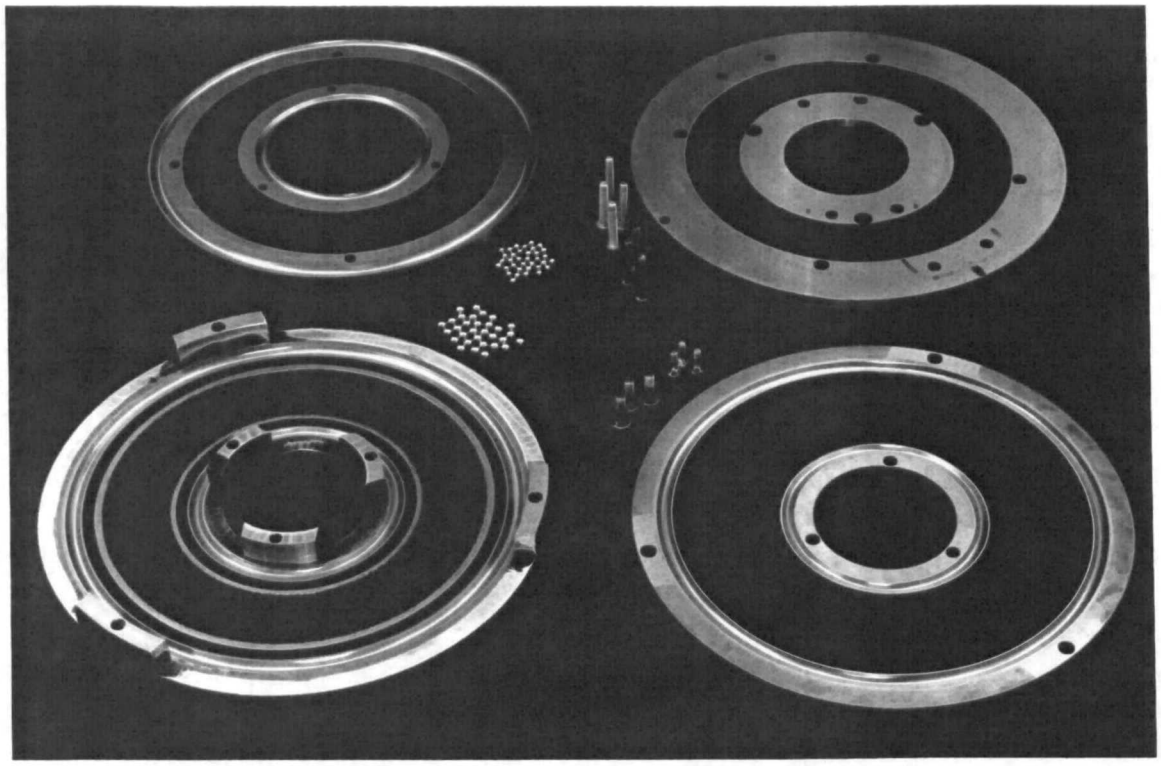


Fig. 4.4.2 Individual parts which constitute the new confining rings

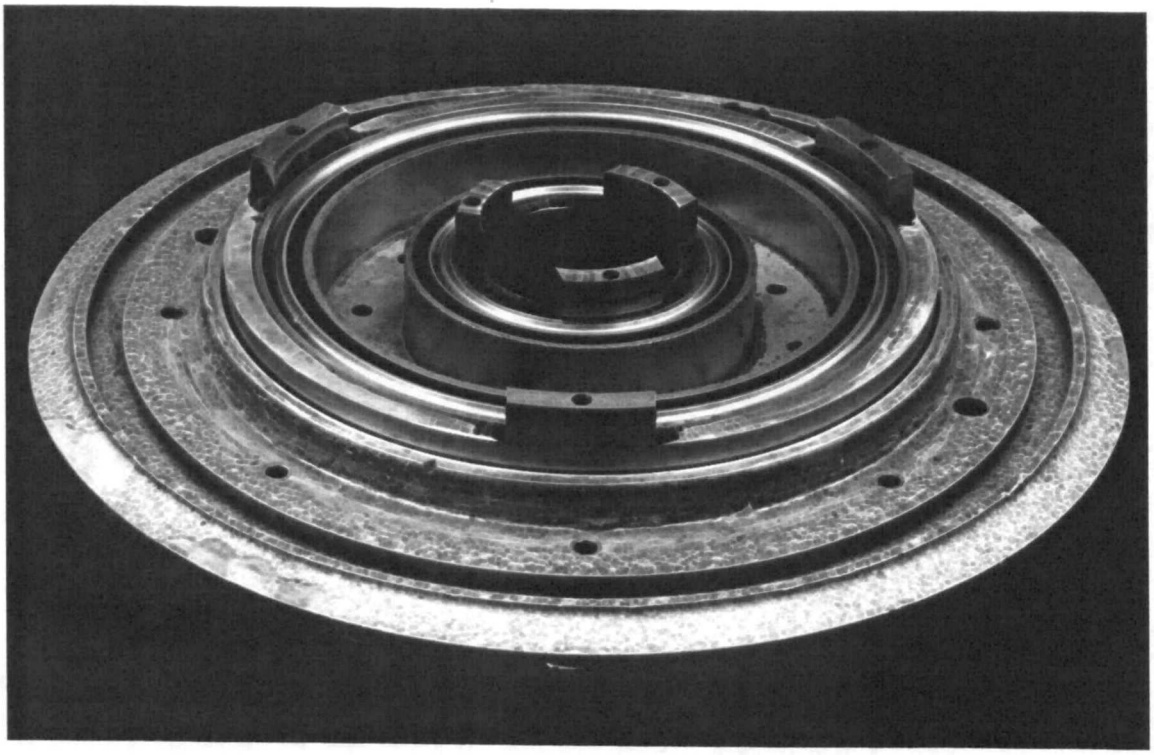


Fig. 4.4.3 Base rings bolted onto rotating table of apparatus

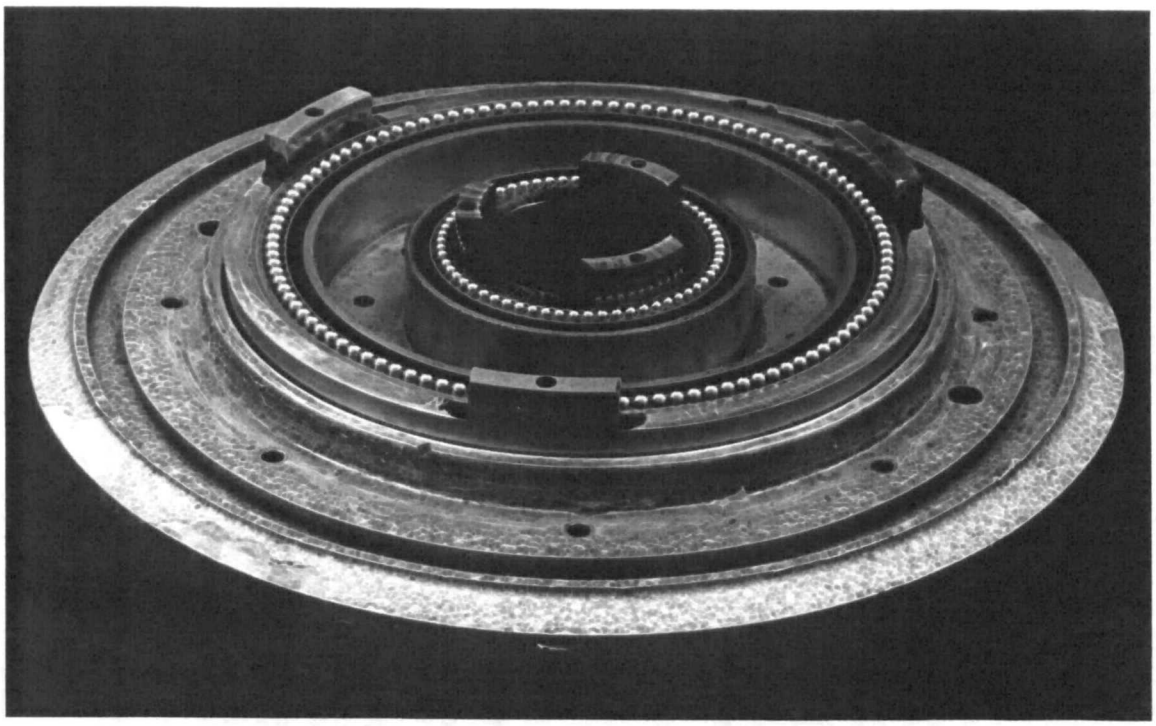


Fig. 4.4.4 Steel balls placed in lubricated grove of base rings

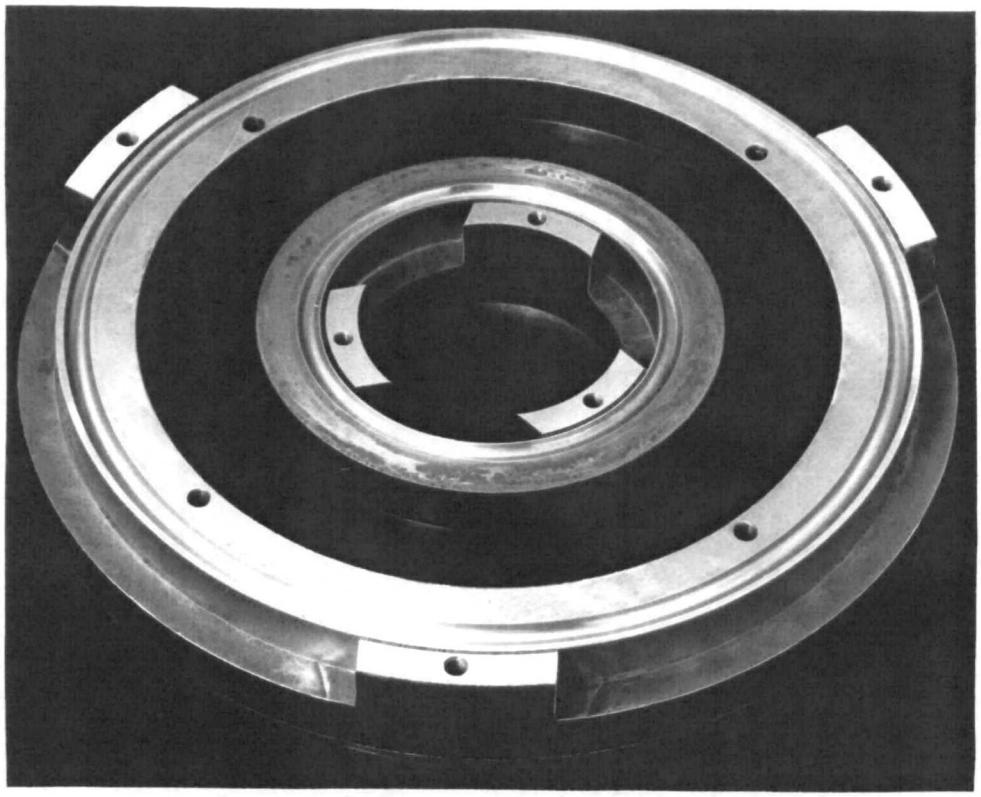


Fig. 4.4.5 Intermediate rotating disk placed on ball-bearing base assembly

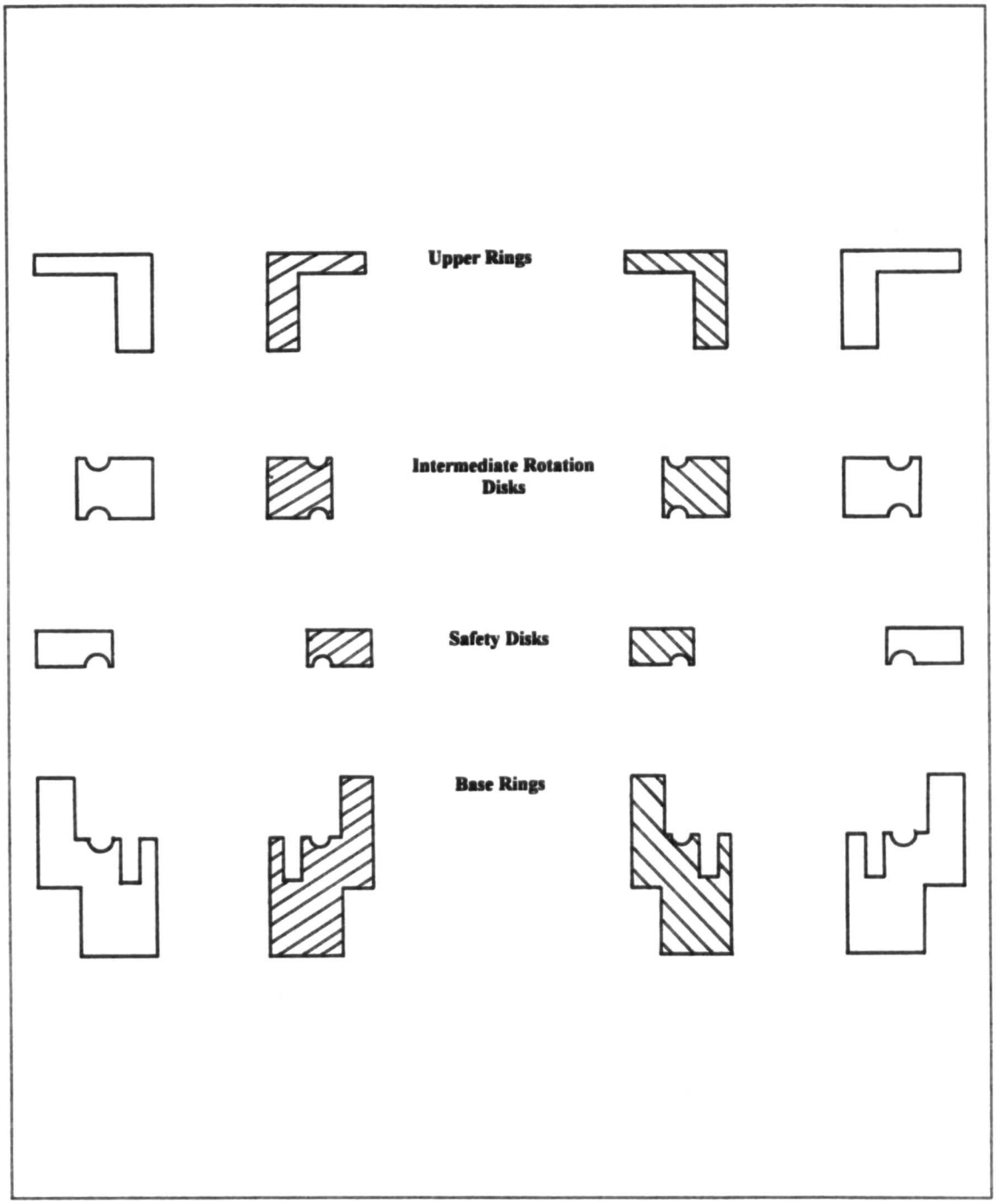


Fig. 4.4.6 Cross sections of individual parts which constitute the new confining rings

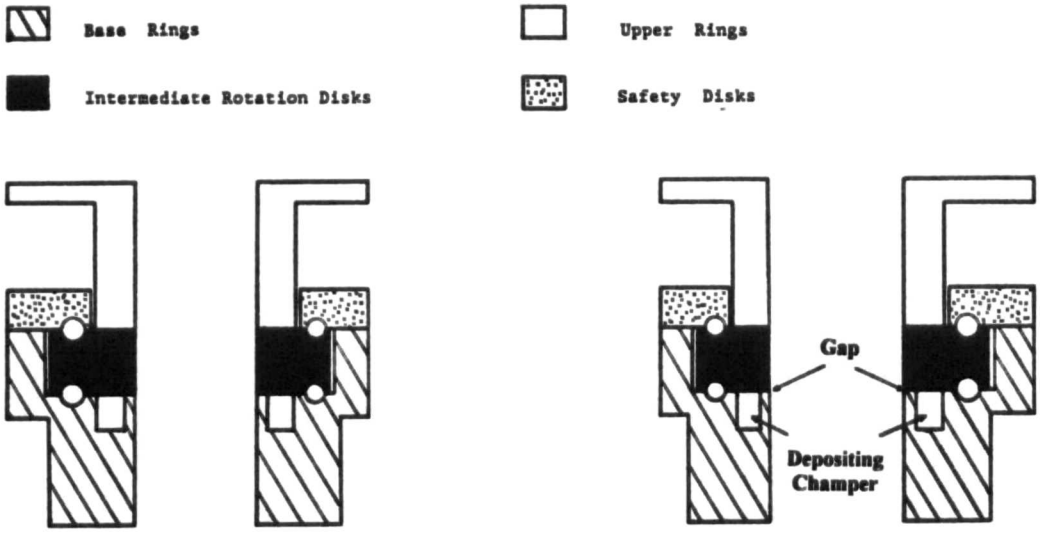


Fig. 4.4.7 Cross section of new confining rings (assembled)

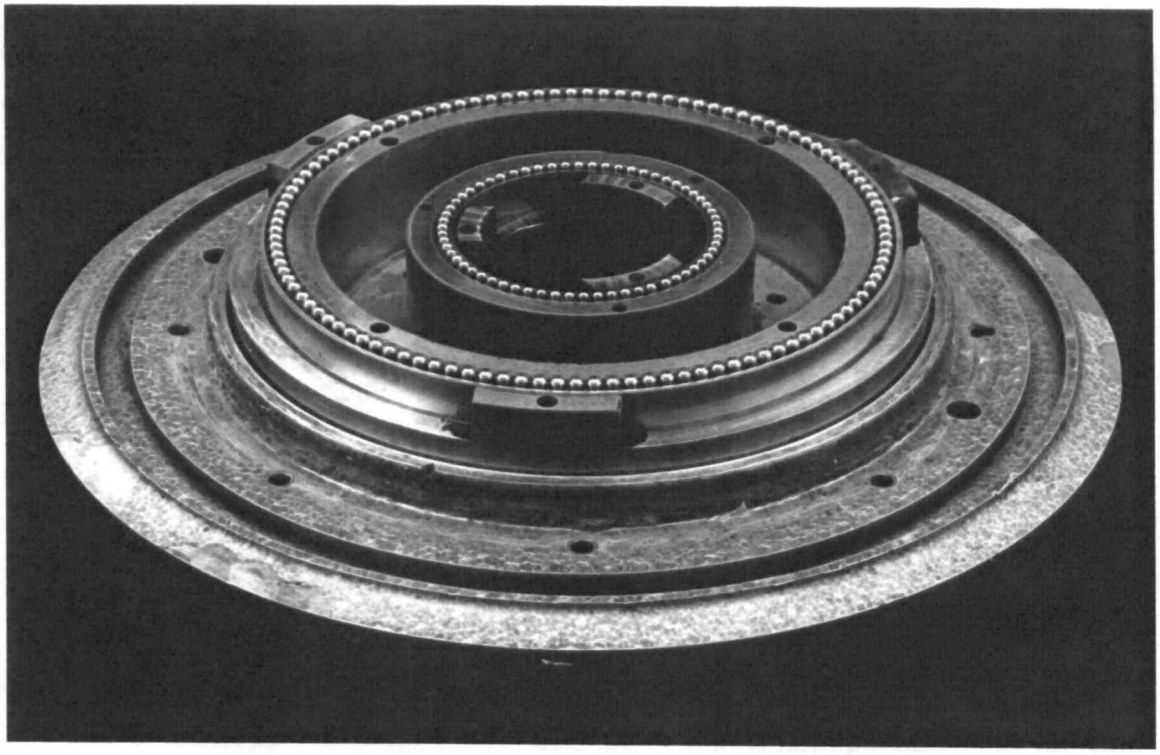


Fig. 4.4.8 Steel balls placed in upper lubricated grove of intermediate rotating disk

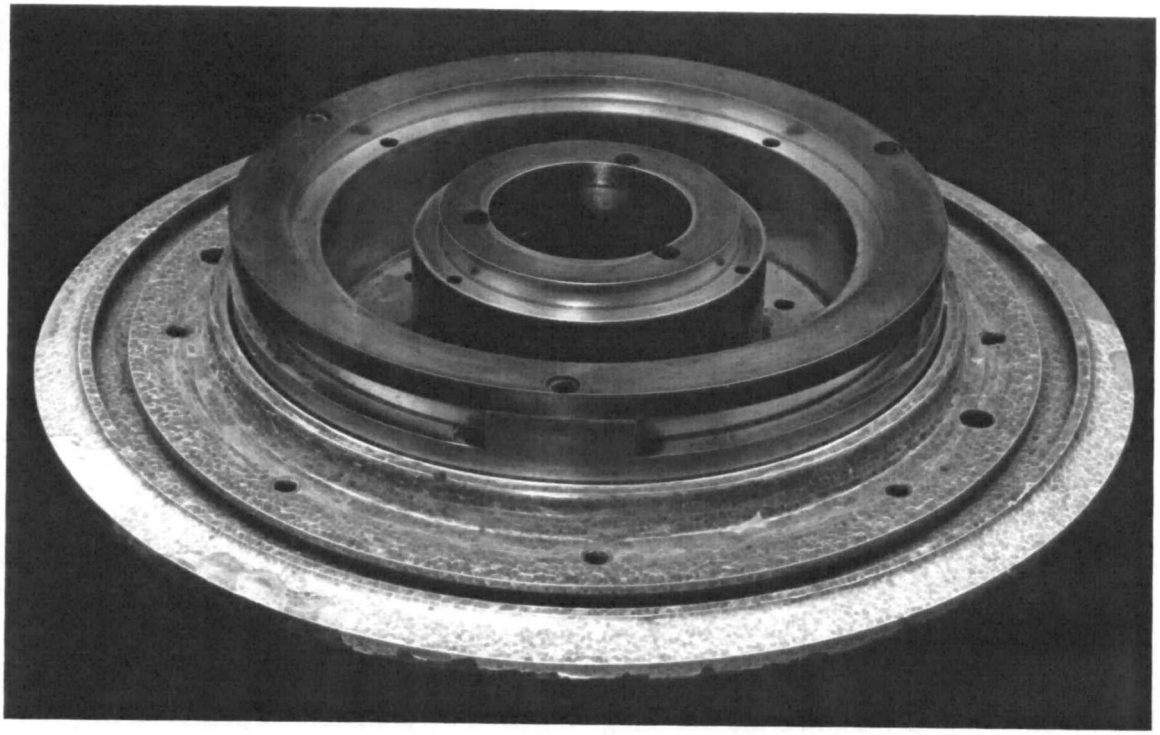


Fig. 4.4.9 Safety disk placed on top of upper set of steel balls and screwed onto base rings

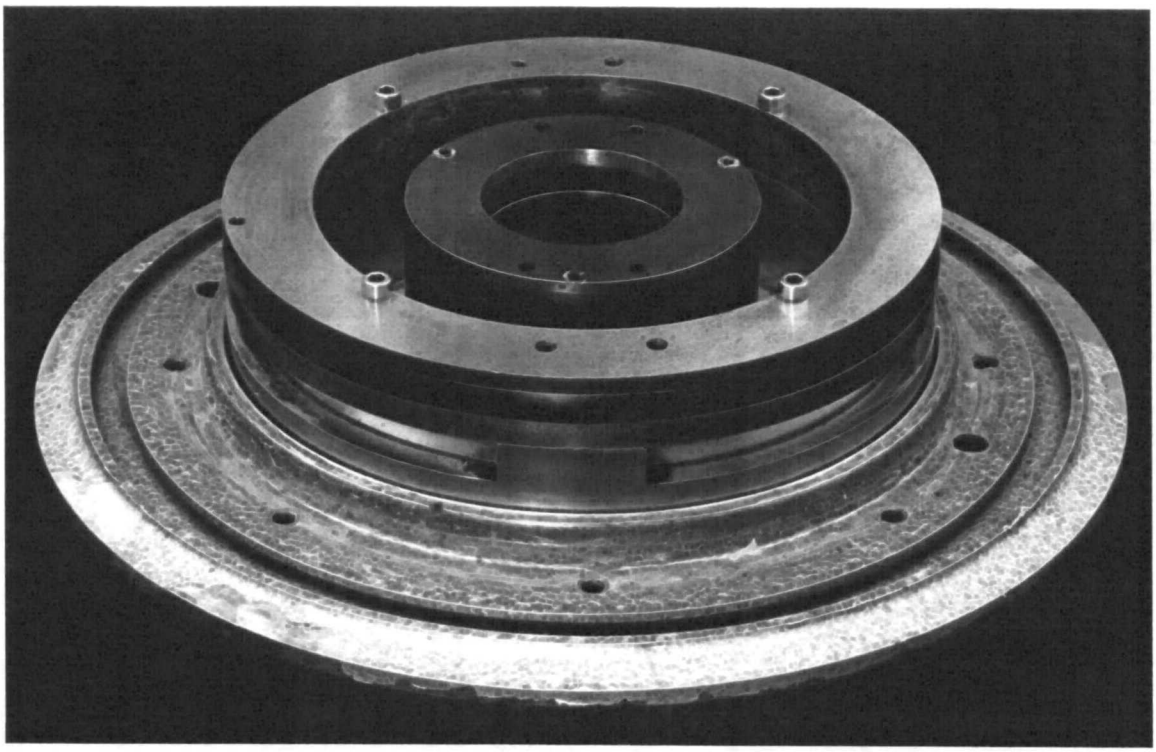


Fig. 4.4.10 Upper rings screwed onto intermediate rotating disk

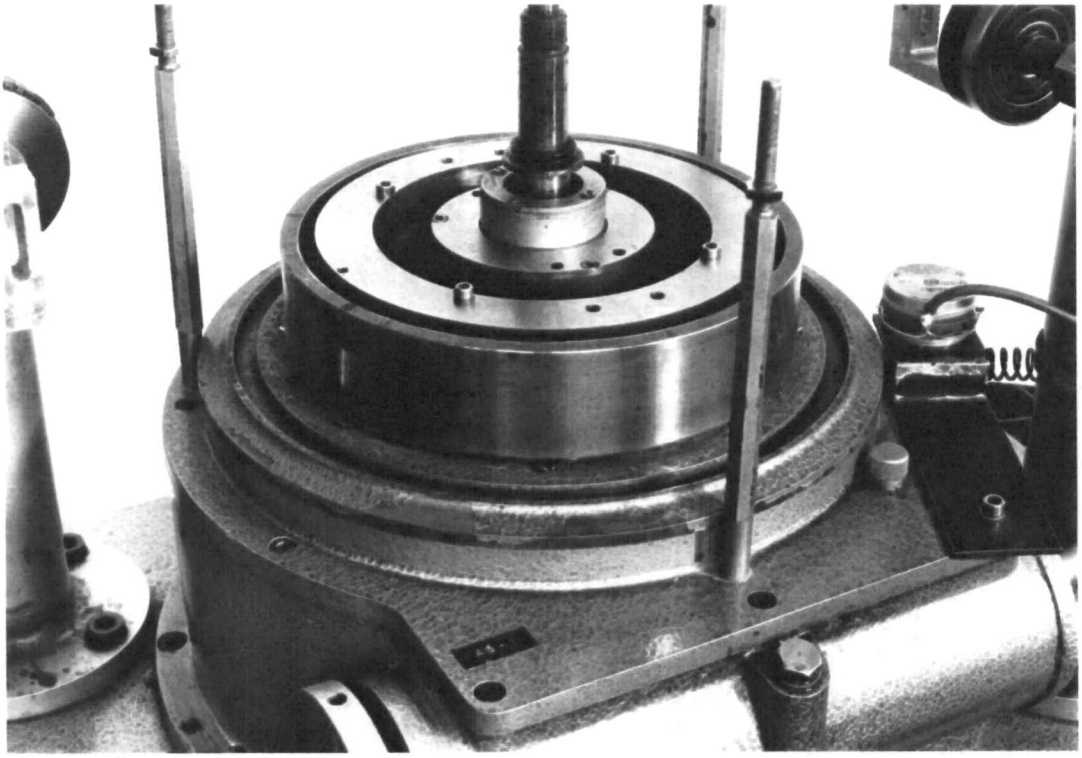


Fig. 4.4.11 Whole assembly placed onto ring shear apparatus

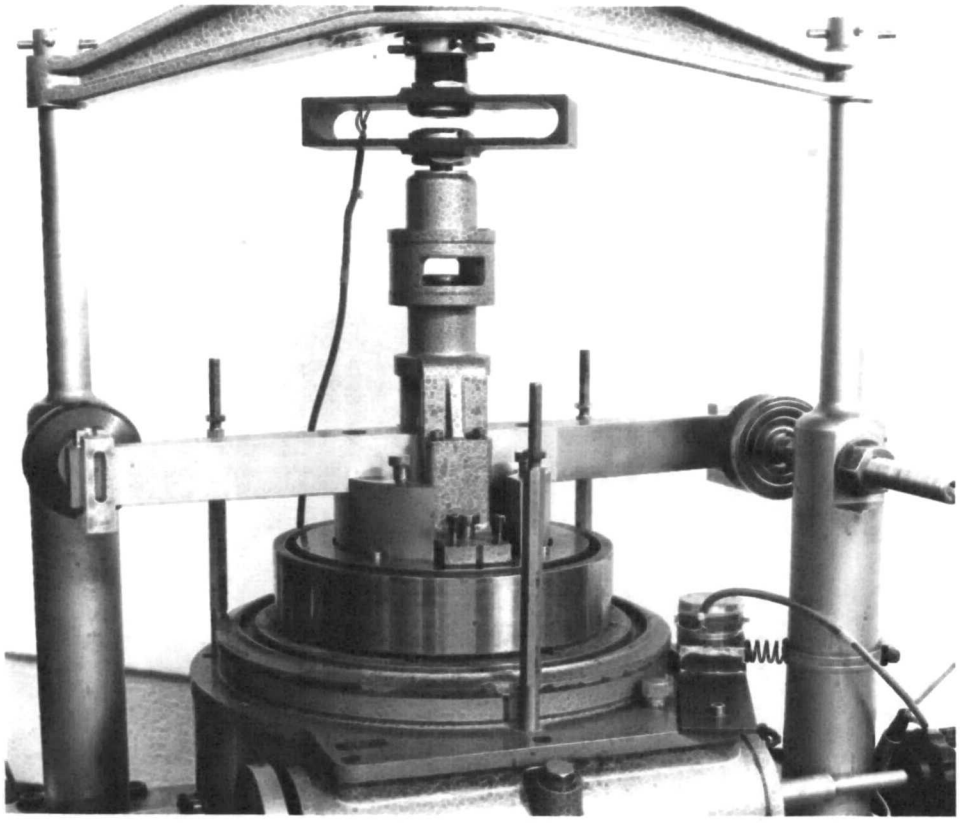


Fig. 4.4.12 Upper rings locked into position with existing frame

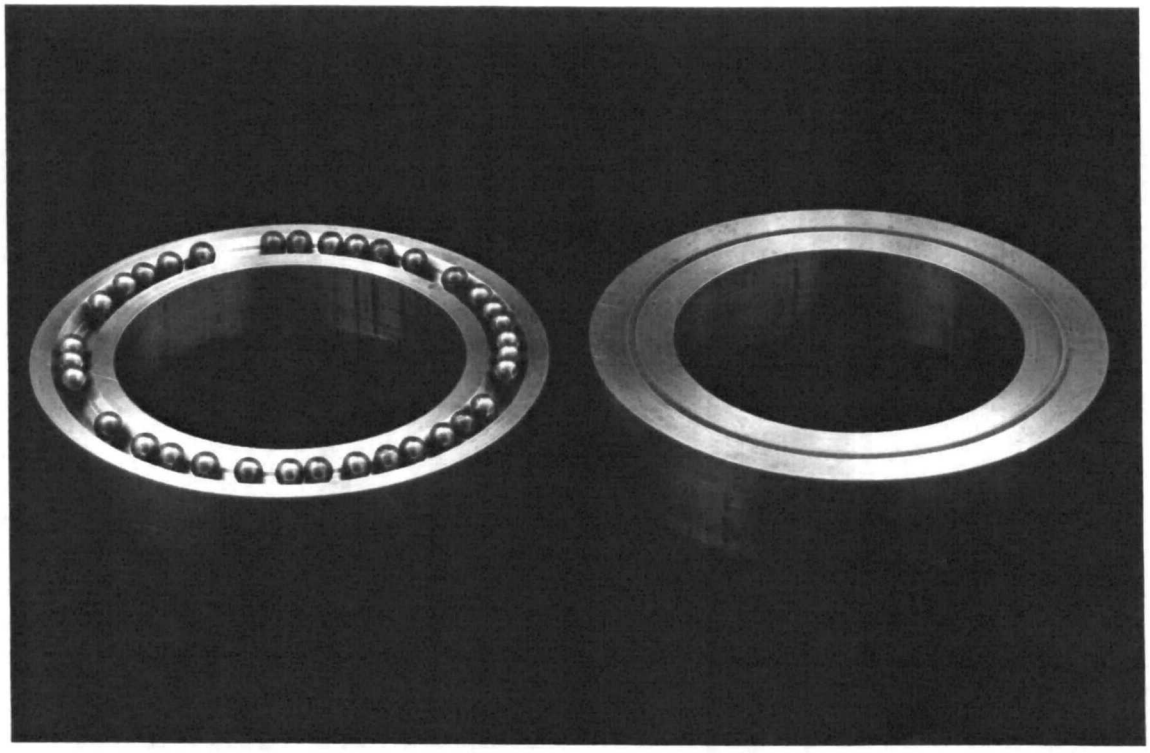


Fig. 4.4.13 Dummy sample

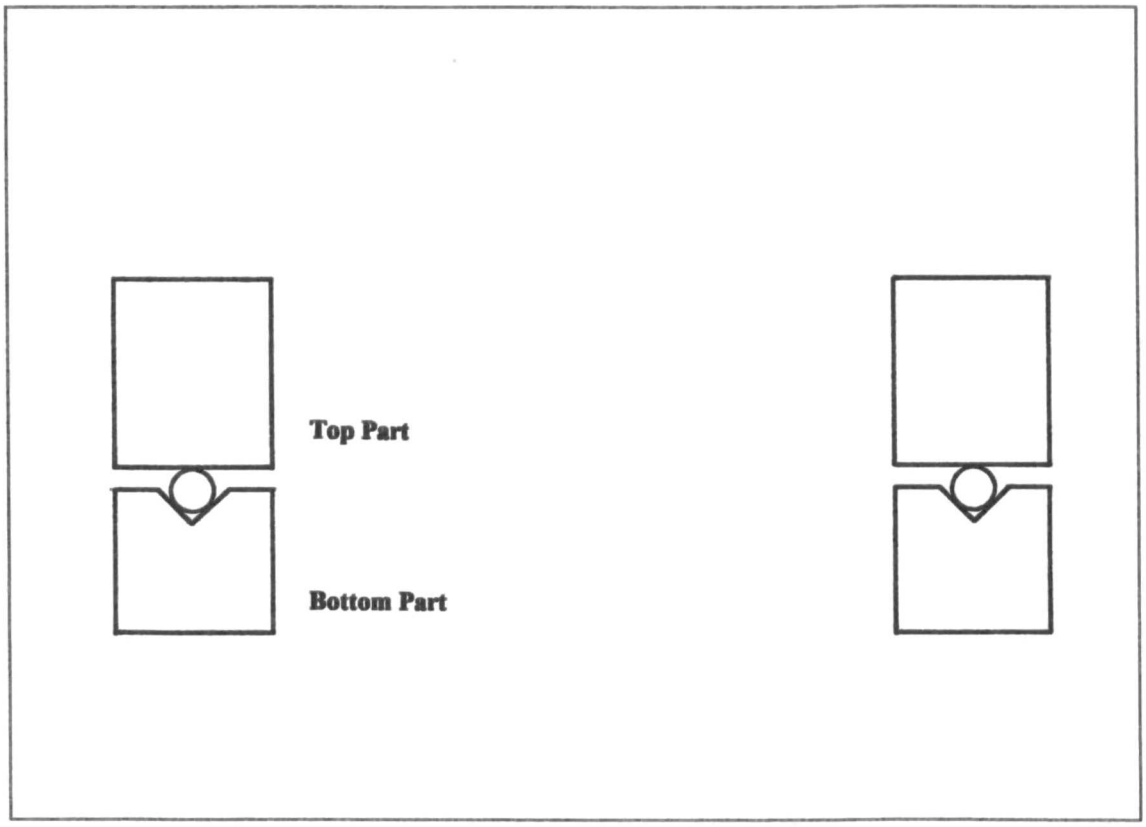


Fig. 4.4.14 Cross section of dummy sample

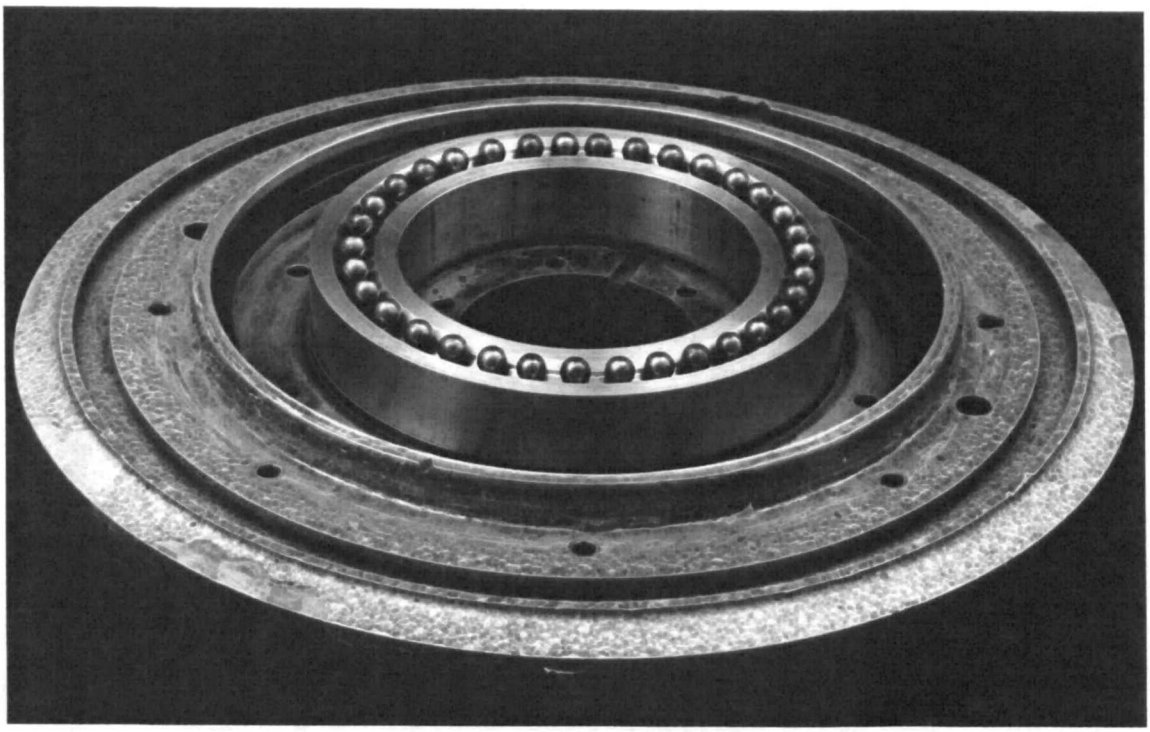


Fig. 4.4.15 Bottom part of dummy sample screwed onto rotating table

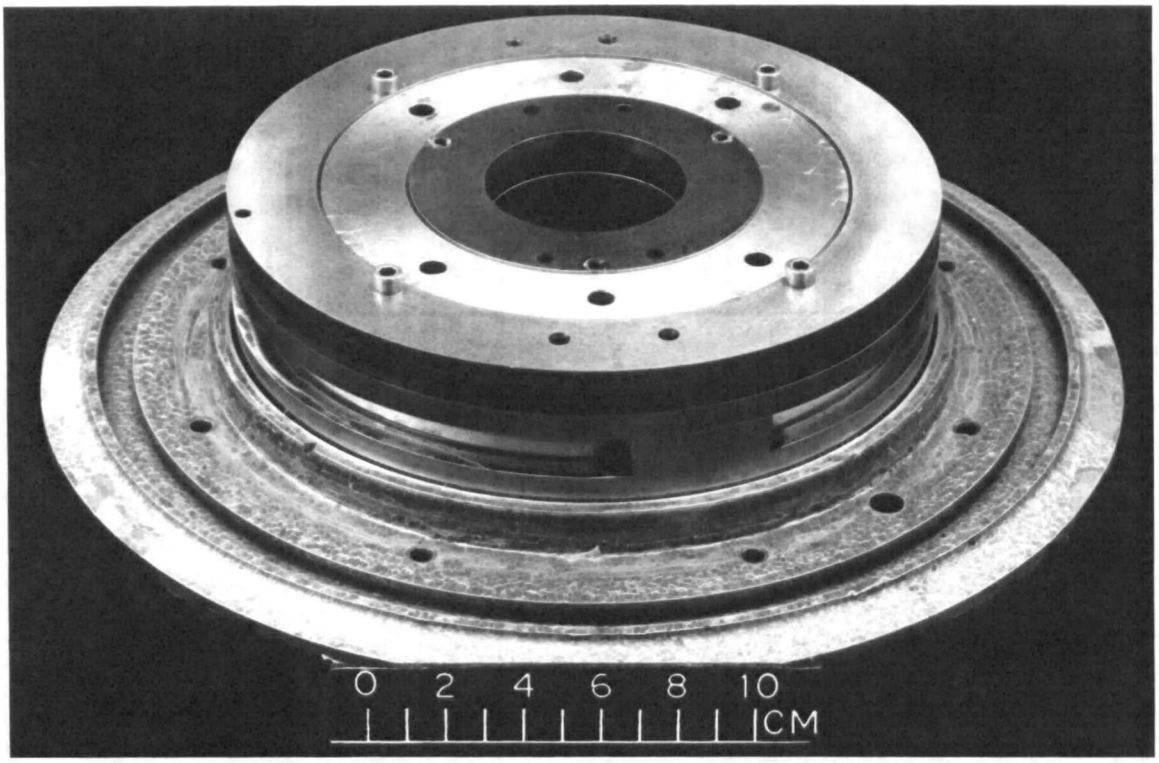


Fig. 4.4.16 Dummy sample placed inside new confining rings

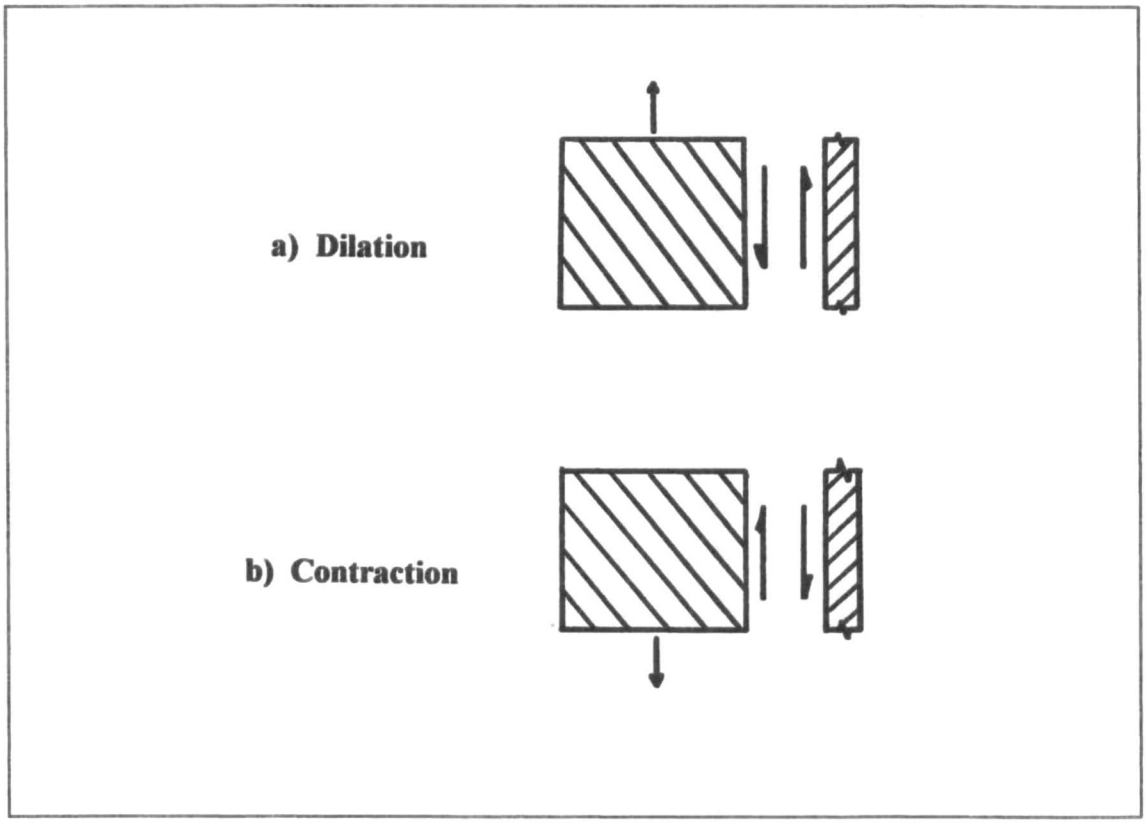


Fig. 4.4.17 Side friction mechanisms: a) dilation, b) contraction

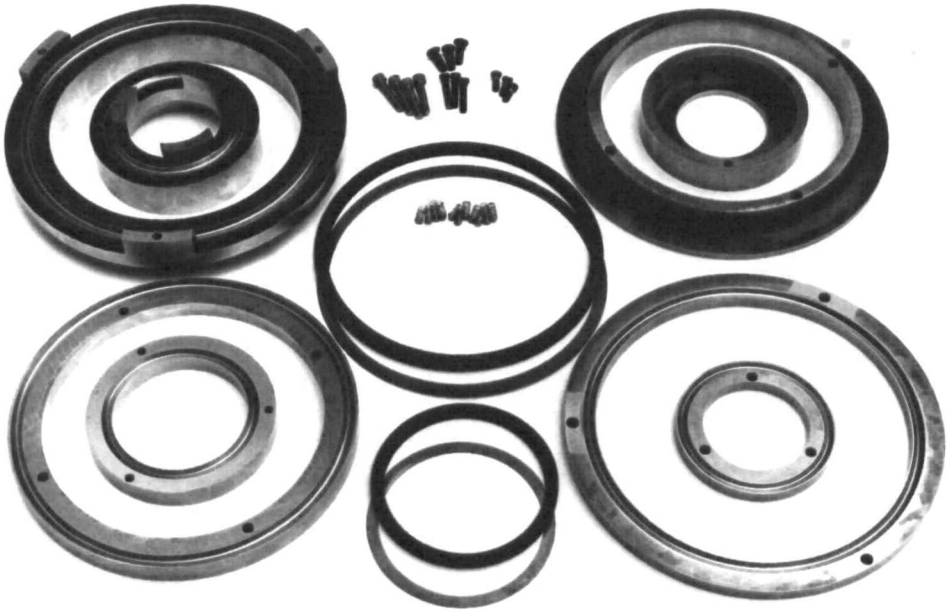


Fig. 4.4.18 Individual parts of new confining rings - Second version

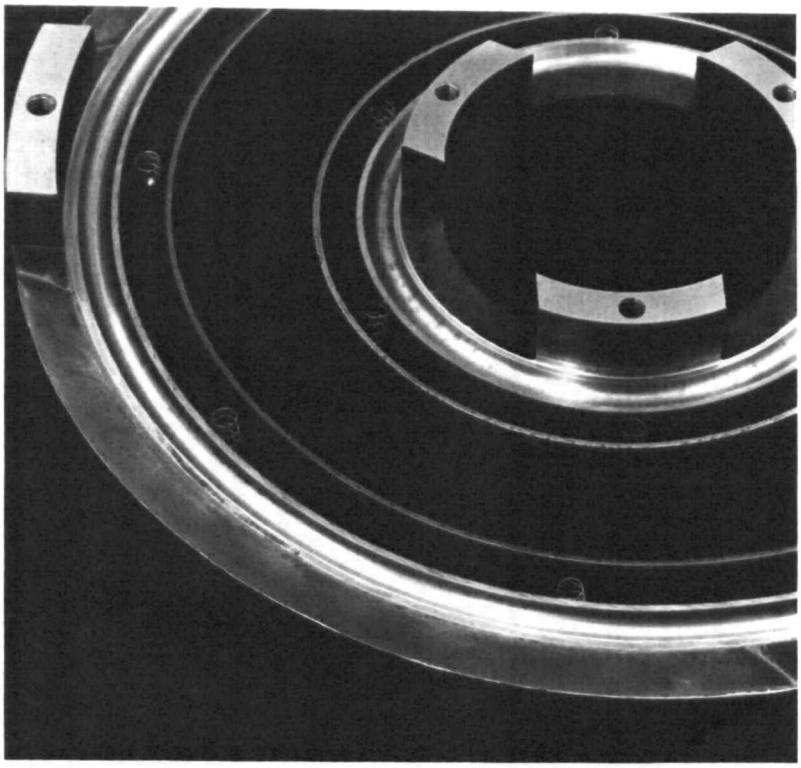


Fig. 4.4.19 Springs positioned in depositing chamber

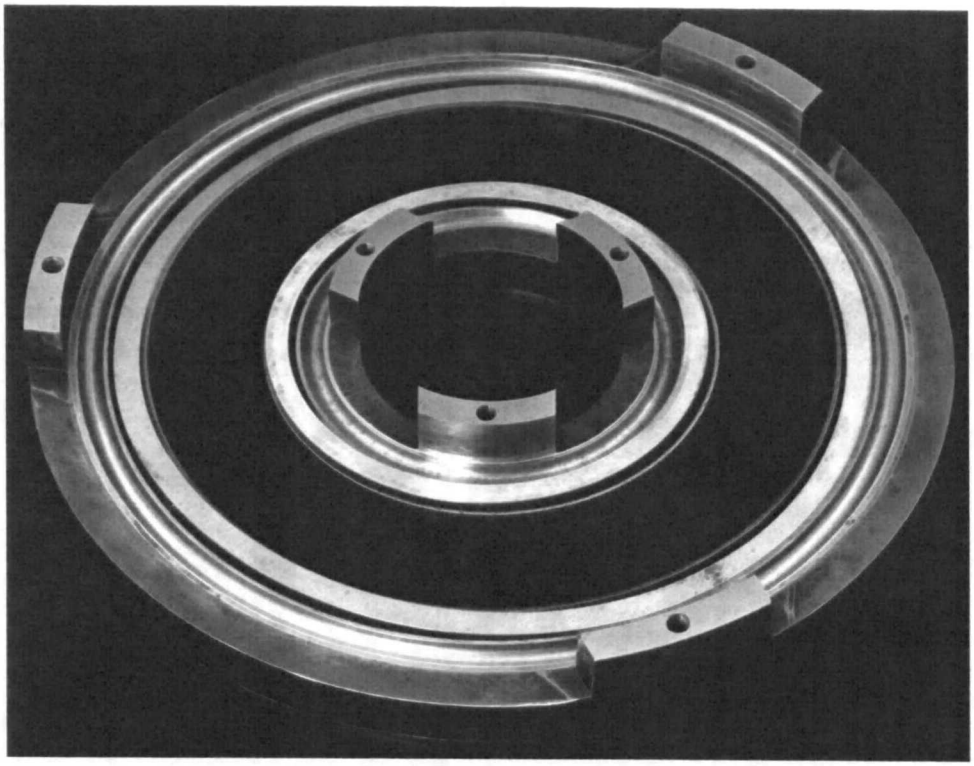


Fig. 4.4.20 Protective brass flange placed on set of springs

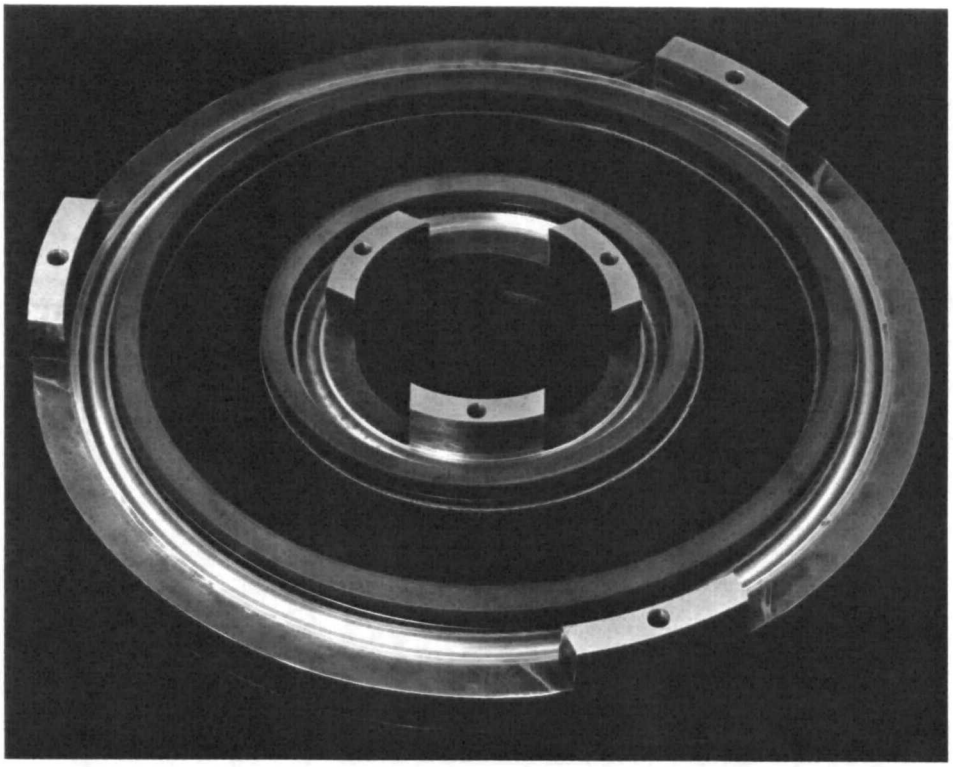


Fig. 4.4.21 Carbon flange placed on protective brass flange

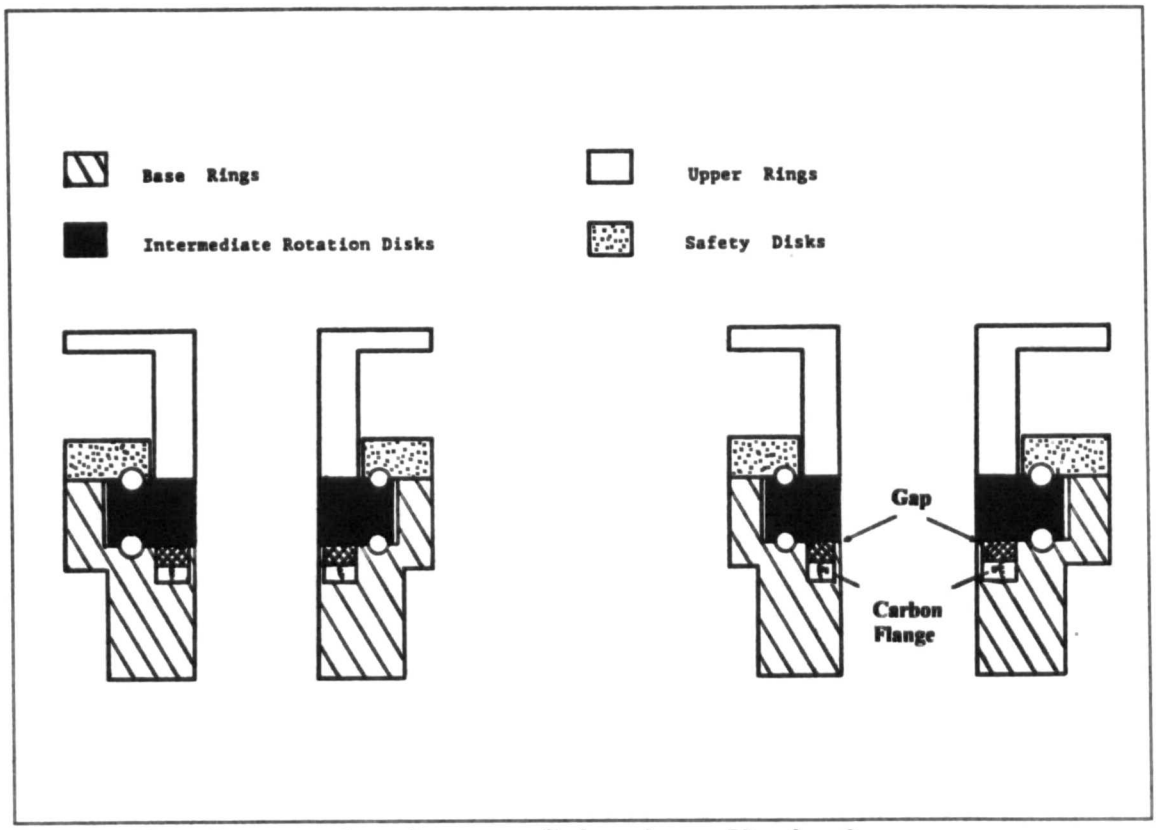


Fig. 4.4.22 Cross section of new confining rings - Version 2

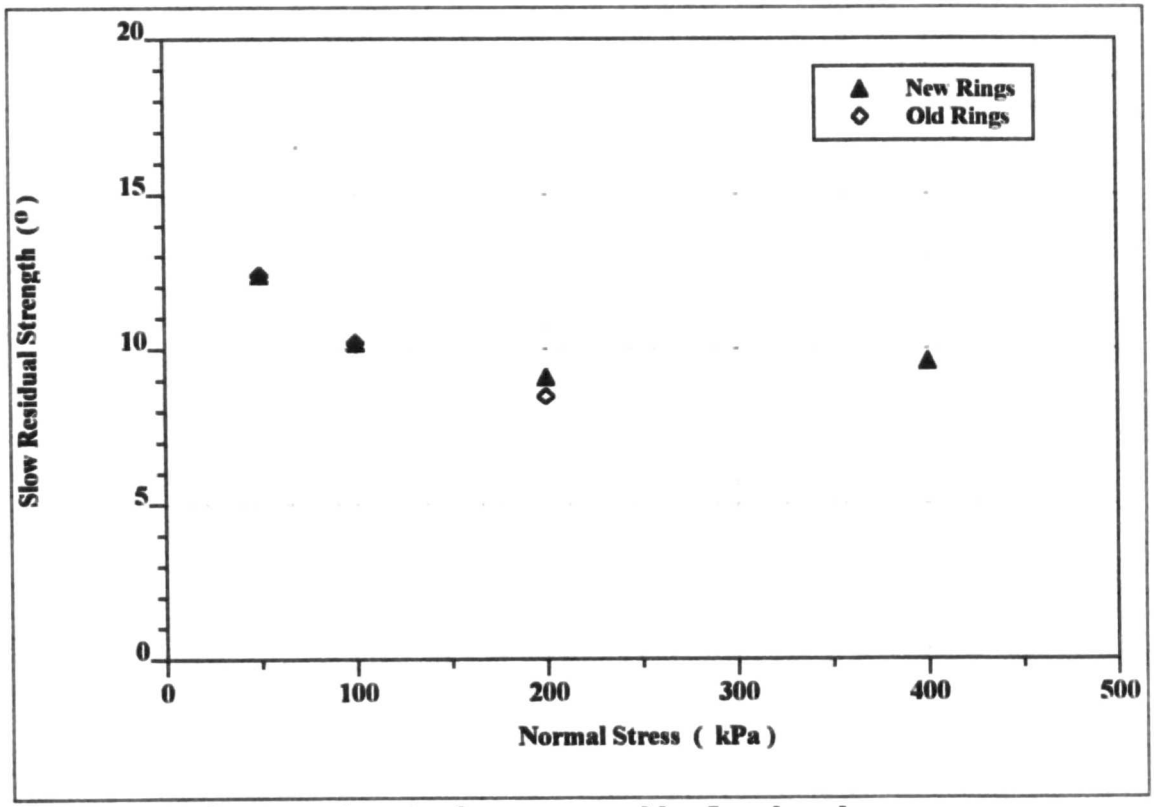


Fig. 4.5.1 Comparative plot from tests on blue London clay

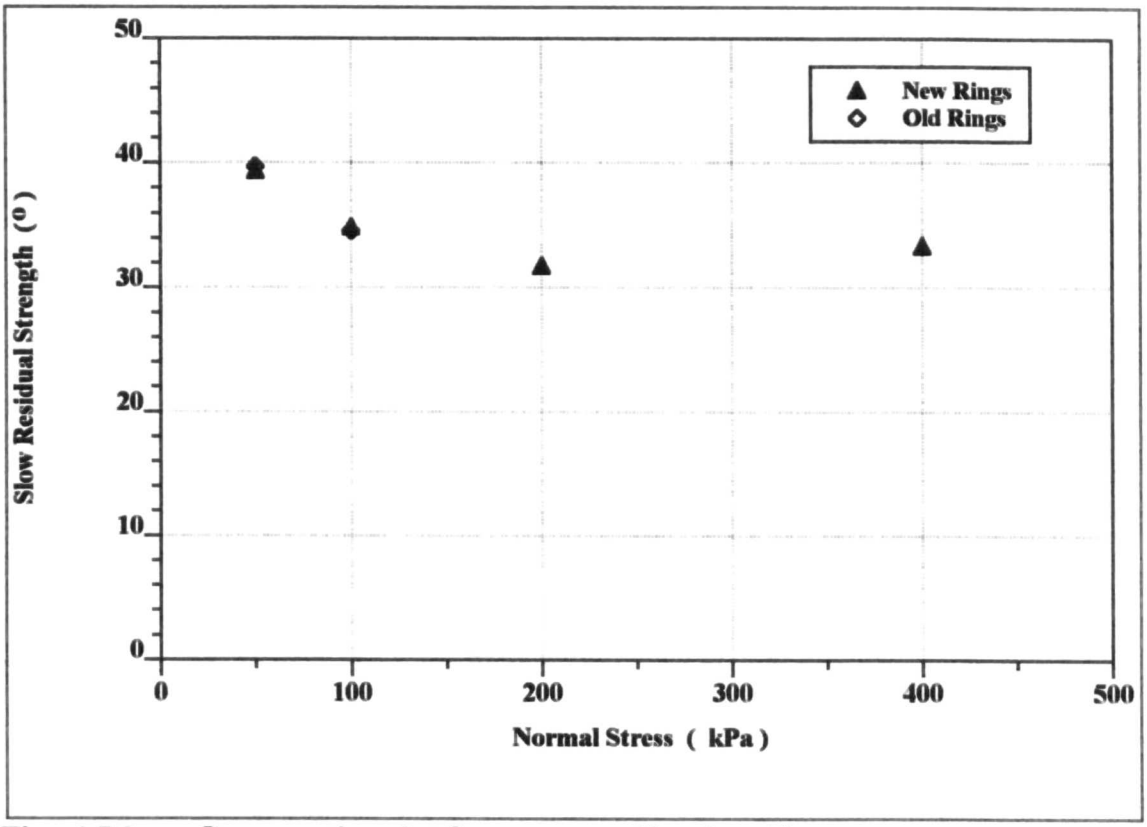


Fig. 4.5.2 Comparative plot from tests on Cowden till

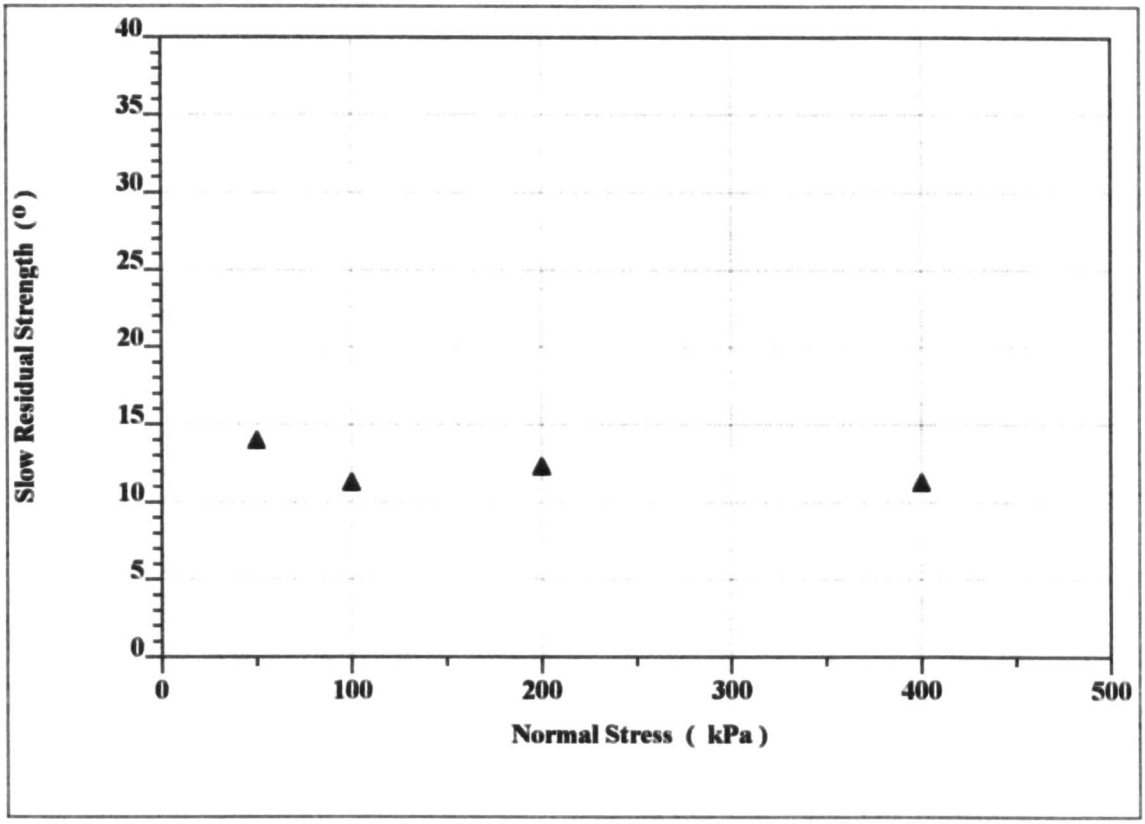


Fig. 4.5.3 Slow residual strength of brown London clay measured by means of new rings

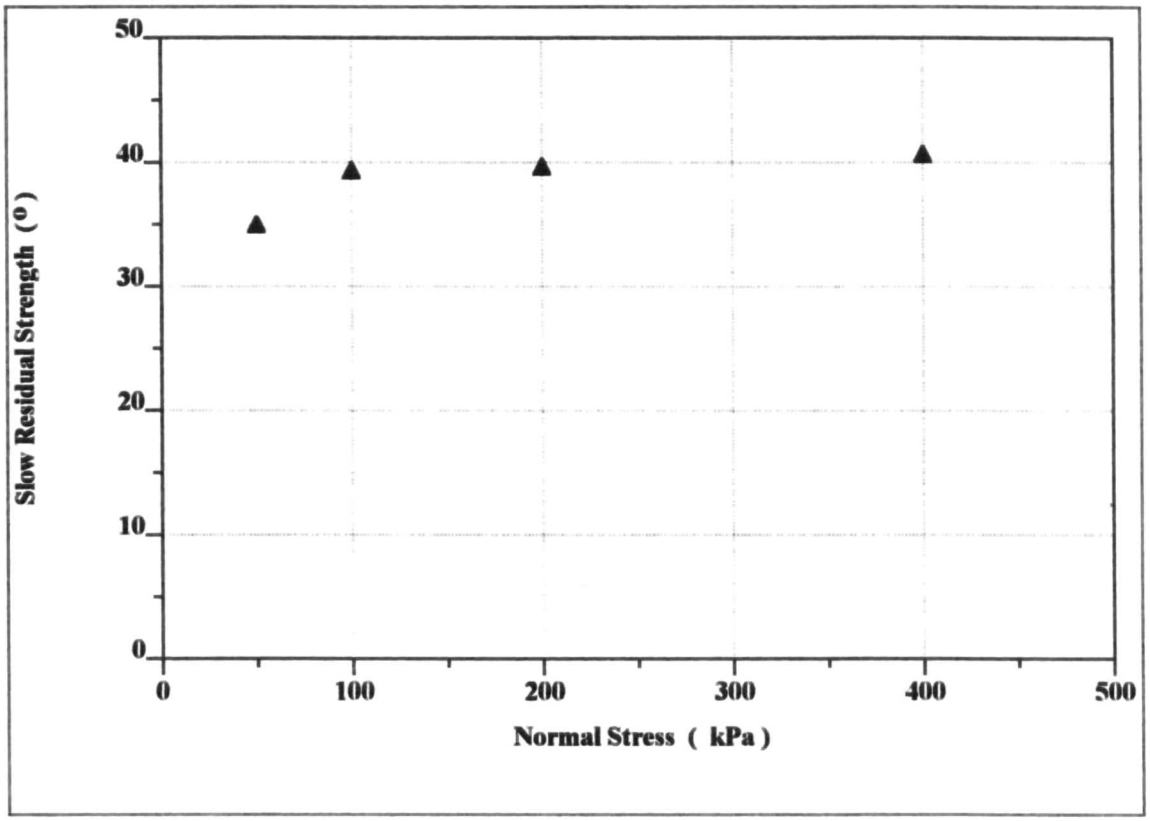


Fig. 4.5.4 Slow residual strength of medium-coarse sand measured by means of new rings

CHAPTER 5

DESCRIPTION OF TESTS

5.1 INTRODUCTION

It has been suggested that the behaviour of soils upon continually varying rates of shearing depends largely on their plasticity, their interparticle friction, and their grain size and shape. Therefore, six soil types were selected (natural and artificial) which would enable the investigation of rate effects within a broad range of plasticity indices, interparticle friction, grain sizes and shapes. The soil types used in this investigation are: brown London clay, Cowden till, pure silica (non-plastic), pure kaolin, artificially mixed pure silica and fine sand, and medium-coarse clean sand. Information regarding these soils is provided in Table 5.2.1, and in Appendices IV, V, and VI.

5.2 SAMPLE PREPARATION TECHNIQUE

The standard preparation procedure which has been described in detail in chapter 3 was followed for all the soil types which were tested in this investigation. The soil was air-dried, pulverised by mechanical means (this did not apply to the soil types which were commercially available), sieved through the 420 μm sieve, remoulded to a water contend around its plastic limit and left overnight to hydrate. After hydration the soil was reworked with a spatula and a sample for the determination of the water content before testing was taken [Table 5.2.1].

5.3 TESTING TECHNIQUE AND CONDITIONS

The multistage technique which has also been described in Chapter 3 was used in order to avoid testing samples of variable nature and also accelerate the testing process. Each sample was tested under three or four normal stress levels (50, 100, 200, and 300

or 400 kPa) depending on whether it was overconsolidated or normally consolidated. According to the standard testing technique which was employed in this investigation, the sample was loaded to the minimum of the normal stress levels that it was to be tested, consolidated, and sheared (drained) until it had reached its residual state at the constant rate of displacement of 0.09 mm/min. After the residual state had been established, the sample was unloaded in shear (*i.e.* the shear stress was reduced to zero) and then sheared fast (undrained) at continually varying rates of displacement which ranged between 10 mm/min and 6000 mm/min. Each fast shearing stage was terminated after a stable condition had been reached or after the shear displacement had approached 4 to 5 metres. Further information about the testing procedures associated with each sample may be found in Appendices IV, V, and VI, where each test is described in detail.

5.4 TEST RESULTS

The shear strength measured during each test is presented as the inverse tangent of the stress ratio calculated using equation (3.6). This may occasionally (low normal stresses, materials containing angular particles) result in high values of angle of friction but, nevertheless, it is considered to provide a comprehensible indication of the strength and behaviour of soils. Furthermore, it should be noted that although the slow shearing stage of each test is considered drained and the slow rate of displacement (0.09 mm/min) is thought to be sufficient to allow the dissipation of pore pressures, conditions during the fast shearing stage are complicated. It is speculated that some dissipation of pore pressures must take place throughout the duration of the fast shearing stage of each test but, since pore pressures cannot be measured with the existing apparatus, the conditions during that stage of each test will be considered undrained and total stresses will be used in the calculation of results. Finally, two units are used in relation to the rate of displacement because it varies substantially between the two (slow and fast) stages of a test and, also, because the results of this investigation are to be used in a seismology project currently in progress at Imperial College. Therefore, the slow rate of displacement will be measured in mm/min and the fast rates of displacement in cm/sec.

5.4.1 Brown London Clay

a) Normally Consolidated, Non-Submerged (Tests IV.1-5)

The sample was tested at four normal stress levels (50, 100, 200 and 400 kPa) under slow (drained) shearing conditions to establish the residual structure and fast (undrained) shearing conditions to study the rate effects. The slow shearing stages were associated with a decrease in sample thickness which was attributed mainly to particle orientation. The response of the soil during fast shearing was quite consistent considering the fact that it did not involve a decrease in shear strength below its slow residual level. Furthermore, except in the case of the first fast test at 50 kPa, the shear strength increased as the rate of displacement increased and vice versa.

The first sample was consolidated at 50 kPa and the measured strength had reached its slow residual value of 15.6° [Fig. IV.1.1], when the slow shearing stage was terminated after 40.5 cm of shear displacement. The rate of displacement was then increased to 0.72 cm/sec [Fig. IV.1.2], the measured strength reached a threshold value of 25.8°, and the sample height decreased by 0.12 mm. As shearing continued at the same rate of displacement the sample height increased by 0.82 mm and the measured strength dropped to 9.7°. An undulating shear surface appeared to be forming with an amplitude which decreased from 0.62 mm, at the beginning of this fast stage, to 0.13 mm at the end of the stage, while the measured strength varied between 15.1° and 24.0° independently of the rate of displacement. The sharp shape of the oscillations of the measured strength ought to be attributed partly to internal ring friction which could not be accounted for at that point. It should be noted that the behaviour of the apparatus at low normal stress levels has been criticised by previous researchers (Lemos, 1986, and Tika, 1989) and it should be considered as one of the handicaps of the system.

The same sample was then consolidated at 100 kPa and sheared slowly to its slow residual strength of 11.7° [Fig. IV.2.1], when the rate of displacement was increased to 0.12 cm/sec. The measured strength increased to 14.0° [Fig. IV.2.2] and, when the rate of displacement was increased further to 1.40 cm/sec, the strength stabilised at 22.5°.

The normal load was increased and the sample was consolidated at 200 kPa and reached its slow residual strength of 11.3° [Fig. IV.3.1] before it was sheared, initially, at

0.05 cm/sec. The measured sample strength increased, only, to 11.5° [Fig. IV.3.2] but when the rate of displacement was increased, suddenly, further to 1.29 cm/sec, the strength reached a threshold value of 20.8°. Further shearing at the same rate of displacement led to a sample dilation of 0.42 mm and a drop in strength to 11.1°. Subsequently, the sample appeared to be contracting while the measured strength stabilised between 16.5° and 17.5°. The shape of the shear surface appeared to became planar as the test progressed.

The same sample was consolidated to 400 kPa and reached its slow residual strength of 12.9° [Fig. IV.4.1] before the rate of displacement was increased to 0.037 cm/sec. The measured strength increased to 13.9° [Fig. IV.4.2] but when the rate of displacement was increased further to 1.38 cm/sec, the strength reached a new threshold value of 21.3°, the sample dilated by 0.53 mm and the strength dropped to 12.2°. Further shearing at the same rate of displacement was associated with a decrease in sample height while the measured strength stabilised between 14.8° and 16.2°. The shear surface appeared to be undulating with an amplitude of 0.17 mm and the measured strength dropped below its slow residual level when the rate of displacement was decreased to 0.15 cm/sec.

All these tests indicated that the measured sample strength increases with increasing rate of displacement, the threshold strength is not so pronounced when the increase of the rate of displacement is moderate, and the immediate drop in strength after it had reached its threshold value apparently related to the dilation of the sample. The measured sample strength on fast shearing dropped below its slow residual value in one case probably because of the dilation of the sample and the undulating nature of the shear surface.

After the test was terminated, the sample was extruded from the confining rings and submerged in Carbowax for the preparation of thin sections. The visual inspection of the thin sections indicated the existence of voids around the mid-height of the sample and a non-planar shear zone which, at some points, extended to the top of the sample [Fig. 5.TH1].

A second sample was prepared, identical with the first, consolidated to 50 kPa and sheared using the previously described procedure. After it had reached its slow residual strength of 12.3°, the sample appeared to be gaining strength [Fig. IV.5.1].

However, this was thought to be either due to internal ring friction or the early development of an undulating shear surface. During the fast shearing stage of this test [Fig. IV.5.2], the sample strength increased, however, the results were obscured by the periodic increased in measured shear stress which was apparently caused by internal ring friction. Apart from the fact that the increase in the rate of displacement had a positive effect on the measured sample strength, no more conclusions could be drawn from this test. Furthermore, the shape of the sample height curve indicated the existence of an undulating shear surface.

After the test was terminated, the sample was carefully removed from the confining rings, inspected visually, one part was submerged in Carbowax and the rest was left to air-dry. The position and thickness of the shear zone of the part of the sample which was not submerged in Carbowax were measured with a digital micrometer. The visual inspection revealed the existence of a distinct shear zone which was separated from the top and bottom parts of the sample by two surfaces, was of a darker colour than the rest of the sample, and had a nearly uniform thickness of 1.5-2.0 mm [Fig. 5.4.1.1.a]. However, in two diametrically opposite points along the periphery of the sample, the lower boundary of the shear zone dipped towards the bottom and the width of the shear zone increased to 6.3 mm [Fig 5.4.1.1.b]. Apart from the aforementioned irregularities which were observed in the 16 % of the periphery of the sample and were associated with the position of the lower boundary of the shear zone, no secondary shear planes or other distinct features were noted [Fig. 5.4.1.2]. The thin sections which were prepared from this sample indicate an inclining shear surface, secondary shear planes and fissures which propagate from the fins of the porous stones and often develop into shear planes. Finally, the existence of a gap of non-uniform width which separates the sample is indicated [Fig. 5.TH2-3].

b) Normally Consolidated, Submerged (Tests IV.6-9)

The samples were prepared and tested in the same procedures with the non-submerged samples. The slow residual values of the submerged samples were lower than those of the non-submerged ones apparently because of the lubricating action of water. Drained shearing was associated with a decrease in sample thickness mainly due to

particle orientation, as the samples approached their residual state, and to a lesser degree to some sample loss.

The first sample was consolidated to 50 kPa and reached its slow residual strength of 14.5° [Fig. IV.6.1] before the rate of displacement was increased to 0.63 cm/sec. The sample appeared to contract by 0.067 mm [Fig. IV.6.2] and reached its threshold strength of 29.8° before it dilated to 0.15 mm and the measured strength dropped to 13.1°. Further shearing at the same rate of displacement led to a decrease in sample height and an increase in measured strength which stabilised around 21.4°. The rate of displacement was increased further to 1.68 cm/sec and the measured strength stabilised around 25.5°. Before this fast stage was terminated, the rate of displacement was varied between 0.2 cm/sec and 1.68 cm/sec. The measured sample strength decreased with decreasing rates of displacement, without dropping below its slow residual value, and appeared to be increasing slightly with cumulative shear displacement. After 150 cm of shearing the measured sample strength which corresponded to 1.68 cm/sec was 25.5°, after 300 cm it was 29.4°, and after 480 cm it was 30.8°. However, this may be an effect of the decreasing sample height due to soil which is being lost through the gap of the rings.

The same sample was consolidated at 400 kPa and reached its slow residual strength of 11.3° [Fig. IV.7.1] before the rate of displacement was increased to 1.31 cm/sec. The measured sample strength reached its threshold value of 20.3° [Fig. IV.7.2] and immediately dropped to 14.0° as the sample dilated by 0.05 mm. Further shearing at the same rate of displacement led to a contraction of the sample and an increase in sample strength which stabilised around 16.4°. The measured sample strength decreased with decreasing rate of displacement, at some points even slightly below its slow residual level. Furthermore, it appeared that any increase of the rate of displacement above 1.0 cm/sec had no effect on the measured sample strength. The thin section which was prepared from this sample showed evidence of the existence of a rather uniform width shear zone which was separated from the top and bottom of the sample by two distinct shear planes [Fig 5.TH4].

A second sample was prepared identical to the first, consolidated at 100 kPa and reached its slow residual strength of 11.3° [Fig. IV.8.1] before the rate of displacement was increased to 0.35 cm/sec. The sample strength reached its threshold value of 19.8°

[Fig. IV.8.2] and stabilised at 18.5° before the rate of displacement was increased further to 1.71 cm/sec. The sample contracted by 0.05 mm and the measured strength reached a new threshold level at 22.8°. Further shearing at the same rate of displacement was associated with a dilation of the sample by 0.21 mm and a decrease in measured strength to 13.2°. An undulating shear zone with a wave-length of 39.4 cm (perimeter of the sample) and an initial amplitude of 0.3 mm was generated. Apparently the amplitude of the oscillations was sufficient to increase the pore pressures within the shear zone and, therefore, decrease the sample strength which dropped below its slow residual level after 116 cm of shearing. The sample strength had stabilised at 5.9° when the rate of displacement was decreased to 0.12 cm/sec. At this lower rate of displacement the measured strength increased to 7.6°, apparently due to dissipation of the pore pressures within the shear zone, and, when the rate was increased again to 1.71 cm/sec, the strength initially increased to 10.0° before it stabilised around 8.1° where it remained until the test was terminated. When the rate of displacement was increased for the second time, the undulating shape of the shear zone appeared to became planar and no additional pore pressures were generated which would have resulted in further decrease of the measured strength.

The same sample was consolidated to 200 kPa and reached its slow residual strength of 12.0° [Fig. IV.9.1] before the rate of displacement was increased to 0.11 cm/sec. The measured strength reached a moderate threshold value of 16.1° [Fig. IV.9.2] and subsequently decreased to 14.0° while the sample appeared to dilate by 0.03 mm. The rate of displacement was increased further to 1.70 cm/sec, the shear zone did not appear to contract or dilate, and the measured strength increased and stabilised around 22.0°. Further variations of the rate of displacement indicated that the strength increases with increasing rate of displacement and vice versa, it does not drop below its slow residual value, and it is not affected by any increase of the rate above 0.7 cm/sec.

After the test was terminated, the sample was extruded from the confining rings, visually inspected and measured. The texture of the sample appeared firm and uniform, the shear zone was of a darker colour than the rest of the sample, and there was no gap separating the shear zone from the top or bottom parts of the sample. The upper and lower boundaries of the shear zone were planar and nearly horizontal. However, at one point along the periphery of the sample, a secondary shear plane extended from the

upper boundary of the shear zone towards the top of the sample [Fig. 5.4.1.3-4]. No other distinct features or secondary shear planes were observed.

It is suggested by this set of tests that the strength increases with increasing rates of displacement and, only when an undulating shear zone is formed, it decreases below its slow residual value.

c) Overconsolidated, Submerged (Test IV.10-12 and VI.1-2)

This sample was prepared identical to the normally consolidated samples, consolidated to 400 kPa and unloaded to 100 kPa. Shearing to residual at 100 kPa was associated with some sample dilation, as the shearing matrix relaxed, and the measured strength reached its slow residual value of 15.5° [Fig. IV.10.1]. The rate of displacement was increased to 1.76 cm/sec and the measured strength reached its threshold value of 36.9° [Fig. IV.10.2]. Further shearing at the same rate of displacement was associated with an increase in sample height (which may be associated with a dilation of the soil matrix within the shear zone) of 0.88 mm and a decrease in strength to 12.6° before it stabilised around 15.5°. An undulating shear zone appeared to be forming, with initial an amplitude of 0.6 mm, which progressively decreased and stabilised at 0.18 mm. However, this undulating shape of the shear surface was thought to be sufficient to induce pore pressures and decrease the measured strength below its slow residual level. The strength oscillated between 1.7° and 9.7°, but changes of the rate of displacement between 1.76 cm/sec and 5.94 cm/sec appeared to have no significant effect on the strength.

The same sample was consolidated to 200 kPa and reached its slow residual strength of 11.4° [Fig. IV.11.1] before the rate of displacement was increased to 3.2 cm/sec. The measured strength reached its threshold value of 24.7° [Fig. IV.11.2] and subsequently decreased to 5.1° as the sample dilated by 0.85 mm. An undulating shear surface appeared to be forming with an initial amplitude of 0.29 mm which decreased with further shearing to 0.08 mm and had become planar towards the end of the test. The amplitude of the oscillations and the magnitude of the rate of displacement were thought to be sufficient to induce pore pressures within the shearing matrix and decrease the measured strength which varied between 1.8° and 4.8°. After the first 230 cm of shearing, while the amplitude of the oscillations had decreased substantially, the strength appeared

to be increasing most probably due to dissipation of the pore pressures. The variations of the rate of displacement between 1.76 cm/sec and 9.41 cm/sec appeared to have no significant effect on the measured strength which had increased around 11.5° before the test was terminated.

The sample was consolidated further to 300 kPa and reached its slow residual strength of 12.3° [Fig. IV.12.1] before the rate of displacement was increased to 6.0 cm/sec. The measured strength reached its threshold value of 24.6° [Fig. IV.12.2] and, while the rate of displacement continued to increase, the former reached the value of 26.6°. Subsequently, the sample dilated by 0.48 mm and the strength dropped to 11.3°. The shear stress curve indicates that an undulating shear zone had formed, although this is not apparent from the sample height curve, probably because of the scaling of the latter curve. The strength appeared to be oscillating between 4.2° and 14.0°, however, it is thought that some part of the measured strength should be attributed to internal ring friction which cannot be accounted for. Nevertheless, the magnitude of the initial increase in sample height appears to be sufficient to result in a substantial decrease of the measured strength. Furthermore, any variation of the rate of displacement between 2.0 cm/sec and 10 cm/sec had no effect on the measured strength.

After this test was terminated, the sample was extruded from the confining rings, visually inspected, measured, one part was submerged in Carbowax, and the rest was left to air-dry. The shear zone appeared as a thin band of soil with soft texture as opposed to the firm texture of the top and bottom parts of the sample. Furthermore, the colour of the shear zone was lighter than the rest of the sample [Fig. 5.4.1.5.a-b] and, at some points along the periphery, a gap had developed between the shear zone and the top part of the sample [Fig. 5.4.1.5.c]. Although the width of the shear zone, when it was measured on the outside end of the sample, appeared to be nearly constant, around 1.0 mm, its shape was undulating [Fig. 5.4.1.6]. Observation of the thin section indicated that the width of the shear zone was increased in the mid-width of the sample and at some points it might have extended to the top or bottom ends of the sample [Fig. 5.TH5].

Evidence suggest that the decrease in shear strength below its slow residual value may be attributed to increased pore pressures within the shear zone due to a "pumping action" induced by its undulating shape. When the undulating shear surface became

planar (at 200 kPa) the "pumping action" ceased and the shear strength increased as pore pressures were thought to dissipate. Furthermore, no response in any change of the rate of displacement was observed in the cases where the shear strength had dropped below its slow residual value. In order to investigate further the response of the overconsolidated brown London clay and to test the assumption that the drop in strength is caused by increased pore pressures, another sample was prepared identical to the one which was previously described. Furthermore, the testing procedure was altered and the sample was sheared at 0.09 mm/min after the end of the fast stage so that, if pore pressures had indeed increased during fast shearing, they would dissipate and the measured strength would gradually return to its slow residual level.

The sample was consolidated to 400 kPa, unloaded to 100 kPa, and reached its slow residual strength of 10.9° [Fig. VI.1.1] before the rate of displacement was increased to 1.5 cm/sec. The measured strength reached its threshold value of 27.0° [Fig. VI.2.2] and subsequently decreased to 11.2° while the sample height increased by 0.2 mm. Further shearing at the same rate of displacement was associated with a decrease in sample height and an increase of the measured strength around 17.0°. A non-undulating shear surface appeared to have formed and the measured strength decreased with decreasing rates of displacement but did not drop below its slow residual value. When the rate of displacement was increased for the second time to 1.5 cm/sec, the measured strength varied between 15.3° and 24.2°, apparently because of non-uniformities of the shear zone, while further increase in the rate above 1.72 cm/sec had no effect on the sample strength which stabilised around 22.3° until this fast stage was terminated. The rate of displacement was subsequently decreased to 0.09 mm/min and the measured strength gradually decreased from an initial value of 16.7° to its slow residual level of 10.9° [Fig. VI.1.3].

The same sample was consolidated to 200 kPa and reached its slow residual strength of 11.0° [Fig. VI.2.1] before the rate of displacement was increased to 1.25 cm/sec. The measured strength increased to 21.5° [Fig. VI.2.2] and stabilised within a range between 18.0° and 20.0°, while shearing at the same rate of displacement continued. The strength decreased with decreasing rate of displacement but did not drop below its slow residual level while a planar shear surface appeared to have been formed. Furthermore, when the rate of displacement was increased to 1.75 cm/sec, the strength

increased to 20.9° and continued to increase moderately as the rate reached the level of 8.0 cm/sec. This increase in measured strength was not proportional to the corresponding increase in the rate of displacement but it was, nevertheless, apparent and contradicted previous findings which suggested that the strength is not affected when the rate increases above a certain level. Finally, when the rate of displacement was decreased to 0.09 mm/min, during the third stage of this test [Fig. VI.2.3], the measured strength decreased gradually from an initial level of 15.6° to its slow residual value of 11.0°.

The visual inspection of the sample after it had been taken out of the confining rings revealed the existence of a nearly uniform shear zone with darker colour than the rest of the sample and a width of around 1.0 mm [Fig. 5.4.1.7.a]. At some points along the periphery of the sample a gap had created between the shear zone and the top part of the sample [Fig. 5.4.1.7.b], while in other points some secondary shear planes appeared to have developed [Fig. 5.4.1.7.c]. Finally, the lower boundary of the shear zone appeared to be nearly horizontal [Fig. 5.4.1.8].

5.4.2 Cowden Till

a) Normally Consolidated, Non-Submerged (Tests IV.13-16)

The soil was remoulded at a water contend of 21.8 % and it was consolidated and sheared using the same procedure which was used when the non-submerged brown London clay was tested. The slow shearing stages at each normal stress level were associated with a decrease in sample height which was mainly due to the reorientation of the soil particles as the sample approached its residual state and to a lesser degree to some soil loss through the gap.

The sample was consolidated at 50 kPa and reached its slow residual strength of 35.4° [Fig. IV.13.1] before the rate of displacement was increased to 1.70 cm/sec. The measured strength reached its threshold value of 47.3° [Fig.IV.13.2] and subsequently decreased to 34.1° while the sample height increased by 0.48 mm. Further shearing at the same rate of displacement led to a decrease in sample height and an increase in the measured strength which stabilised around 42.5°. An undulating shear surface appeared to be forming with an initial amplitude of 0.09 mm which decreased and became planar before the test was terminated. Although the evidence is not conclusive about the

strength decreasing with decreasing rates of displacement, it is clear that the strength increases with increasing rates. Furthermore, it appears that the measured strength is not affected by rates of displacement above 1.55 cm/sec.

The normal stress was increased to 100 kPa, the sample was consolidated and reached its slow residual strength of 31.4° [Fig. IV.14.1] before the rate of displacement was increased to 0.09 cm/sec and the measured strength increased and stabilised at 37.7° [Fig. IV.14.2]. The rate of displacement was increased further to 1.68 cm/sec, the sample appeared to contract by a moderate 0.06 mm and its strength increased and stabilised around 42.7°. A subsequent increase in sample height by 0.05 mm appeared to have no effect on the measured strength which decreased with decreasing rates of displacement and vice versa.

At the higher normal stress of 200 kPa the sample reached its slow residual strength of 31.4° [Fig. IV.15.1] and, when the rate of displacement was increased to 1.78 cm/sec, its threshold strength of 39.5° [Fig. IV.15.2]. Further shearing at the same rate of displacement was associated with a small dilation of the sample by 0.01 mm and a drop in strength to 32.9° before the sample contracted and its strength stabilised around 38.5°. Furthermore, the results indicate that the strength decreased with decreasing rates of displacement (but it did not drop below its slow residual value) and was not affected by any changes of the rate of displacement above 1.2 cm/sec.

Finally, the sample was consolidated at 400 kPa, it reached its slow residual strength of 31.4° [Fig. IV.16.1] and, when the rate of displacement was increased to 1.69 cm/sec, its threshold strength of 38.7° [Fig. IV.16.2]. Subsequently, the sample height increased by 0.34 mm and the strength dropped to 25.6°. Further shearing at the same rate of displacement led to a decrease in sample height and an increase in strength which stabilised between 33.8° and 36.2°. An irregularly shaped shear surface appeared to be forming during the first half of this stage and that was reflected on the fluctuations of the shear stress. However, during the second half of this stage, the shear surface became planar and the measured strength became more steady. The sample strength increased with increasing rates of displacement but did not appear to be affected when the rate increased above 1.4 cm/sec.

All tests indicated an increase in strength with increasing rates of displacement and the shear zone of the sample appeared to have an irregular shape during the first part

of the fast stages of the tests which became planar as shearing progressed. Furthermore, it appeared that the magnitude of the increase in sample height, immediately after the rate of displacement was increased, controlled the magnitude of strength drop below the threshold level.

The visual inspection of the sample after it had been extruded from the confining rings revealed the existence of a shear zone with a darker colour than the rest of the sample which was divided by a distinct shear surface [Fig. 5.4.2.1]. The shear zone had a variable width which ranged from 0 to 2.0 mm and an irregular, or undulating, shape [Fig. 5.4.2.2]. The texture of the sample appeared, otherwise, firm and uniform.

b) Normally Consolidated, Submerged (Tests IV.17-20 and VI.3-4)

This sample was remoulded to an initial water content of 22 % and consolidated to 50 kPa. Having being sheared under drained conditions [Fig. IV.17.1], it reached its slow residual strength of 31.8° before the rate of displacement was increased to 1.8 cm/sec. The measured strength reached its threshold value of 45.3° [Fig. IV.17.2] and subsequently dropped to 23.5° as the sample height increased by 1.8 mm. Further shearing at the same rate of displacement led to a decrease in sample height and an increase in strength to 36.2°. An undulating shear zone appeared to be forming with an amplitude which varied between 0.24 mm and 0.6 mm. This, initially, was reflected on the measured strength which fluctuated between 32.0° and 39.1° but, as shearing progressed, the undulating shape of the shear zone was thought to induce increased pore pressures which led to a decrease in measured strength. After the sample had been sheared for 152.8 cm, the measured strength dropped below its slow residual level and, when the rate of displacement was increased further between 1.8 cm/sec and 9.9 cm/sec, decreased further and fluctuated between 16.6° and 25.7°. The amplitude of the undulating shear zone did not appear to decrease with shearing.

The same sample was consolidated to 100 kPa and reached its slow residual strength of 31.7° [Fig. IV.18.1] before the rate of displacement was increased to 1.3 cm/sec. The measured strength reached its threshold value of 39.7° [Fig. IV.18.2] and subsequently dropped to 21.6° as the sample height increased by 2.09 mm. An undulating shear zone was formed with an initial amplitude of 0.91 mm which decreased to 0.65 mm as shearing progressed. During the first 200 cm of this stage the undulating shape of the

shear surface was reflected on the measured shear stress which fluctuated between 9.02° and 16.8°. However, during the remainder of the test, where the rate of displacement was increased between 1.87 cm/sec and 10 cm/sec, the continuing undulating nature of the shear surface led to further decrease of the measured strength which varied between 4.04° and 7.2°.

Subsequently, the normal stress was increased to 200 kPa and, after consolidation was completed, the sample was sheared drained and reached its slow residual strength of 31.4° [Fig. IV.19.1]. Then, the rate of displacement was increased to 1.25 cm/sec and the measured strength reached its threshold value of 40.4° [Fig. IV.19.2] before it dropped to 31.4° and 23.4° while the sample height increased by 0.175 mm and 0.35 mm respectively. An undulating shear surface appeared to have formed, with an initial amplitude of 0.53 mm, which apparently gave rise to the pore pressures and led to further decrease in strength. The strength decreased even further, to 10.6°, and stabilised at 7.02° as shearing continued. During the second part of the test, where the rate of displacement varied between 2 cm/sec and 10 cm/sec, the measured strength initially fluctuated between 4.09° and 9.03° but, while the amplitude of the undulating shear zone decreased from 0.53 mm to 0.14 mm, it gradually increased to 15.5° at the end of the test. The variations of the rate of displacement appeared to have no significant effect on the measured strength once it dropped below its slow residual level whereas the amplitude of the undulating shear surface influenced substantially the measured strength.

Subsequently, the sample was consolidated at 300 kPa and reached its slow residual strength of 31.7° [Fig. IV.20.1] before the rate of displacement was increased to 1.35 cm/sec. The measured strength reached its threshold value of 52.0° [Fig. IV.20.2] and decreased to 29.7° while the sample height increased by 0.6 mm. An undulating shear zone formed with an initial amplitude of 0.5 mm which became more planar as the test progressed. The measured strength dropped as low as 10.2° and fluctuated between 11.3° and 18.0°. During the second part of this stage, where the rate of displacement varied between 1.8 cm/sec and 10 cm/sec, the strength appeared to be gradually increasing probably because the flattening of the shear zone allowed the pore pressures to dissipate. Before the test was terminated, the measured strength had increased to its slow residual level of 31.7°. The changes of the rate of displacement had no effect on the measured strength once it dropped below its slow residual level.

After the test was terminated, several efforts were made in order to extrude the sample intact from the confining rings. However, the material within the shear zone had softened and the top and bottom parts of the sample were separated. Nevertheless, by observing the cross sectional view of the lower part of the sample it is indicated that the shear surface has an upward conceiving shape [Fig. 5.4.2.3.a]. Evidence for the existence of thin shear zone with striations [Fig. 5.4.2.3.b] and a few irregularities were also apparent. The side view of one part of the sample which was managed to be kept intact gave evidence of a shear zone with increased voids [Fig. 5.4.2.3.c]

It has been speculated that the decrease in strength below its slow residual level during fast shearing is caused by increased pore pressures which are induced by the undulating shape of the shear zone of the sample. Therefore, another set of tests was conducted on an identically prepared Cowden till sample. A second drained shearing stage, which followed the fast shearing stage, was included in the testing procedure in order to allow for the dissipation of pore pressures, if they were the cause of the decrease in strength. The sample was remoulded to an initial water content of 22.0 %, consolidated at 100 kPa, and reached its slow residual strength of 30.7° [Fig. VI.3.1] before the rate of displacement was increased to 1.39 cm/sec. The measured strength reached its threshold value of 41.0° [Fig. VI.3.2] and subsequently decreased to 27.0° while the sample appeared to dilate by 0.89 mm. Further shearing at the same rate of displacement was associated with continuing strength decrease while an undulating shear surface, with an initial amplitude of 0.5 mm, appeared to be forming. The strength stabilised within a range between 15.0° and 18.4° but, when the rate of displacement was decreased to 0.11 cm/sec, it appeared to increase from 16.3° to 21.9°. During the second part of this fast stage, the rate of displacement was varied between 1.95 cm/sec and 10.0 cm/sec, the amplitude of the undulating shear zone decreased only to 0.39 mm, and the range within which the measured strength varied was not altered. Subsequently the rate of displacement was decreased to 0.09 mm/min and the measured strength increased gradually and reached its slow residual value after 80 mins of shearing [Fig. VI.3.3].

The same behaviour was observed when the sample was tested at 200 kPa. After reaching its slow residual strength of 30.8° [Fig. VI.4.1], the sample was sheared at 1.29 cm/sec and its strength dropped from the threshold value of 47.1° to 25.6°, while the sample height increased by 0.30 mm [Fig. VI.4.2]. As shearing continued at the same

rate of displacement, an undulating shear surface with a constant amplitude of 0.29 mm formed, and the measured strength fluctuated between 13.8° and 19.4°. When the rate of displacement was decreased to 0.09 cm/sec the strength increased from 18.4° to 26.0°, probably due to partial dissipation of pore pressures. Towards the end of the test, the measured strength decreased even further to 10.8° probably because of the persisting "pumping action" induced by the undulating shear zone. Subsequently, the rate of displacement was decreased to 0.09 mm/min and the sample regained its slow residual strength [Fig. VI.4.3].

It was observed in all tests that the measured strength was not affected by any increase in the rate of displacement once it had dropped below its slow residual level. However, there were cases where the strength increased with decreasing rates of displacement apparently because the increased pore pressures were allowed to partially dissipate.

After the test was terminated, the sample was extruded from the confining rings, visually inspected and submerged into Carbowax. The visual inspection revealed the existence of a shear zone separated from the top and bottom parts of the sample by two distinct planes [Fig. 5.4.2.4.a] with a non-uniform width which varies between 0 [Fig. 5.4.2.4.b] and 3 mm. Furthermore, several secondary shear planes were observed which at some points along the periphery of the sample were extended to its top and bottom ends [Fig. 5.4.2.4.c]. Further evidence about the shear zone were provided by the thin sections prepared from this sample [Fig. 5.TH6-7]. Also, the position of the undulating shear zone and its non-uniform width were measured for the part of the sample which was not used for the preparation of the thin sections [Fig. 5.4.2.5].

c) Overconsolidated, Submerged (Tests IV.21-23)

The soil was remoulded to an initial water content of 21.5 %, consolidated to 400 kPa and then unloaded to 100 kPa. After the end of the consolidation stage, the sample was sheared drained and reached its slow residual strength of 33.8° while the shearing matrix appeared to relax and dilate [Fig. IV.21.1]. The rate of displacement was increased to 1.68 cm/sec and the measured strength reached its threshold value of 40.6° before it dropped to 35.2°, while the sample appeared to have dilated by 0.20 mm [Fig. IV.21.2]. Subsequently, the sample height decreased and the strength increased to 40.9°.

Shearing at the same rate of displacement led to the development of an undulating shear surface with an amplitude of 0.09 mm. Some irregularities of the shear surface were reflected on the measured strength which, at some point during the test, fluctuated between 43.5° and 33.8°. However, when the regularly undulating shear surface reappeared, the strength stabilised around 41.1°. The strength decreased with decreasing rates of displacement and vice versa, did not drop below its slow residual level, and was not significantly affected by any increase in rate above 1.0 cm/sec. It was speculated that the small amplitude of the undulating shear surface was not able to induce sufficient pore pressures which would cause a decrease in measured strength.

The same sample was consolidated to 200 kPa and reached its slow residual strength of 31.1° [Fig. IV.22.1] before the rate of displacement was increased to 1.70 cm/sec. The measured strength reached its threshold value of 34.7° [Fig. IV.22.2] and subsequently dropped to 25.9° while the sample appeared to dilate by 0.3 mm. Further shearing at the same rate of displacement was associated with an apparent sample contraction and led to an increase in strength which fluctuated between 35.5° and 37.2°. An undulating and, at some points during the test, irregular shear surface, with an initial amplitude of 0.05 mm which decreased to 0.025 mm towards the end of the test, appeared to have formed. The sample strength decreased with decreasing rates of displacement but did not drop below its slow residual level and was not affected when the rate was increased above 0.96 cm/sec.

Finally, the sample was consolidated to 300 kPa and reached its slow residual strength of 31.2° [Fig. IV.23.1] before the rate of displacement was increased to 1.71 cm/sec. The measured strength reached its threshold level of 33.8° [Fig. IV.23.2] and subsequently dropped to 19.8°, while the sample height increased substantially by 1.09 mm. An undulating shear surface with an amplitude which varied between 0.31 mm and 0.68 mm appeared to be forming. This must have induced increased pore pressures and caused the measured strength to decrease further and stabilise within a range between 7.3° and 10.5°. When the rate of displacement was decreased to 0.08 cm/sec the measured strength increased by 3° to 4° within 10 cm of shearing. Towards the end of the test the measured strength increased gradually from 9.5° to 20.2° probably because of the accumulating dissipation of pore pressures which took place when the rate of displacement was decreased to 0.08 mm/min.

The sample separated into its top and bottom parts during the extrusion from the confining rings despite the efforts that were made to avoid it. The visual inspection of the parts which constituted the sample revealed that the material within the shear zone was soft and the surfaces, which separated the shear zone from the top and bottom parts of the sample, were irregular and striated [Fig. 5.4.2.6].

5.4.3 Kaolin (Tests VI.13-14)

Two tests were completed on a sample which was prepared from commercially available kaolin and was remoulded to an initial water content of 39 %. These tests were not intended to provide a comprehensive study on the behaviour of kaolin upon fast shearing but to ensure that it soil follows the same patterns as the other two plastic soils which have been investigated, *i.e.* brown London clay and Cowden till. Therefore, the same sample was tested normally and overconsolidated at 200 kPa. The procedure involved consolidating the sample to 200 kPa, shearing it drained and fast, consolidating it to 400 kPa, unloading it to 200 kPa, and repeating the shearing procedure.

Having been normally consolidated at 200 kPa, the sample reached its slow residual strength of 15.7° [Fig. VI.13.1], the rate of displacement was increased to 1.28 cm/sec, and the measured strength reached its threshold value of 25.5° [Fig. VI.13.2]. The sample did not appear to dilate, however, its strength decreased to 14.6° and, subsequently, began to increase. The first full rotation of the sample suggested the development of an undulating shear surface with an amplitude of 0.48 mm. This was reflected on the fluctuation of the measured shear stress, although the stress was also influenced by some internal ring friction which became insignificant after the first 120 cm of shearing. Furthermore, after the second rotation of the sample, the amplitude of the shear surface decreased to 0.2 mm before it became insignificant. The measured strength gradually increased to 23.4°, while the sample was being sheared at 1.28 cm/sec, and decreased with decreasing rates of displacement but at no point did it drop below its slow residual value. The fluctuations of the measured strength between 19.5° and 25.2° were attributed to non-uniformities of the shear surface. During the later part of the test, where the rate of displacement was increased between 1.74 cm/sec and 10.0 cm/sec, the measured strength varied between 19.8° and 27.9° respectively. The strength appeared to

increase continuously with increasing rates of displacement. Furthermore, it should be noted that the non-uniformities of the shear surface may be responsible for a part of the strength variations when the sample was being sheared at the same displacement rate.

Subsequently, the sample was overconsolidated at 200 kPa, reached its slow residual strength of 16.6° [Fig. VI.14.1] and, when the rate of displacement was increased to 1.30 cm/sec, its threshold strength of 29.3° [Fig. VI.14.2] before it appeared to dilate by 0.04 mm and the measured strength decreased to 28.1°. Further shearing at the same rate of displacement led to a decrease of the sample height and an increase in measured strength which fluctuated between 28.8° and 30.5°. The measured strength decreased with decreasing rates of displacement and vice versa but did not drop below its slow residual value.

This soil did not present any evidence about the response of plastic soils upon fast shearing which had not already been encountered during the previously described tests and, therefore, no additional testing was conducted. The thin sections which were prepared from this sample indicate that a shear zone of oval shape and variable width occupies the middle half of the sample [Fig. 5.TH8-9].

5.4.4 Pure Silica

The soil which was used in the following sets of tests was a commercially available non-plastic silica, non pre-treated, and remoulded to water contents which varied between 20.4 % and 21.2 %.

a) Normally Consolidated, Non-Submerged (Test V.1-4)

The sample was consolidated to 50 kPa and reached its slow residual strength of 44.4° [Fig. V.1.1] before the rate of displacement was increased to 1.7 cm/sec. The measured strength reached its threshold value of 54.3° [Fig. V.1.2] and decreased to 45.0° while the sample height increased by 0.06 mm. Further shearing at the same rate of displacement led to an increase in strength to 46.1° while the sample height decreased probably due to the contracting nature of the structure and soil loss. The rate of displacement varied between 0.1 and 1.7 cm/sec, before the test was terminated, while the measured strength fluctuated between 43.1° and 45.5° respectively.

A similar behaviour was observed at the higher normal stress of 100 kPa. Having reached its slow residual strength of 44.4° [Fig. V.2.1], the sample was sheared at 1.69 cm/sec and dilated by 0.064 mm [Fig. V.2.2] while its strength dropped from its threshold level of 51.3° to 46.3°. Further shearing at rates of displacement which varied between 0.05 and 1.69 cm/sec had no profound effect on the measured strength which fluctuated between 43.1° and 44.4°.

The sample was consolidated to 200 kPa, reached its slow residual strength of 45.4° [Fig. V.3.1] and sheared at 1.69 cm/sec. The measured strength reached its threshold value of 52.0° [Fig. V.3.2] and dropped to 44.4° while the sample dilated by 0.08 mm. The rate of displacement was varied several times between 0.087 and 1.69 cm/sec before the test was terminated, while the measured strength remained unaffected by these rate changes and fluctuated between 41.9° and 44.4°.

The same behaviour was observed at 300 kPa [Figs. V.4.1-2]. The slow residual and threshold strengths were 45.2° and 52.0°, respectively, and the sample strength during the fast shearing stage of this test fluctuated between 42.2° and 45.2° without being affected by the changes in the rate of displacement.

b) Normally Consolidated, Submerged (Tests V.5-7)

This sample exhibited an obvious decrease in strength with increasing rates of displacement. The soil was remoulded to a water content of 21 %, consolidated to 50 kPa and reached its slow residual strength of 49.2° [Fig. V.5.1] before the rate of displacement was increased to 1.2 cm/sec. The measured strength reached a threshold value of 49.2° [Fig. V.5.2], decreased and stabilised between 41.7° and 46.3°. When the rate of displacement was decreased to 0.14 cm/sec for the first time, the strength dropped to 40.4° but this response was not repeated the second time the rate was decreased to that level. During the second part of this stage, the rate of displacement was increased to 1.67 cm/sec and the strength appeared to increase to another threshold level of 51.3° before it dropped and stabilised between 40.4° and 41.7°. Variations of the rate of displacement between 1.67 and 10 cm/sec had no effect on the strength.

At the next normal stress of 100 kPa the sample reached a slow residual strength of 47.2° [Fig. V.6.1] and a threshold strength of 50.7° [Fig. V.6.2] when the rate of displacement was increased to 1.35 cm/sec. Further shearing at rates of displacement

which varied between 0.135 cm/sec and 10 cm/sec was associated with a decrease in strength which fluctuated between 40.4° and 43.5° without being affected by rate variations. However, when the shearing process paused, the sample was unloaded in shear in order to increase the displacement rates above 1.4 cm/sec, and shearing restarted, the measured strength reached a new threshold value before it decreased and fluctuated within the previously established range. The same was observed during testing at the other two normal stresses of this multistage test.

Similar behaviour was encountered when the sample was tested at 200 kPa. The measured strength reached its slow residual value of 47.2° [Fig. V.7.1] and its threshold value of 68.3° [Fig. V.7.2] before it stabilised within a range between 42.5° and 44.9° and was not affected by rate of displacement variations between 0.175 and 10 cm/sec.

c) Overconsolidated, Submerged (Tests V.8-11)

The soil was remoulded to a water content of 21.2 %, consolidated at 400 kPa, unloaded to 50 kPa, and reached its slow residual strength of 49.5° [Fig. V.8.1] before the rate of displacement was increased to 1.29 cm/sec. The measured strength reached its threshold value of 53.8° [Fig. V.8.2] and subsequently decreased and stabilised around 45.0°. This decrease was attributed to an apparent dilation of the sample by 0.12 mm at the beginning of the fast shearing stage. The variations of rate of displacement between 0.15 and 10 cm/sec had no apparent effect on the measured strength. The periodically observed sharp peaks in measured shear stress were attributed to internal ring friction.

The same behaviour was observed when the sample was tested under of 100 kPa. The measured strength reached its slow residual and threshold values of 45.8° [Fig. V.9.1] and 47.2° [Fig. V.9.2], respectively, before it decreased and fluctuated between 38.7° and 40.4°. The variations of the rate of displacement between 0.084 and 10 cm/sec had no significant effect on sample strength, whereas, the periodic increases in measured shear stress were again attributed to internal ring friction.

The sample was consolidated further to 200 kPa and 250 kPa. At 200 kPa [Figs. V.10.1-2] the measured strength reached its slow residual value of 45.8° and its threshold value of 55.0° before it decreased and fluctuated between 41.4° and 44.6°. At 250 kPa [Figs. V.11.1-2] the strength reached the slow residual and threshold values of 47.7° and 62.8°, respectively, before it decreased and stabilised around 43.5°. In both cases the

changes of the rate of displacement between 0.059 and 10 cm/sec (at 200 kPa) and 1.69 and 6.55 cm/sec (at 250 kPa) had no effect on the measured strength.

In all tests the measured strength appeared to reach a new threshold strength when the shearing process was paused, the sample unloaded in shear, and shearing restarted at rates of displacement above 1.4 cm/sec. This apparent increase in strength was always followed by a decrease to the level at which the strength had stabilised before the previously mentioned pause.

5.4.5 Artificial Sandy Silt

This set of tests was conducted on material which was prepared by thoroughly mixing, in the laboratory, dry, commercially available, pure silica (3 parts) with dry, fine (0.150 mm < d < 0.300 mm), clean sand (1 part) and remoulding the mixture to a water contend around 20%.

a) Normally Consolidated, Non-Submerged (Tests V.12-15)

The sample was remoulded to a water content of 20.2 %, consolidated to 50 kPa and reached its slow residual strength of 50.6° [Fig. V.12.1] before the rate of displacement was increased to 1.70 cm/sec. The measured strength reached its threshold value of 55.6° [Fig. V.12.2] and, subsequently, decreased, while the sample appeared to dilate by 0.135 mm, and stabilised within a range between 44.9° and 48.1°. The rate of displacement was varied between 0.11 cm/sec and 1.70 cm/sec without having any profound effect on the measured strength or the behaviour of the sample.

A more pronounced decrease in strength below its slow residual value and a fluctuation within a wider range were observed when the sample was tested at 100 kPa. After the sample reached its slow residual strength of 45.0° [Fig. V.13.1], the rate of displacement was increased to 1.69 cm/sec and the measured strength increased to its threshold level of 51.3° [Fig. V.13.2]. Further shearing at the same rate of displacement was associated with an apparent dilation of the sample by 0.105 mm and a decrease in measured strength. The strength fluctuated initially between 41.1° and 45.0° and, after the sample had been sheared for 150 cm, between 35.5° and 41.1°. Rate of displacement variations between 0.093 and 1.69 cm/sec had no significant effect on the strength.

Similar behaviour was observed when the sample was tested under 200 kPa and 250 kPa. After being consolidated at 200 kPa the sample reached its slow residual strength of 45.0° and, subsequently, the rate of displacement was increased to 1.69 cm/sec [Figs. V.14.1-2]. The measured strength decreased from its threshold value of 51.3°, while the sample dilated by 0.075 mm, and fluctuated between 40.2° and 43.9° independently of the rate of displacement which varied between 0.11 cm/sec and 1.69 cm/sec. Finally, after being consolidated at 250 kPa, the sample reached its slow residual strength of 46.3° [Fig. V.15.1], the rate of displacement was increased to 1.72 cm/sec and the strength dropped from its threshold value of 50.2° to a range between 42.0° and 45.0° [Fig. V.15.2]. The decrease in strength, after it had reached its threshold level, was associated with a apparent dilation of the sample by 0.102 mm. The rate of displacement which varied between 0.107 and 1.72 cm/sec had no effect on the measured strength.

b) Normally Consolidated, Submerged (Tests V.16-17)

This sample was remoulded to a water content of 20.2 %, consolidated to 50 kPa and reached its slow residual strength of 47.7° [Fig. V.16.1] before the rate of displacement was increased to 1.69 cm/sec. The measured strength reached its threshold value of 54.8° [Fig. V.16.2] and, while the sample appeared to dilate by 0.09 mm, decreased and stabilised within a range between 39.1° and 43.9°. The changes in the rate of displacement between 0.09 and 1.69 cm/sec had no effect on the strength of the sample. A similar behaviour was observed at the higher normal stress of 100 kPa [Figs. V.17.1-2]. The measured strength reached its slow residual and threshold levels of 48.0° and 52.9°, respectively, before it decreased and fluctuated between 42.0° and 46.7°. The decrease in strength was attributed to an apparent dilation of the sample by 0.11 mm while the variations of the rate of displacement between 0.11 cm/sec and 1.68 cm/sec had no effect on the behaviour of the sample.

c) Overconsolidated, Submerged (Tests V.18-20)

Having been remoulded to a water content of 19.9 %, the sample was consolidated to 400 kPa, unloaded to 100 kPa and sheared until it reached its slow residual strength of 46.4° [Fig. V.18.1]. The rate of displacement was increased to 1.25 cm/sec, the measured strength reached its threshold value of 49.7° [Fig. V.18.2] and

decreased, while the sample appeared to dilate by 0.14 mm, to a range between 42.0° and 45.9°. The sharp peaks associated with the measured shear stress may be partly attributed to internal ring friction. The changes of the rate of displacement between 0.07 and 10 cm/sec had no effect on the strength and behaviour of the sample.

The sample was consolidated further to 200 kPa and reached its slow residual strength of 46.9° [Fig. V.19.1] before the rate of displacement was increased to 1.5 cm/sec. The measured strength decreased from its threshold level of 50.4° to a range between 43.7° and 45.9°, while the sample appeared to have dilated initially by 0.075 mm [Fig. V.19.2]. Variations of the rate of displacement between 0.08 and 10 .0 cm/sec had no effect on the measured strength.

Similar behaviour was observed when the sample was tested under 250 kPa [Figs. V.20.1-2]. Having reached its slow residual and threshold strengths of 49.0° and 55.2°, respectively, the sample appeared to dilate by 0.105 mm and its strength decreased and fluctuated between 43.9° and 47.7°. The varying rate of displacement had no effect on the behaviour of the sample.

The slow shearing stages of all tests conducted on this soil were associated with an apparent dilation of the sample. This may be considered as an indication of the turbulent nature of the shearing mechanism. The turbulent shear mode may also be used to explain the decrease in measured strength during fast shearing. Finally, it should be mentioned that, in cases where the shearing process was paused and restarted at higher rates of displacement, a new threshold strength was observed before the strength decreased and fluctuated within its pre-established range.

5.4.6 Coarse Uniform Clean Sand

In order to investigate the response of sand upon shearing at varying rates of displacement, a commercially available, uniform, coarse (1.18 mm < d < 2.00 mm), clean sand was used. Two series of tests were conducted, one on a completely dry sample and another on a wet, non- submerged sample, so that the influence of water on the behaviour of this soil type could be assessed. The procedure which was followed in these sets of tests involved consolidating and shearing the samples at a constant rate of displacement between 0.075 and 0.085 cm/sec until the measured strength had reached a

constant value. Subsequently, the rate of displacement was increased gradually to a level between 1.50 and 1.60 cm/sec and decreased in the same manner before the test was terminated.

a) Normally Consolidated, Completely Dry (Tests V.21-23)

The sample was consolidated and sheared progressively at 50, 100, and 200 kPa and its strength stabilised at 36.2°, 25.2°, and 26.1°, respectively [Figs. V.21.1-23.1]. The changes in the rate of displacement had no effect on the behaviour and strength of the sample. The nature of the sample resulted in vibrations of the system which were apparent in the test results. The variation of the residual strengths measured during each test was attributed mainly to the fact that particle crushing during shearing would have resulted in different structures at increasing normal stress. An indication of the magnitude of particle crushing may be given by the decrease in sample height during testing, since no soil loss was observed. The results from this and the following set of tests should not be considered for the determination of the residual strength of the original sample but for the evidence they provide about the behaviour of sand on fast shearing. Finally, it should be noted that the sample appeared to dilate at the beginning of each test. This dilation was 1.43 mm when the sample was sheared at 50 kPa but, as the normal stress increased and the interparicle movements became more restricted, its magnitude decreased.

b) Normally Consolidated, Moist (Tests 24-27)

The soil was submerged in water, placed into the confining rings and consolidated. The sample was tested under 50, 100, 200, and 400 kPa and its residual strength was 35.8°, 25.0°, 28.8°, and 20.3°, respectively [Figs. V.24.1-27.1]. A dilation of 0.43 mm was observed during the first test at 50 kPa, whereas, particle crushing increased substantially with increasing normal stress. No rate effect was observed.

5.5 Series of Tests on Completely Dry Soil

The presence of water in the environment of the sample and the increase of pore pressures appear to affect greatly the behaviour of plastic soils during fast shearing. In order to further investigate the effect of the presence of water in the behaviour of plastic

soils, a series of tests was conducted on completely dry samples. The testing procedure which was followed was similar to the general testing procedure, which has been used in this investigation, with the exemption that the same sample was used for the normally consolidated and the overconsolidated tests.

a) Brown London Clay (Tests VI.5-8)

The sample was consolidated at 100 kPa and reached its slow residual strength of 37.4° [Fig. VI.5.1] before the rate of displacement was increased to 1.08 cm/sec. The measured strength reached its threshold value of 43.0° [Fig. VI.5.2] and subsequently dropped to 30.0° while the sample appeared to dilate by 0.15 mm. Further shearing at rates of displacement which varied between 0.11 and 10 cm/sec led to a gradual increase in strength from 30.3° to 37.4° to 41.9°. This increase was attributed to the decrease in the sample height which must have been caused by the contraction of the sample and by the loss of soil which must have resulted in new material entering the shear zone. The variations of the rate of displacement had no significant effect on the measured strength while the variations of the strength were attributed to the shape of the shear zone.

Having been consolidated at 200 kPa, the sample reached its slow residual and threshold strength of 37.6° and 42.3°, respectively [Figs. VI.6.1-2]. The changes in the rate of displacement between 0.1 and 9.02 cm/sec had no effect on the measured strength which varied between 41.2° and 43.8° throughout the duration of the test. The fluctuations of the measured strength were attributed to the shape and the non-uniformities of the shear zone.

The same sample was consolidated to 400 kPa, unloaded to 100 kPa, and reached its slow residual strength of 42.1° [Fig. VI.7.1] before the rate of displacement was increased to 0.16 cm/sec. The measured strength reached its threshold value of 44.7° [Fig. VI.7.2] but, while the rate of displacement continued to increase to 1.36 cm/sec, the strength also increased to 45.6°. Further shearing at the same rate of displacement led to a drop in strength which fluctuated between 38.8° and 42.1° throughout the duration of this test. The changes in the rate of displacement between 0.16 cm/sec and 9.14 cm/sec had no effect on the measured strength. A similar behaviour was observed when the sample was consolidated to 200 kPa. Having reached the slow residual strength of 41.5° [Fig. VI.8.1], the rate of displacement was increased to 1.15 cm/sec. The measured

strength reached its threshold value [Fig. VI.8.2] which coincided with the slow residual value and, for the first 100 cm of shear displacement, varied between 42.3° and 45.3°. Further shearing at the same rate of displacement led to a decrease in strength which fluctuated between 38.0° and 41.5°.

b) Cowden Till (Tests VI.9-12)

The sample was consolidated to 100 kPa and reached its slow residual strength of 42.0° [Fig. VI.9.1] before the rate of displacement was increased to 1.72 cm/sec. The measured strength reached its threshold value of 48.0° [Fig. VI.9.2], dropped to 40.5°, while the sample appeared to dilate by 0.02 mm, and continued to fluctuate between 36.9° and 42.8°. The wide range of this fluctuation is partly attributed to internal ring friction. The variations of the rate of displacement between 0.09 and 1.72 cm/sec had no further effect on the measured strength. A similar behaviour was observed when the sample was sheared under 200 kPa. After the sample had reached its slow residual strength of 41.4° [Fig. VI.10.1], the rate of displacement was increased to 1.69 cm/sec and the measured strength reached its threshold value of 42.0° [Fig. VI.10.2]. Further shearing at rates of displacement which varied between 0.11 and 1.69 cm/sec led to a decrease in strength which fluctuated between 38.0° and 41.4°. Variations in the rate of displacement had no significant effect on the measured strength which was affected by some internal ring friction.

The same sample was consolidated to 400 kPa, unloaded to 100 kPa, and reached its slow residual strength of 42.6° [Fig. VI.11.1] before the rate of displacement was increased to 1.71 cm/sec. The measured strength reached its threshold value of 44.3° [Fig. VI.11.2] and subsequently dropped and stabilised between 35.5° and 38.5°. Apparently the overconsolidation procedure must have caused additional contamination of the rings which led to increased shear stress measurements at one point along their periphery. The periodic increases in measured strength should not be taken into account upon analysing these tests. Therefore, the measured strength should be considered to have stabilised within the aforementioned range. The changes in the rate of displacement between 0.08 and 1.71 cm/sec had no further effect on the behaviour or strength of the sample. Similar behaviour was observed when the sample was sheared under 200 kPa. The rate of displacement was increased from 0.09 mm/min to 1.70 cm/sec and the

measured strength increased from its slow residual value of 41.8° to its threshold value of 42.9° [Fig. VI.12.1-2]. The subsequent increase in measured strength was thought to be artificial because it was attributed to internal friction caused by ring contamination. The periodic increase in measured strength attributed to internal ring friction decreased as the test progressed and should not be taken into account in the analysis of the results. The measured strength decreased and fluctuated between 37.8° and 39.8° until the test was terminated. The changes in the rate of displacement between 0.073 cm/sec and 1.70 cm/sec had no further effect on the strength or the behaviour of the sample.

5.6 Tests on "Intact" Samples

In order to investigate the effect which the presence of a pre-existing shear zone has on the behaviour of soils upon fast shearing, two sets of tests were conducted, one on an overconsolidated brown London clay sample and another on a normally consolidated Cowden Till sample. Both samples were prepared according to the standard preparation procedure and were sheared at rates of displacement which varied between 0.04 cm/sec and 1.40 cm/sec immediately after the end of the consolidation stage. After testing the sample at 100 kPa, the sample was unloaded in shear, the normal stress was increased to 200 kPa, and shearing restarted. In a sense, only the first test was conducted on an intact sample. Comparisons between the measured strength during fast shearing and the slow residual strength were made in reference to similarly prepared and tested samples from the earlier parts of this investigation.

a) Brown London Clay (Tests VI.15-16)

The sample was remoulded to a water content of 35.3 %, consolidated to 400 kPa, unloaded to 100 kPa, and sheared at an initial rate of displacement of 1.29 cm/sec [Fig. VI.15.1]. The measured strength reached a threshold value of 49.5° and subsequently decreased to 26.0° while the sample appeared to dilate by 0.52 mm. Further shearing at the same rate of displacement was associated with a decrease in sample height and an increase in strength which fluctuated between 27.8° and 33.1°. The shear surface appeared initially to have an undulating shape which was being reflected on the measured strength. However, after the third rotation of the sample, the shear zone

became irregular. Due to irregularities of the shear zone the measured strength decreased further and fluctuated between 19.0° and 24.2° while the sample was still being sheared at 1.29 cm/sec. Towards the end of the test, while the shear surface had become less irregular, the strength decreased with decreasing rates of displacement and vice versa. Since the sample had not been pre-sheared to its residual state, no comparison with the slow residual strength was made.

A similar but more steady response was observed when the sample was tested under 200 kPa [Fig. VI.16.1]. The sample was sheared at 1.40 cm/sec and its strength fluctuated between 18.9° and 21.1°. An undulating shear surface appeared to be forming with an initial wave-length greater than one sample perimeter. During the middle third of the test the oscillations of the shear zone became more apparent, with an amplitude of 0.25 mm and a wave-length of 40 cm. This led to a decrease in measured strength which fluctuated between 16.6° and 19.6°. During the last third of the test the shear surface had became nearly planar and the strength increased and varied between 18.1° and 19.6°. The strength decreased with decreasing rates of displacement and vice versa.

After the test was terminated, the sample was extruded from the confining rings, visually inspected, one part was submerged in Carbowax and the remainder was left to air-dry. The shear zone was of variable width and separated from the top and bottom parts of the sample by two main and several secondary shear surfaces [Fig. 5.6.1.a-b]. The shear surfaces were polished and the lower one had a conceiving shape which suggested the existence of an oval-shaped shear zone [Fig. 5.6.1.c-d]. Furthermore, the irregularities of the shear zone may be observed by the top view of the dried sample which was split into its top and bottom parts [Fig. 5.6.1.e]. More evidence on the condition of the sample after testing and the shape of the shear zone may be provided by the thin sections [Fig. 5.TH10-11].

b) Cowden till (Tests VI.17-18)

The sample was remoulded at a water content of 21 %, consolidated to 100 kPa, and sheared initially at 1.26 cm/sec [Fig. VI.17.1]. The measured strength reached its threshold value of 34.2° and subsequently decreased while the sample appeared to contract by 0.42 mm. The contraction of the sample was thought to have induced increased pore pressures which resulted in a decrease in measured strength. Further

shearing at the same rate of displacement was associated with an increase in sample height which, combined with the increased pore pressures already mentioned, led to further decrease in strength. The strength stabilised within a range between 8.35° and 14.0° and appeared to increase when the rate of displacement was decreased to 0.158 cm/sec, probably because shearing at that rate may have allowed for a partial dissipation of pore pressures. Other than that, the measured strength was not affected by the changes in the rate of displacement.

The same sample was consolidated to 200 kPa and sheared initially at 1.28 cm/sec [Fig. VI.18.1]. The measured strength reached its threshold value of 28.8° and subsequently dropped to 23.4° while the sample appeared to contract by 0.04 mm. Further shearing at the same rate of displacement led to an increase in sample height by 0.52 mm and the development of an undulating shear surface with an initial amplitude of 0.48 mm. The shear surface retained its undulating shape throughout the test and its amplitude varied between 0.57 mm and 0.24 mm. Reflecting the shape of the shear surface, the strength fluctuated between 20.9° and 26.6° during the first 160 cm of shearing before it stabilised within a range between 7.24° and 10.04°. When the rate of displacement was decreased to 0.046 cm/sec the strength appeared to increase, probably due to dissipation of pore pressures. However, once the measured strength dropped to its lowest level, it became insensitive to any increase in the rate of displacement.

The drop in strength below its slow residual value has been encountered in several tests during this investigation and it has been speculated that it is caused by increased pore pressures within the shear surface of the sample which are induced by a "pumping action" generated by the undulating shape of the shear surface. The water content of the sample was measured at three points along its height, the top, the bottom and the middle. The results indicated that while the soil at the top and bottom parts of the sample was around its plastic limit, the soil within the shear zone was close to its liquid limit [Table 5.2.2].

The visual inspection of the sample after it had been extruded from the confining rings indicated that the shear zone consisted of a soft material which appeared to have higher porosity than the remainder of the sample and an irregular, or undulating, shape [Figs. 5.6.2-3]. Further evidence about the nature and shape of the shear zone are provided by the thin sections [Fig. 5.TH12-13].

5.7 Tests on Reversed Direction of Shearing

Finally, for the completion of this investigation two sets of tests were conducted in order to investigate the effect which the reversing of the shearing direction may have on the residual soil strength. One set of tests was conducted on an overconsolidated brown London clay sample and another on a normally consolidated Cowden till sample. The samples were consolidated and sheared drained until they established their residual structure before the rate of displacement was increased. While the sample was being sheared at high rates of displacement, the shearing direction was reversed repeatedly and the changes in strength were measured.

a) Brown London Clay (Tests VI.19-20)

The sample was remoulded to a water content of 36.2 %, consolidated to 400 kPa, unloaded to 100 kPa, and reached its slow residual strength of 10.4° [Fig. VI.19.1] before the rate of displacement was increased to 1.74 cm/sec. The measured strength reached its threshold value of 28.4° [Fig. VI.19.2] and began to decrease. While the strength was decreasing, the shearing direction was reversed and the strength continued to decrease. The direction of shearing was reversed ten times before this test was terminated. It was concluded that the direction of shearing has no significant effect on the measured strength. The sample exhibited the rate dependent characteristics which had been observed in previous tests. The same response upon reversing the direction of shearing was observed when the sample was tested under 200 kPa [Figs. VI.20.1-2]. The thin sections which were prepared from this sample suggest that a main shear surface had developed at its mid-height and the secondary shear planes, which may be connected to the main, constitute a zone of material which is thought to be involved in the shearing mechanism [Fig. 5.TH14-15].

b) Cowden till (Tests VI.21-22)

The sample was remoulded to a water content of 21.3 %, consolidated to 100 kPa, and reached its slow residual strength of 38.6° [Fig. VI.21.1] before the rate of displacement was increased to 1.38 cm/sec. The measured strength reached its threshold value of 45.3° [Fig. VI.21.2] and began to drop. The decrease of the measured strength

continued when the shearing direction was reversed. Throughout the duration of the test the direction of shearing was reversed eleven times. The results indicate that the measured strength was not significantly affected by the reversing of the shearing direction. However, when the sample was being sheared at a low rate of displacement and the direction of shearing was reversed, the measured strength regained its previous magnitude gradually. This may suggest that the particles, which had already been aligned, require some shear displacement in order to re-establish their position in the opposite direction. The same response and similar results were observed when the sample was tested under 200 kPa [Figs. VI.22.1-2]. Nevertheless, as it has been indicated by the results from the tests on brown London clay, the measured strength regained its previously established value when the direction of shearing was reversed. The thin sections which were prepared from this sample do not give clear evidence about the shape and size of the shear zone [Fig. 5.TH16-17].

5.8 Summary

The results from the tests which have been conducted for the purposes of this investigation were presented in this chapter [Table 5.7.1]. It has been observed that the plastic soils exhibit a positive rate effect as their strength increases above its slow residual value with increasing rates of displacement. However, in cases where the shape of the shear zone is not planar and the sample is submerged in water, an apparent increase in pore pressures within the shearing soil matrix has been shown to lead to substantial decreases in strength during fast shearing. This is been shown not to apply for the non-submerged samples. Furthermore, it has been shown that non-plastic soils exhibit a negative rate effect and their strength decreases below its slow residual value when the rate of displacement increases. Nevertheless, this decrease in not as substantial as the one which is associated with the plastic soils. Similar behaviour was observed in the case of completely dry samples of plastic soils. The clean, coarse sand which was used in this investigation did not appear to be affected by any variations in the rate of the displacement. Finally, the reversing of the shearing direction did not appear to have a significant effect on the measured soil strength.

Soil Type	Atterberg Limits (%)			Clay Fraction (%)	Test Type	Rate Effect	Water Content (%)	
	PL	LL	PI				Before Testing	After Testing
Kaolin	36	72	36	82	NC OC	Positive	39	33.3
brown London clay	28	69	41	58	NC NS	Positive	36	31.2
					NC S	Positive	36.2	33.1
					OC S	Negative	35.9	32.6
					OC S 3St	Positive	36	32.3
					OC S Int	Positive	35.3	31.8
					OC S Rv	Positive	36.2	32.2
Cowden Till	15	28	13	31	NC NS	Positive	21.8	14.7
					NC S	Negative	22	18.2
					OC S	Positive	21.5	16.5
					NC S 3St	Negative	22	16.3
					NC S Int	Negative	21	17.7*
					NC S Rv	Positive	21.3	14.4
Artificial sandy silt	-	28	-	7	NC NS	Negative	20	19.1
					NC S	Negative	20.2	19.3
					OC S	Negative	19.9	19.7
Pure silica	-	27	-	9	NC NS	Negative	20.4	18
					NC S	Negative	21	19.2
					OC S	Negative	21.2	23.8
* Refer to Table 5.2.2								

Table 5.2.1 Water Contents of Samples Before and after Testing

Water Content at	(%)
Top	17.7
Middle	25.4
Bottom	15.1

Table 5.2.2 Water Contents at Different Positions for Cowden Till - NC S Int

TABLE 5.7.1 Tests performed on the ring shear apparatus

soil type and condition	test number	normal stress (kpa)	slow residual strength (degrees)	maximum shearing rate (cm/sec)	shear displace-ment (cm)	shear zone shape	rate effect	remarks
Brown London clay (NC) non submerged	IV.1	50	15.6	1.8	540	oscillating	positive	apparatus effect
	IV.5	50	12.3	1.45	510	oscillating	positive	
	IV.2	100	11.7	1.55	430	planar	positive	
	IV.3	200	11.3	1.35	310	planar	positive	
	IV.4	400	12.4	1.38	275	non-planar	positive	
Brown London clay (NC) submerged	IV.6	50	14.5	1.86	510	planar	positive	
	IV.8	100	11.3	1.8	350	oscillating	negative	
	IV.9	200	11.9	1.85	410	planar	positive	
	IV.7	400	11.3	1.3	360	planar	positive	
Brown London clay (OC) submerged	IV.10	100	15.5	4.5	670	oscillating	negative	
	IV.11	200	11.4	9.3	550	oscillating	negative	
	IV.12	300	12.3	10	480	oscillating	negative	
Cowden Till (NC) non submerged	IV.13	50	35.4	1.85	360	non-planar	positive	
	IV.14	100	31.4	1.85	335	planar	positive	
	IV.15	200	31.4	1.84	280	planar	positive	
	IV.16	400	31.4	1.83	450	planar	positive	
Cowden Till (NC) submerged	IV.17	50	31.8	10	430	oscillating	negative	
	IV.18	100	31.7	10	570	oscillating	negative	
	IV.19	200	31.4	10	570	oscillating	negative	
	IV.20	300	31.7	9.5	280	oscillating	negative	
Cowden Till (OC) submerged	IV.21	100	33.8	1.85	470	planar	positive	
	IV.22	200	31.1	1.86	435	planar	positive	
	IV.23	300	31.2	1.9	300	oscillating	negative	
Pure Silica (NC) non submerged	V.1	50	44.4	1.87	380	planar	none	
	V.2	100	44.4	1.86	440	planar	negative	small drop below slow residual
	V.3	200	45.4	1.8	335	planar	negative	
	V.4	300	45.2	1.95	430	planar	negative	
Pure Silica (NC) submerged	V.5	50	49.2	10	590	non-planar	negative	small drop below slow residual
	V.6	100	47.2	10	730	oscillating	negative	
	V.7	200	47.2	10	640	planar	negative	
Pure Silica (OC) submerged	V.8	50	49.5	10	660	planar	negative	small drop below slow residual
	V.9	100	45.8	10	630	planar	negative	
	V.10	200	45.8	10	660	planar	negative	
	V.11	250	47.7	6.55	1,000	planar	negative	

CONTINUES

soil type and condition	test number	normal stress (kpa)	slow residual strength (degrees)	maximum shearing rate (cm/sec)	shear displace-ment (cm)	shear zone shape	rate effect	remarks
Sandy Silt (NC) non submerged	V.12	50	50.6	1.85	425	planar	negative	insignificant rate effect
	V.13	100	45	1.85	450	planar	negative	
	V.14	200	45	1.85	430	planar	negative	
	V.15	250	46.3	1.8	480	planar	negative	
Sandy Silt (NC) submerged	V.16	50	47.7	1.85	480	planar	negative	small drop below slow residual
	V.17	100	48	1.85	450	planar	negative	
Sandy Silt (OC) sub-merged	V.18	100	46.4	10	660	planar	negative	insignificant rate effect
	V.19	200	46.9	10	640	planar	negative	
	V.20	250	49	8.51	660	planar	negative	
Coarse Sand (NC) dry	V.21	50	36.2	1.5	170	planar	none	excessive vibrations observed
	V.22	100	25.2	1.54	158	planar	none	
	V.23	200	26.1	1.58	162	planar	none	
Coarse Sand (NC) moist	V.24	50	35.8	1.6	162	planar	none	excessive vibrations observed
	V.25	100	25	1.54	166	planar	none	
	V.26	200	28.8	1.54	162	planar	none	
	V.27	400	20.3	1.54	144	planar	none	
Kaolin (NC)	VI.13	200	15.7	10	770	planar	positive	
Kaolin (OC)	VI.14	200	16.6	1.51	335	planar	positive	
Brown London Clay (OC)	VI.1	100	10.9	10	660	planar	positive	3-stage test
	VI.2	200	11	10	600	planar	positive	
Cowden Till (NC)	VI.3	100	30.7	10	610	oscillating	negative	3-stage test
	VI.4	200	30.9	1.37	295	oscillating	negative	
Brown London Clay (NC)	VI.5	100	37.4	10	570	planar	none	dry sample
	VI.6	200	37.6	10	650	planar	none	
Brown London Clay (OC)	VI.7	100	42.1	10	570	planar	none	
	VI.8	200	41.5	10	570	planar	none	
Cowden Till (NC)	VI.9	100	42	1.8	435	planar	none	dry sample
	VI.10	200	41.4	1.8	480	planar	none	
Cowden Till (OC)	VI.11	100	42.6	1.8	590	planar	none	
	VI.12	200	37.8	1.8	550	planar	none	
Brown London Clay (OC)	VI.15	100	-	1.5	435	non-planar	positive	intact sample
	VI.16	200	-	1.5	410	planar	positive	
Cowden Till (NC)	VI.17	100	-	1.5	425	oscillating	negative	intact sample
	VI.18	200	-	1.5	390	oscillating	negative	

CONTINUES

soil type and condition	test number	normal stress (kpa)	slow residual strength (degrees)	maximum shearing rate (cm/sec)	shear displace -ment (cm)	shear zone shape	rate effect	remarks
Brown London Clay (OC)	VI.19	100	10.4	1.8	580	planar	positive	reversed direction
	VI.20	200	10.2	1.8	550	planar	positive	
Cowden Till (NC)	VI.21	100	38.6	1.8	535	planar	positive	reversed direction
	VI.22	200	38	1.8	520	planar	positive	



Fig. 5.4.1.1 a) **Shear zone of normally consolidated, non-submerged brown London clay sample (50 kPa)**



Fig. 5.4.1.1 b) **Shear zone irregularity of normally consolidated, non-submerged brown London clay sample (50 kPa)**

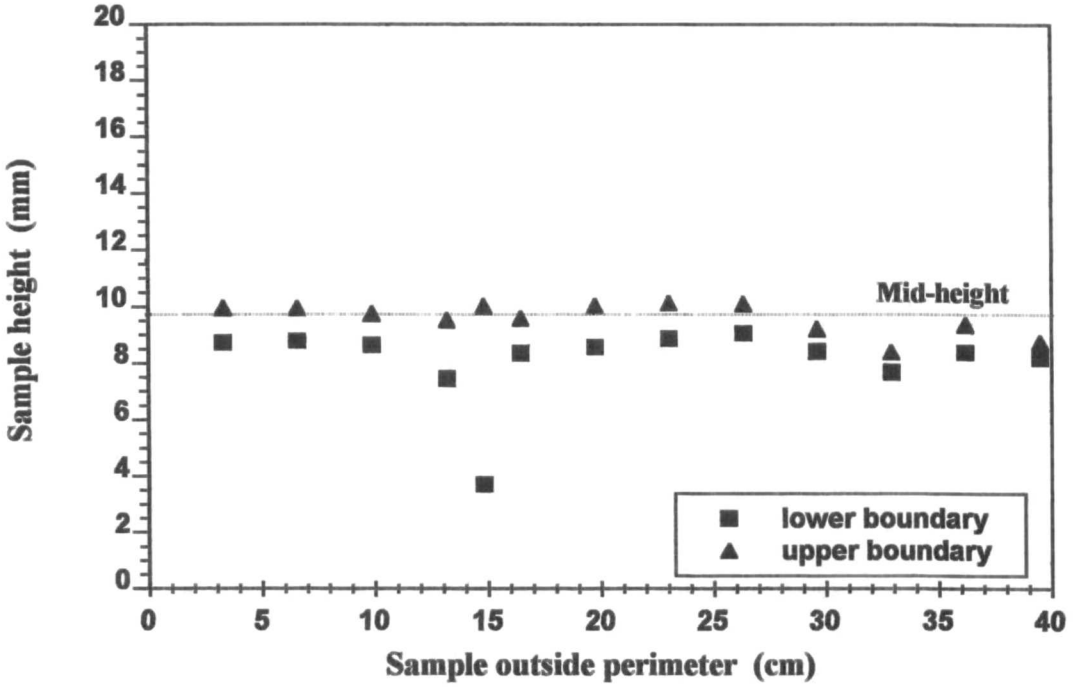


Figure 5.4.1.2 Shape and position of the shear zone of the normally consolidated, non-submerged, brown London clay sample (Test IV.5)



Fig. 5.4.1.3 a) Shear zone of normally consolidated, submerged brown London clay sample (100 & 200 kPa)



Fig. 5.4.1.3 b) **Shear zone of normally consolidated, submerged brown London clay sample (100 & 200 kPa)**



Fig. 5.4.1.3 c) **Shear zone of normally consolidated, submerged brown London clay sample (100 &200 kPa)**

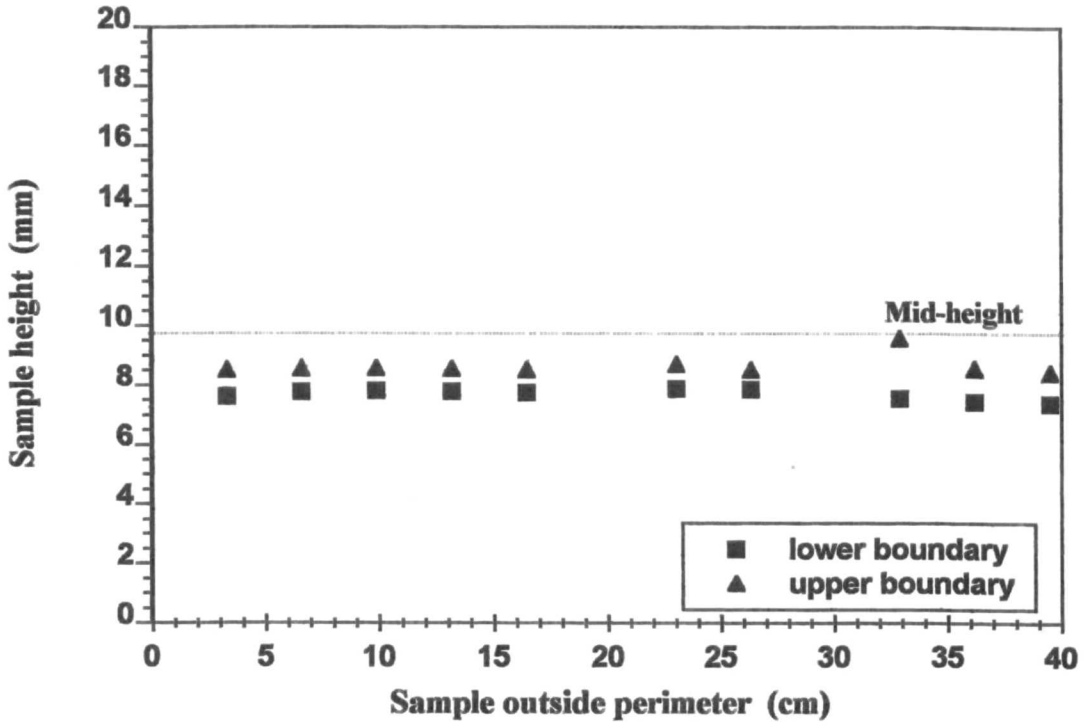


Figure 5.4.1.4 Shape and position of the shear zone of the second normally consolidated, submerged, brown London clay sample (100 & 200 kPa)

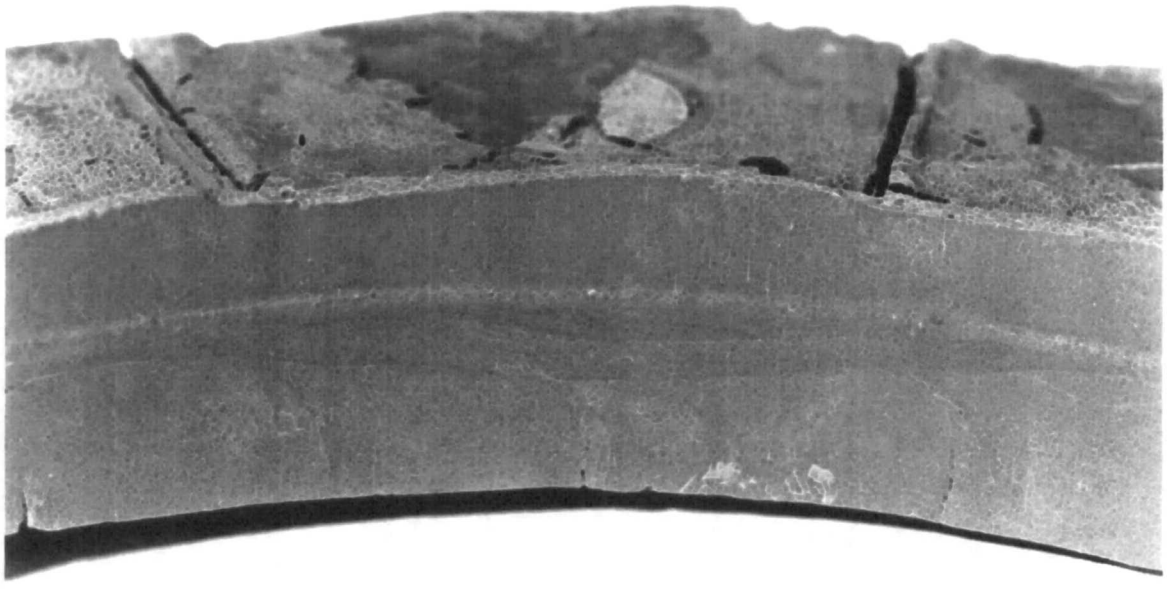


Fig. 5.4.1.5 a) Shear zone of over-consolidated, submerged brown London clay sample



Fig. 5.4.1.5 b) **Shear zone of over-consolidated, submerged brown London clay sample**



Fig. 5.4.1.5 c) **Shear zone of over-consolidated, submerged brown London clay sample**

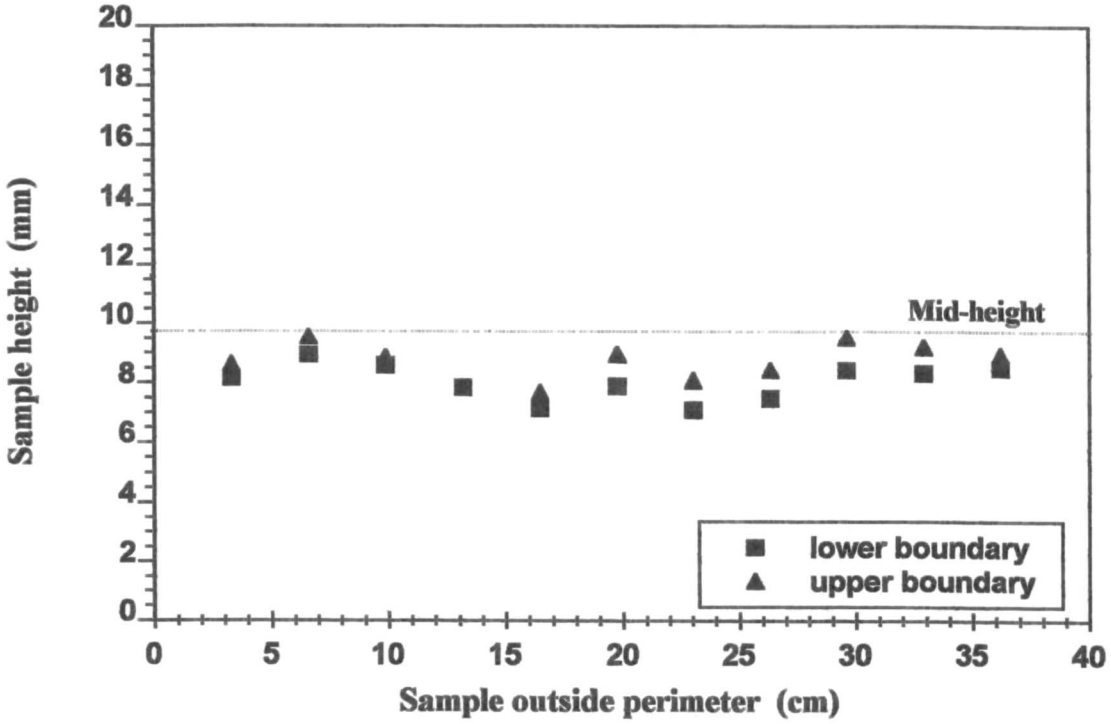


Figure 5.4.1.6 Shape and position of the shear zone of the first over-consolidated, submerged, brown London clay sample (100-300 kPa)



Fig. 5.4.1.7 a) Shear zone of second over-consolidated, submerged brown London clay sample (3-stage test)



Fig. 5.4.1.7 b) **Shear zone of second over-consolidated, submerged brown London clay sample (3-stage test)**



Fig. 5.4.1.7 c) **Shear zone of second over-consolidated, submerged brown London clay sample (3-stage test)**

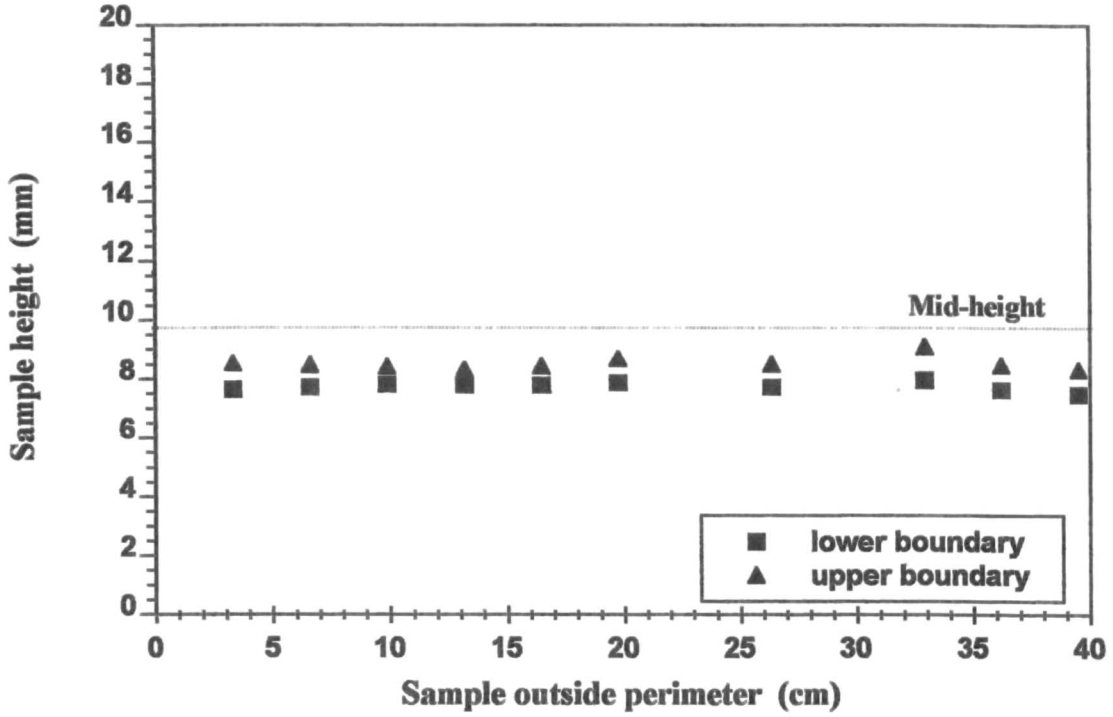


Figure 5.4.1.8 Shape and position of the shear zone of the second over-consolidated, submerged, brown London clay sample (3-stage test)



Fig. 5.4.2.1 a) Shear zone of normally consolidated, non-submerged Cowden till

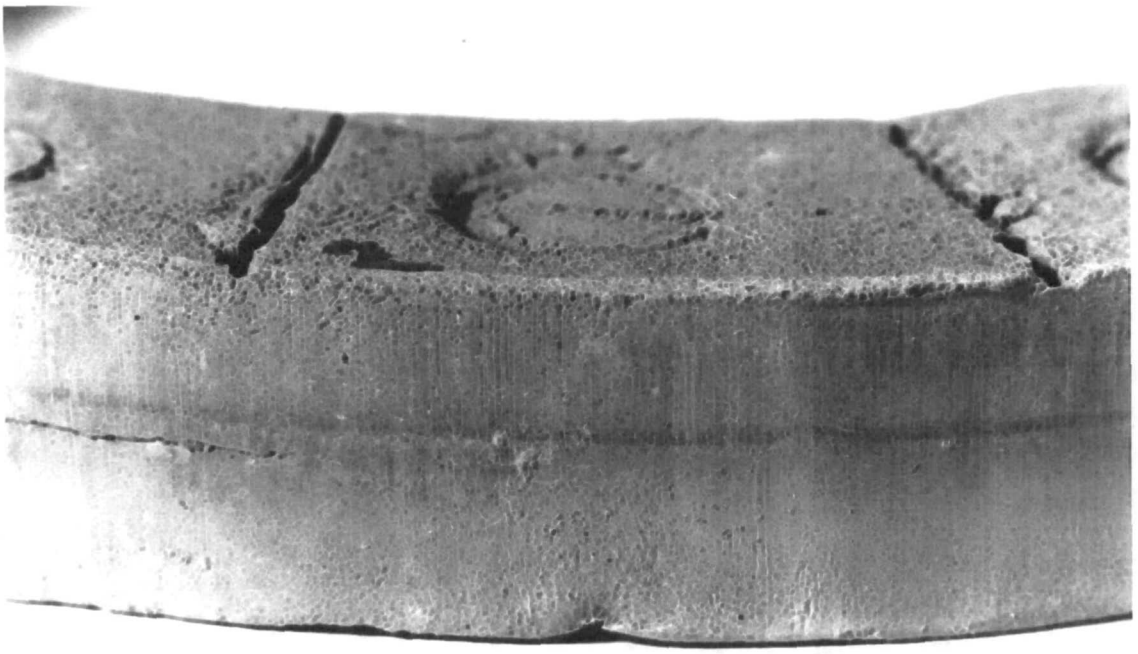


Fig. 5.4.2.1 b) Shear zone of normally consolidated, non-submerged Cowden till

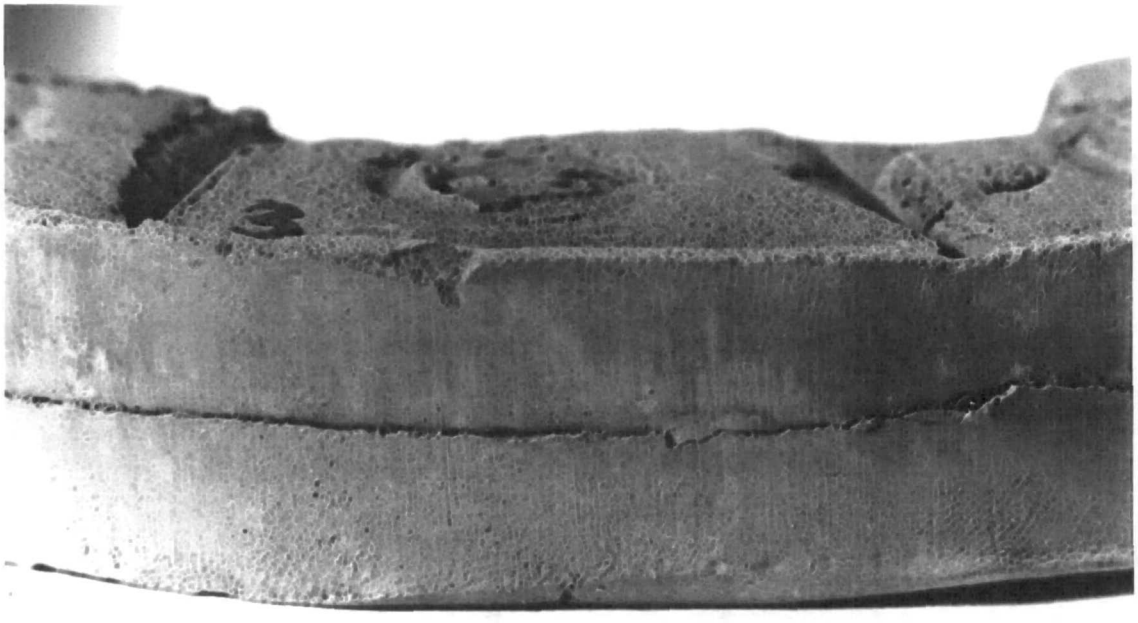


Fig. 5.4.2.1 c) Shear zone of normally consolidated, non-submerged Cowden till

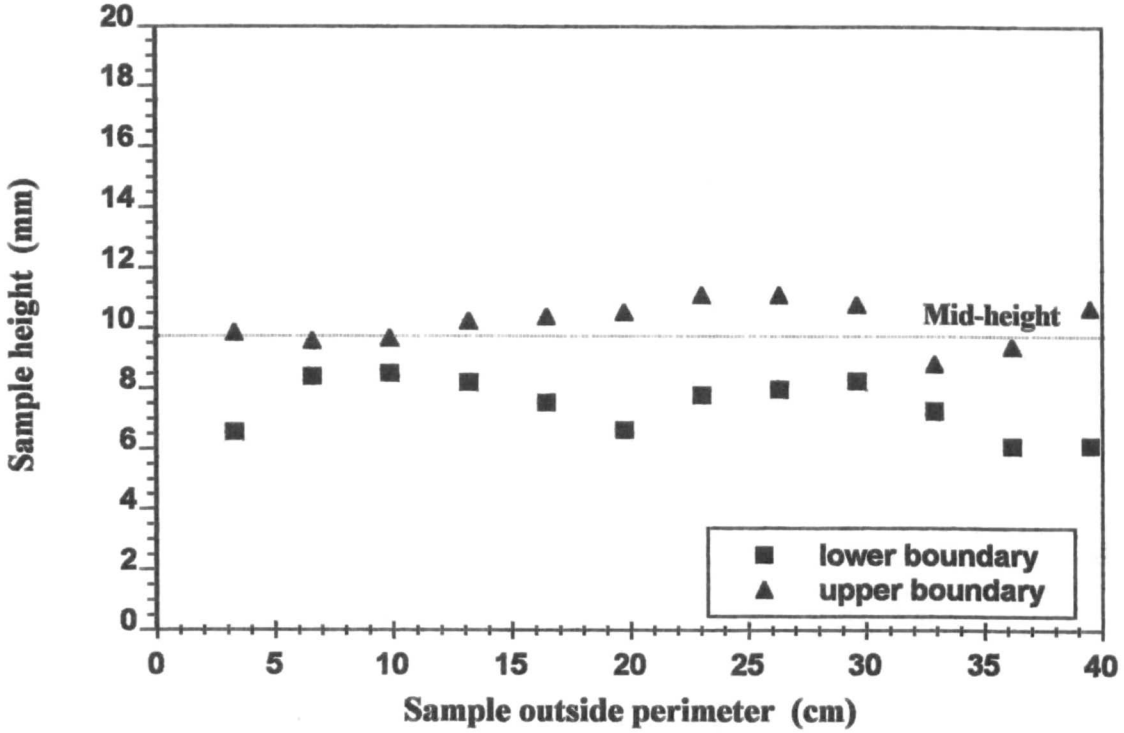


Figure 5.4.2.2 Shape and position of the shear zone of the normally consolidated, non-submerged, Cowden till sample

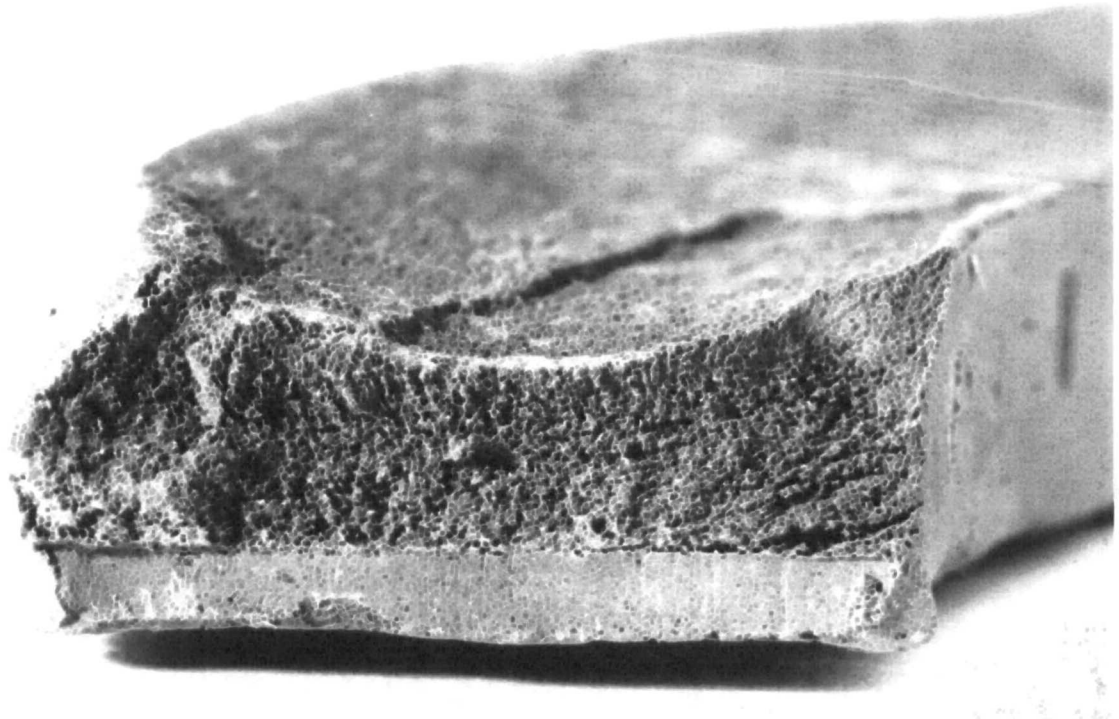


Fig. 5.4.2.3 a) Shear surface of normally consolidated, submerged Cowden till sample



Fig. 5.4.2.3 b) **Shear surface of normally consolidated, submerged Cowden till sample**

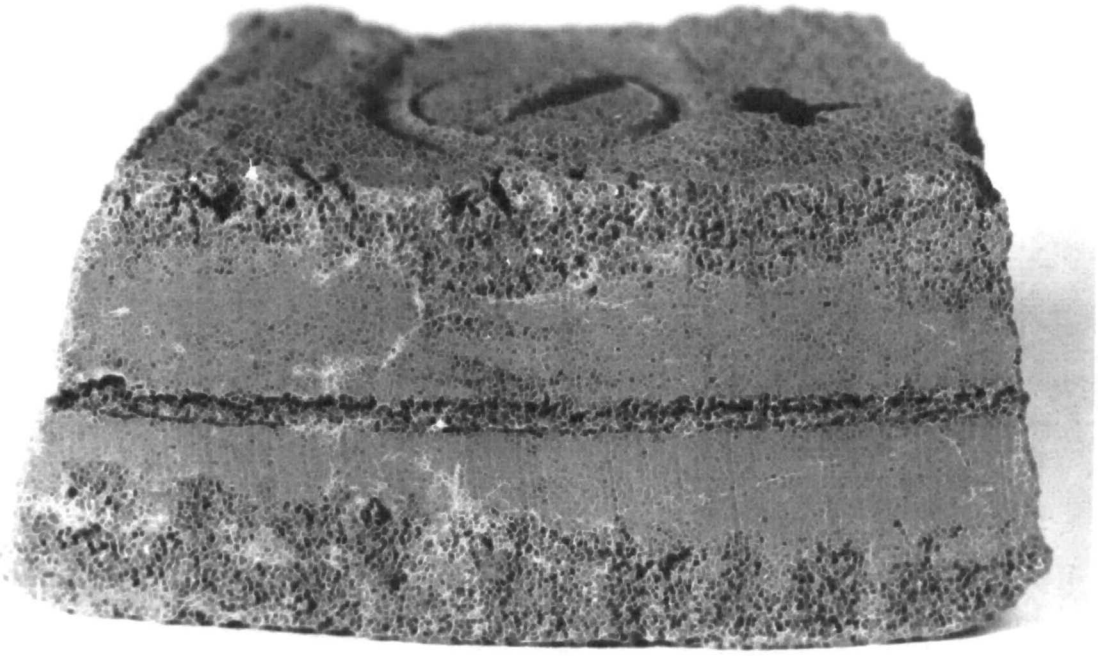


Fig. 5.4.2.3 c) **Shear surface of normally consolidated, submerged Cowden till sample**

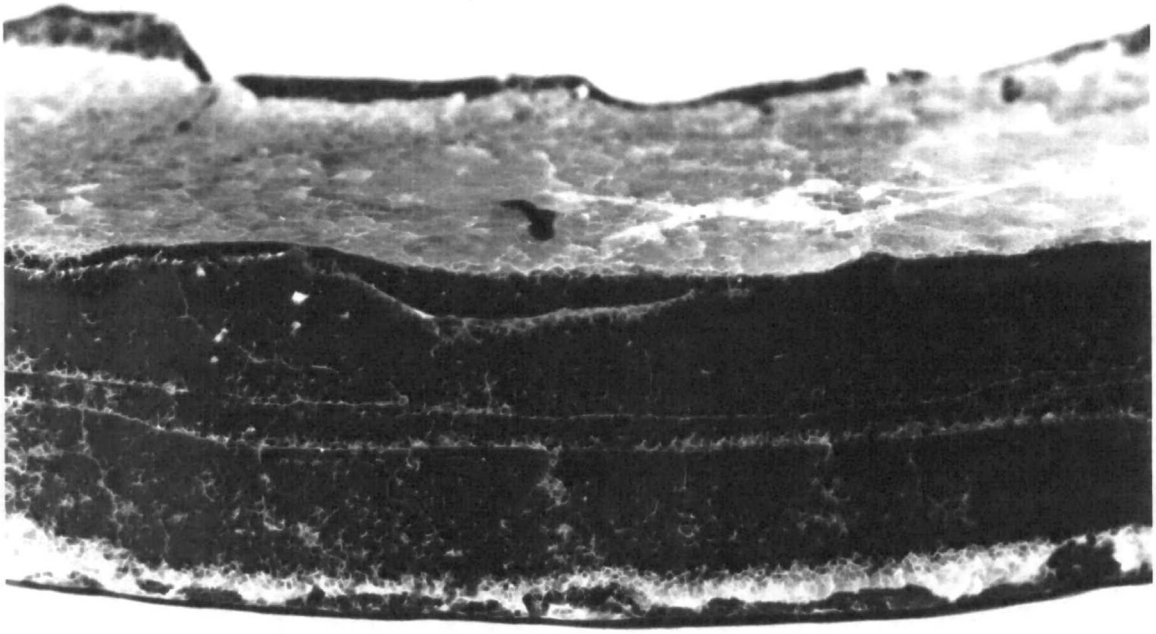


Fig. 5.4.2.4 a) **Shear surface of second normally consolidated, submerged Cowden till sample (3-stage test)**

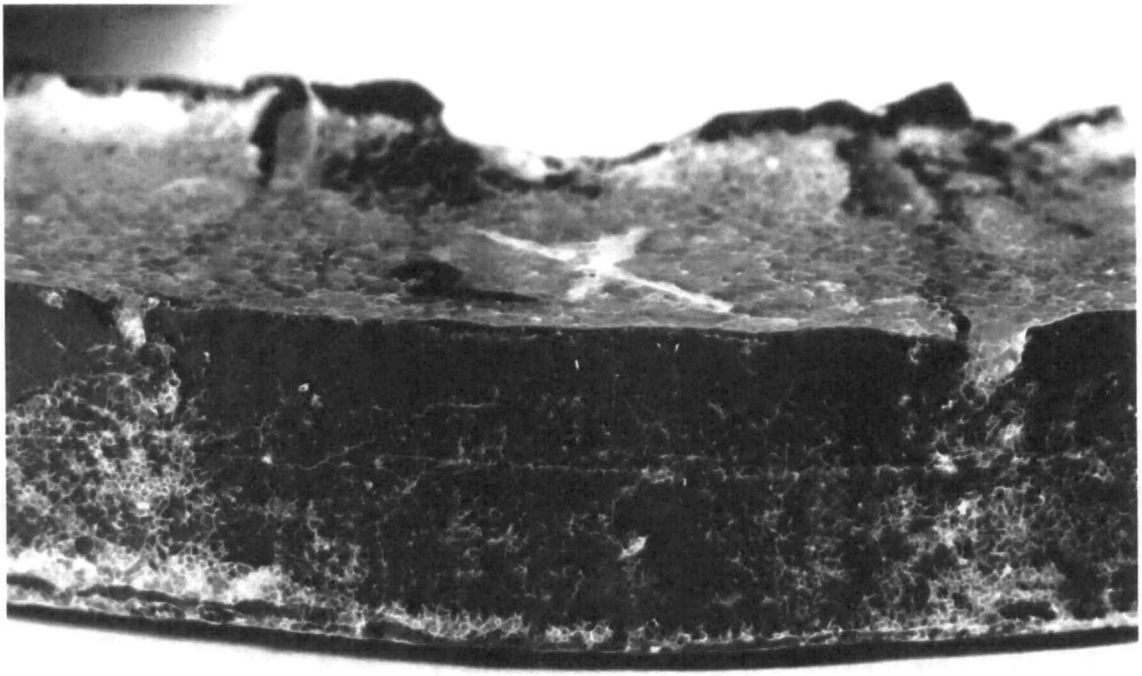


Fig. 5.4.2.4 b) **Shear surface of second normally consolidated, submerged Cowden till sample (3-stage test)**

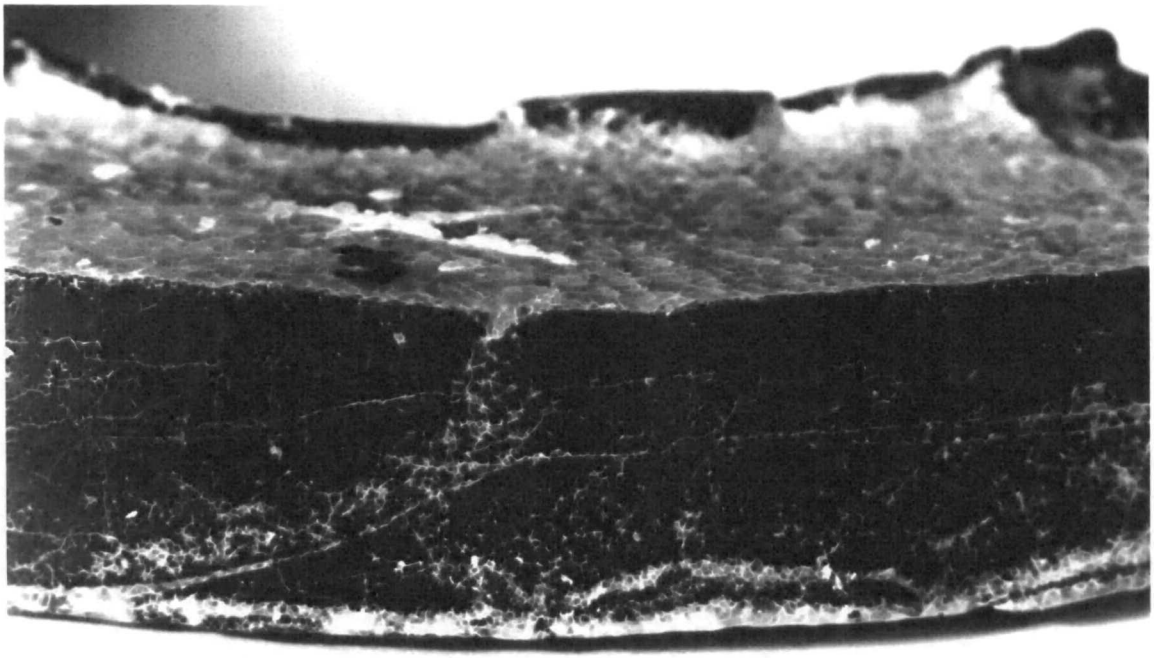


Fig. 5.4.2.4 c) **Shear surface of second normally consolidated, submerged Cowden till sample (3-stage test)**

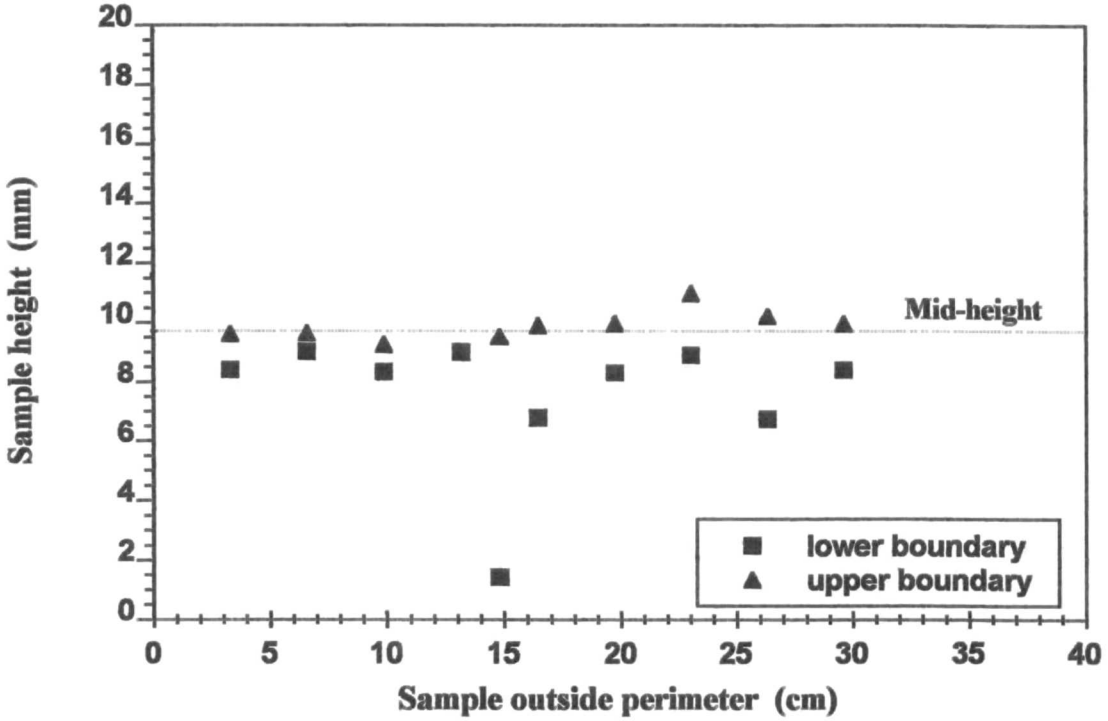


Figure 5.4.2.5 Shape and position of the second normally consolidated, submerged, Cowden till sample (3-stage test)



Fig. 5.4.2.6 a) **Shear surface of top part of over-consolidated, submerged Cowden till sample**

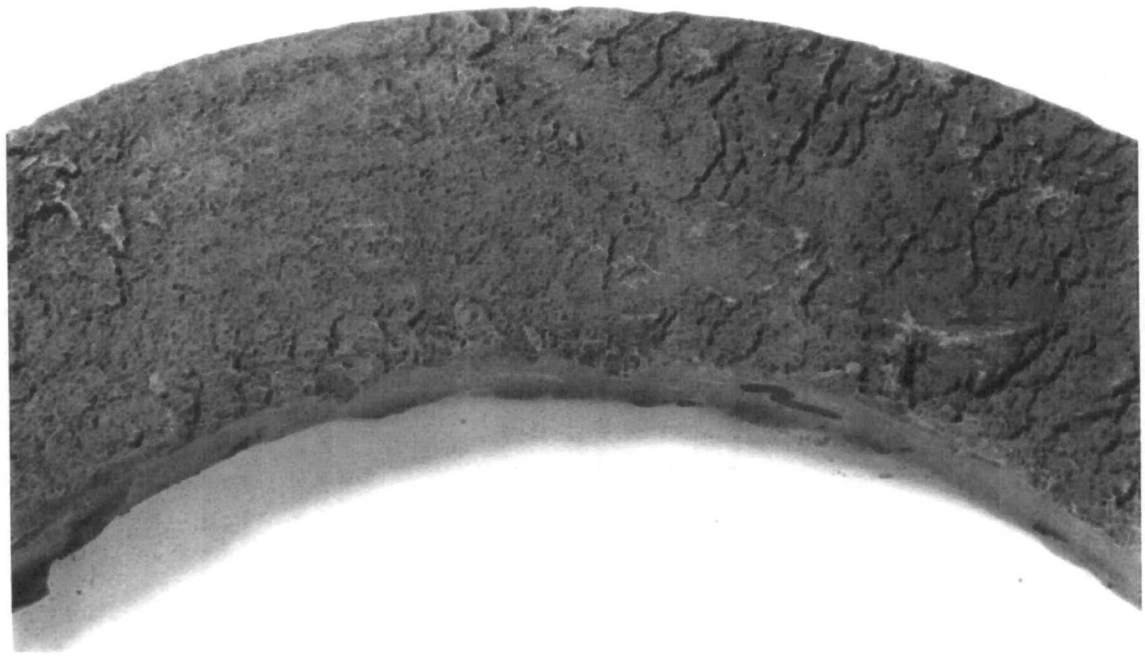


Fig. 5.4.2.6 b) **Shear surface of top part of over-consolidated, submerged Cowden till sample**

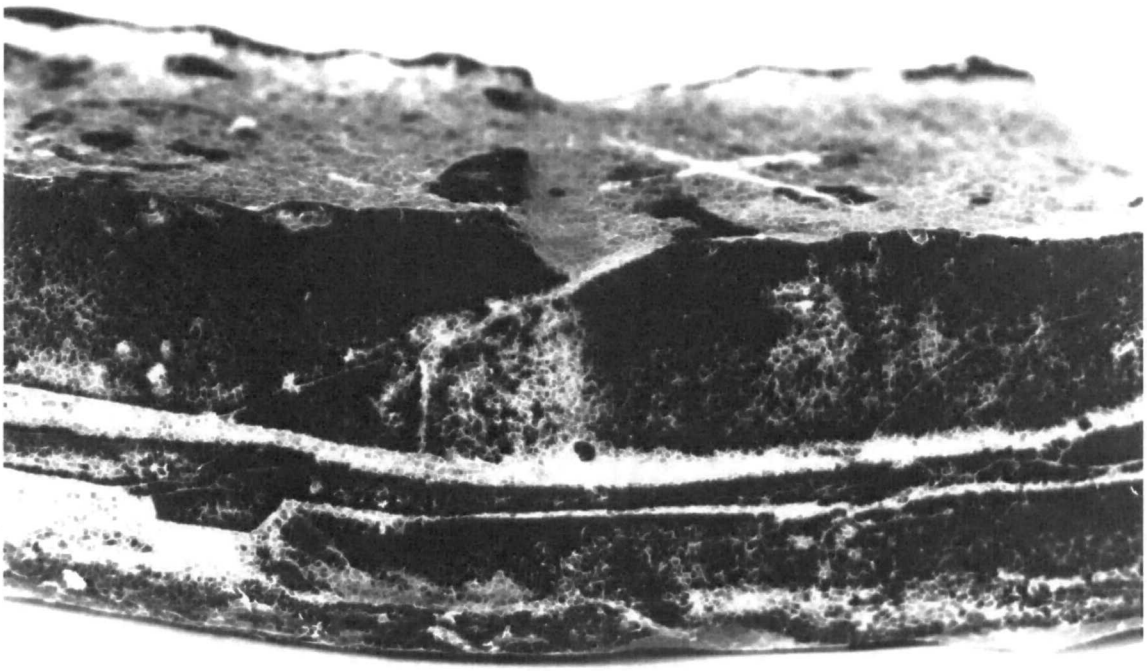


Fig. 5.6.1 a) Shear zone irregularities of over-consolidated, submerged brown London clay "intact" sample (submerged in Carbowax)

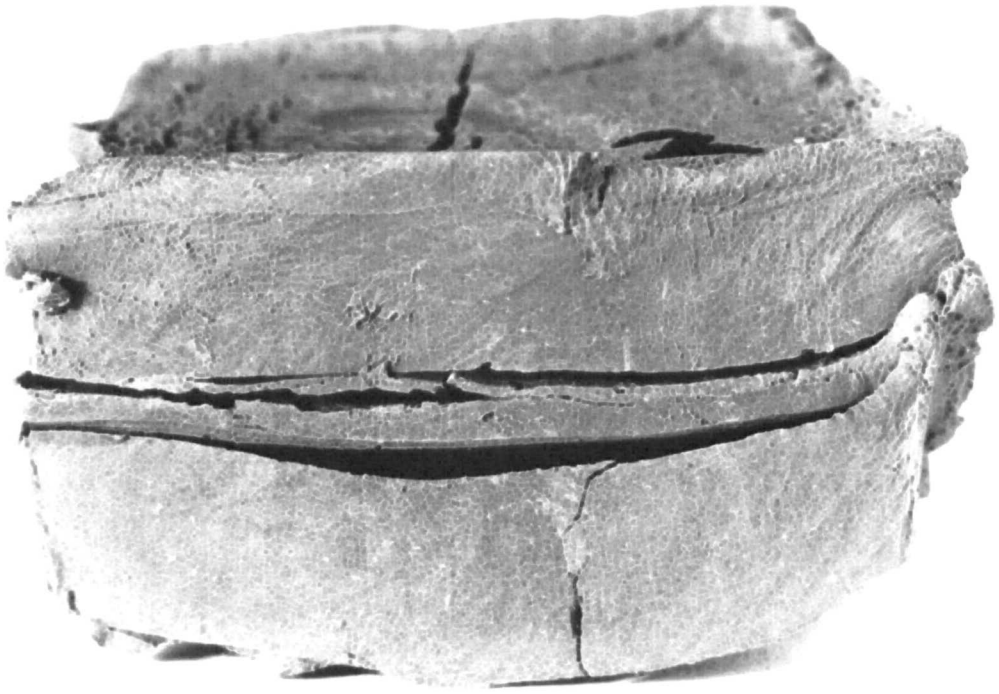


Fig. 5.6.1 b) Shear zone irregularities of over-consolidated, submerged brown London clay "intact" sample (air-dried)



Fig. 5.6.1 c) **Detail of shear surface of over-consolidated, submerged brown London clay "intact" sample**



Fig. 5.6.1 d) **Detail of shear surface of over-consolidated, submerged brown London clay "intact" sample**



Fig. 5.6.1 e) **Detail of shear surface of over-consolidated, submerged brown London clay "intact" sample**

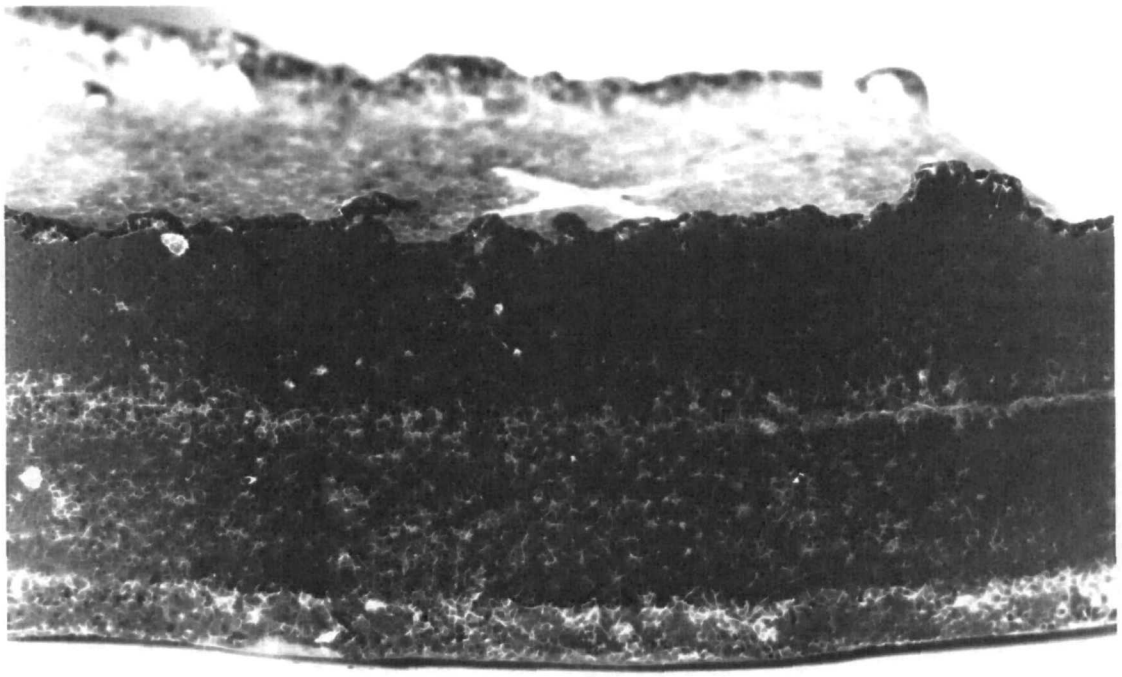


Fig. 5.6.2 a) **Shear zone of normally consolidated, submerged Cowden till "intact" sample (submerged in Carbowax)**

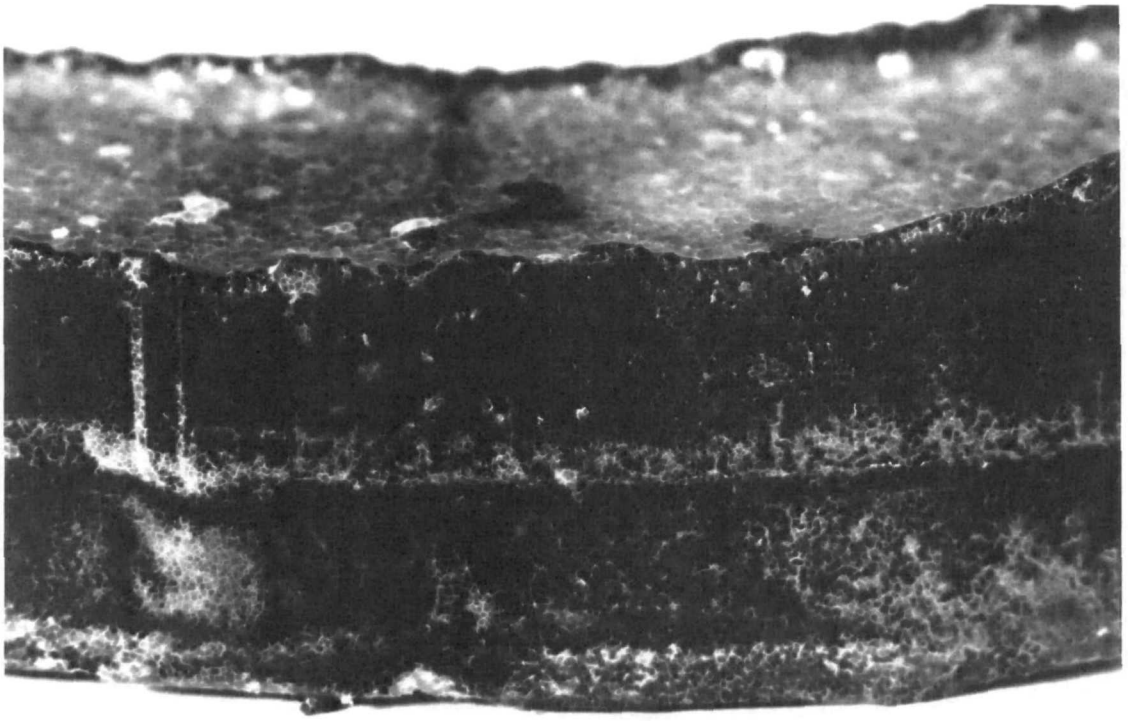


Fig. 5.6.2 b) Shear zone of normally consolidated, submerged Cowden till "intact" sample (submerged in Carbowax)

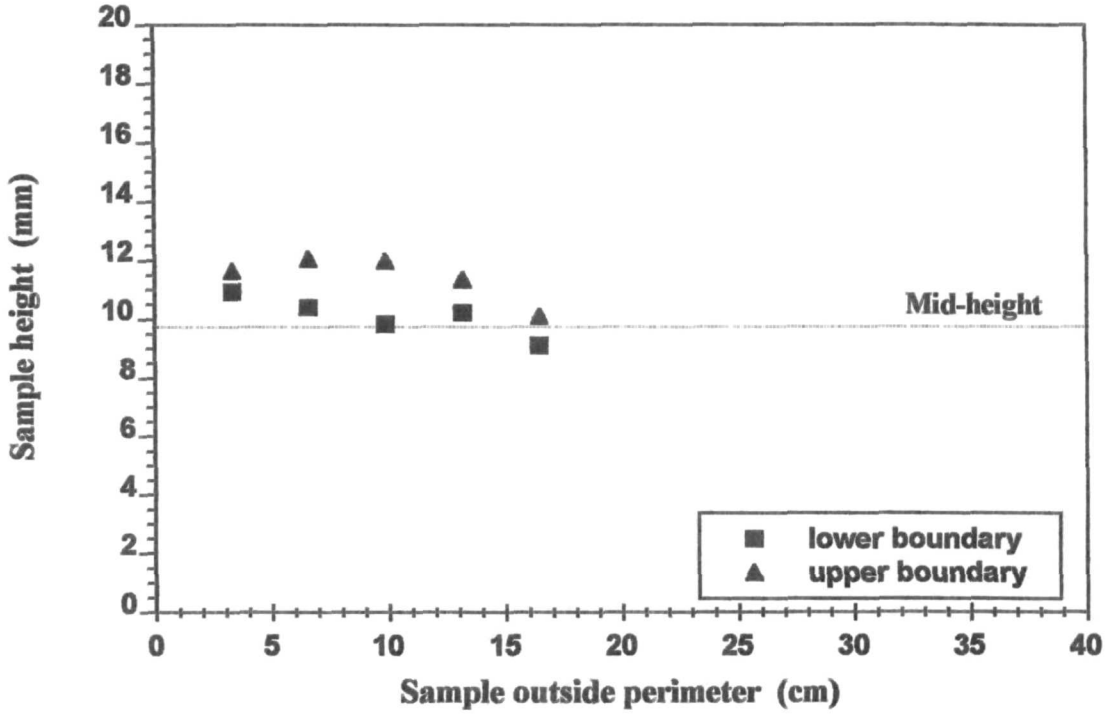


Figure 5.6.3 Shape and position of the shear zone of the normally consolidated, submerged, Cowden till intact sample

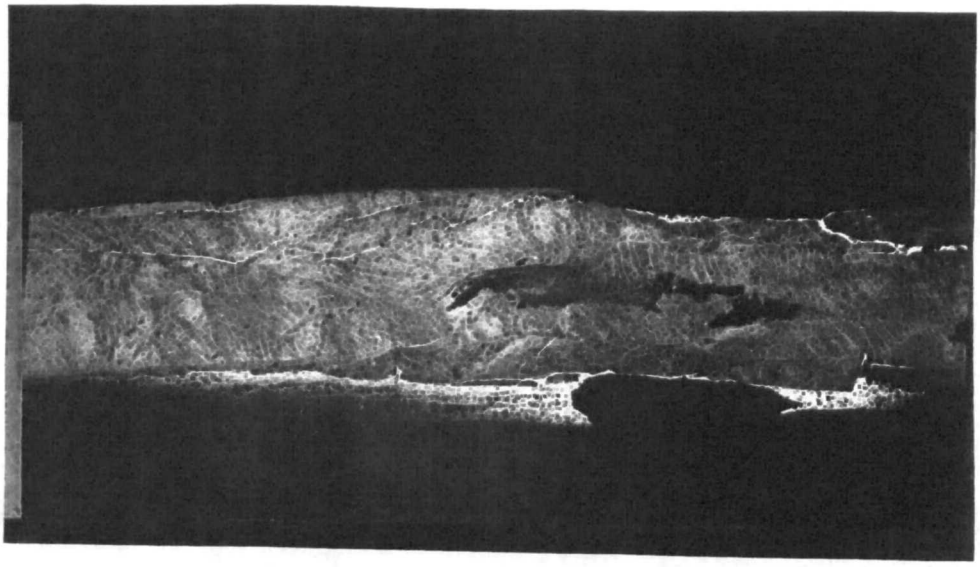


Fig. 5.TH1 Thin section from shear zone of normally consolidated, non-submerged, brown London clay sample (400 kPa)

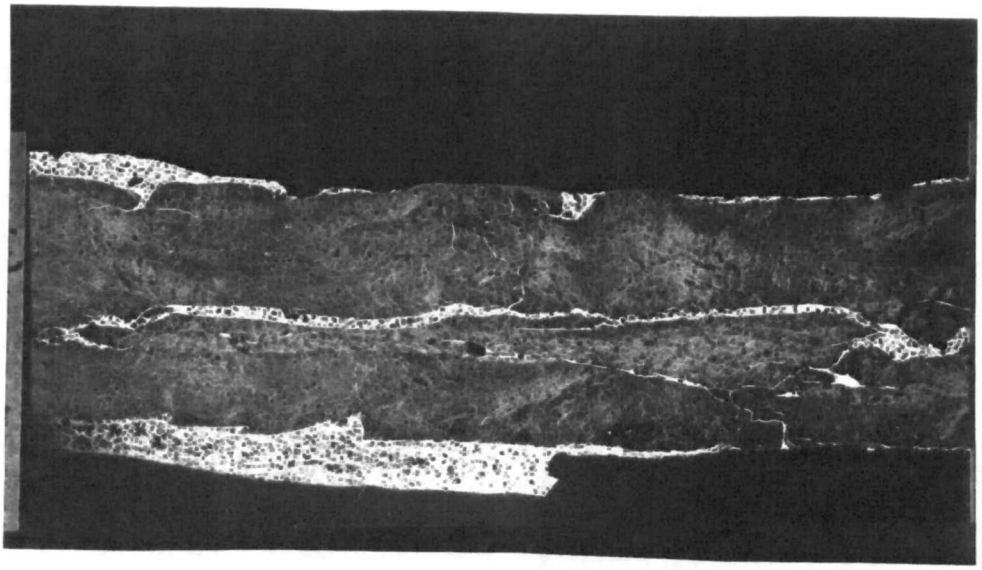


Fig. 5.TH2 Thin section from shear zone of normally consolidated, non-submerged, brown London clay sample (50 kPa)

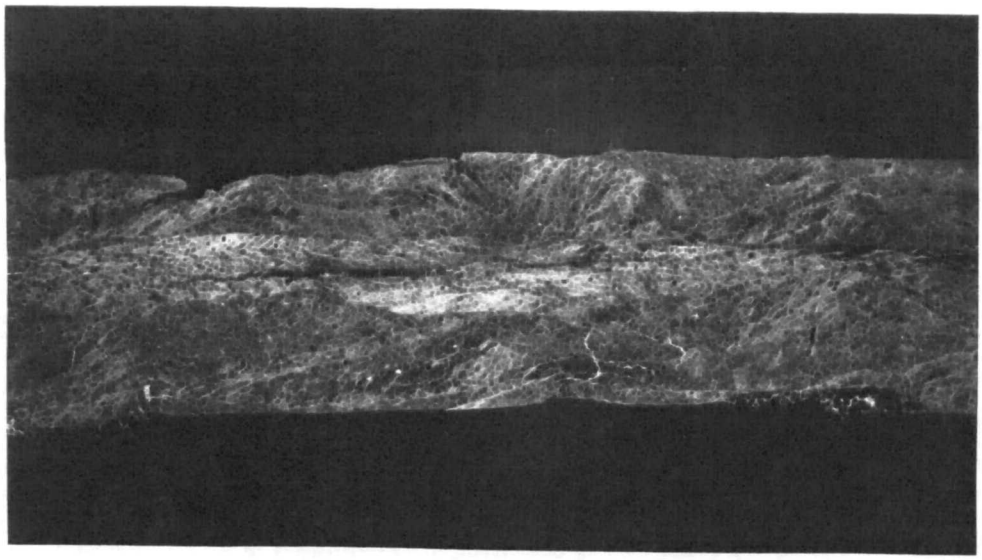


Fig. 5.TH3 Thin section from shear zone of normally consolidated, non-submerged, brown London clay sample (50 kPa)

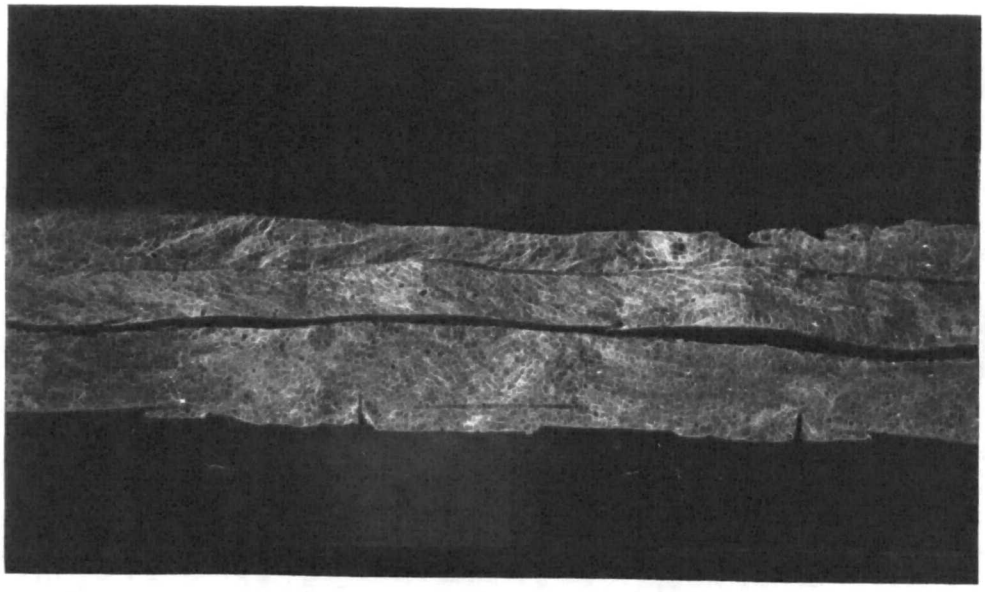


Fig. 5.TH4 Thin section from shear zone of normally consolidated, submerged, brown London clay sample (400 kPa)

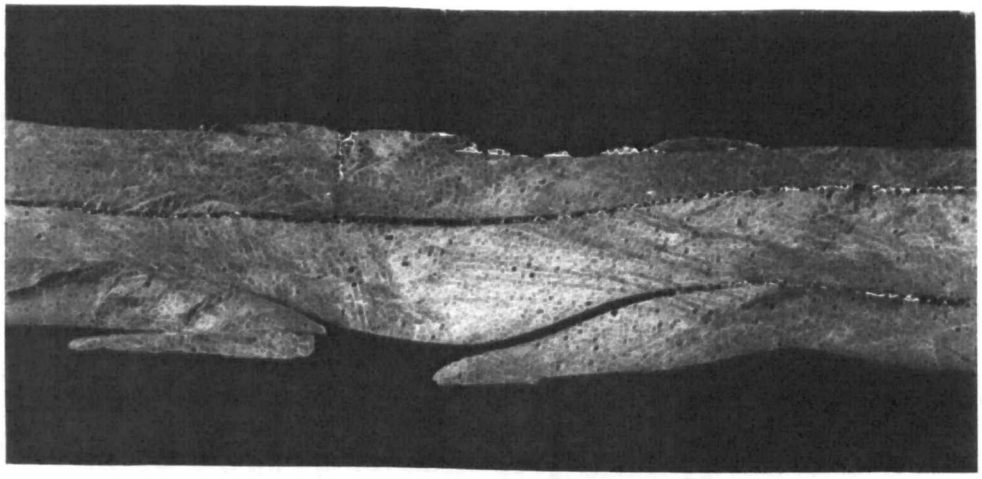


Fig. 5.TH5 Thin section from shear zone of over-consolidated, submerged, brown London clay sample (300 kPa)

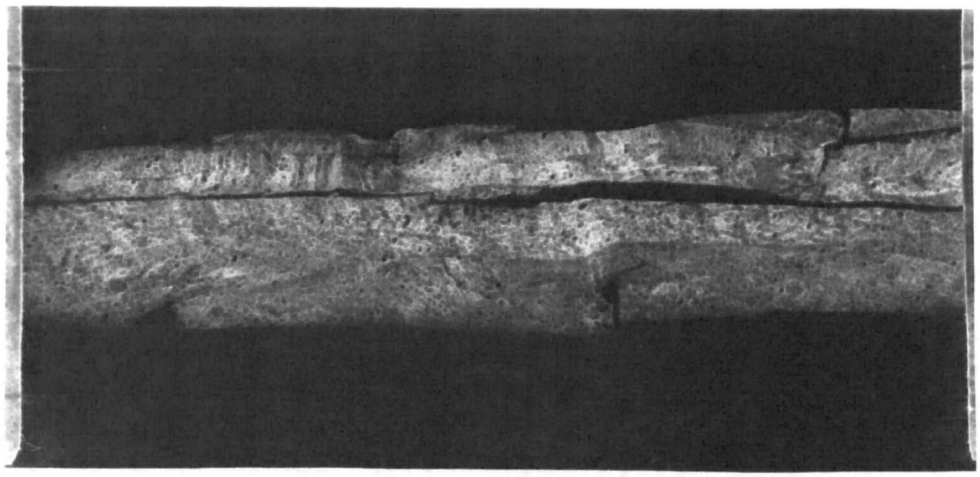


Fig. 5.TH6 Thin section from shear zone of normally consolidated, submerged, Cowden till sample (3-stage test)

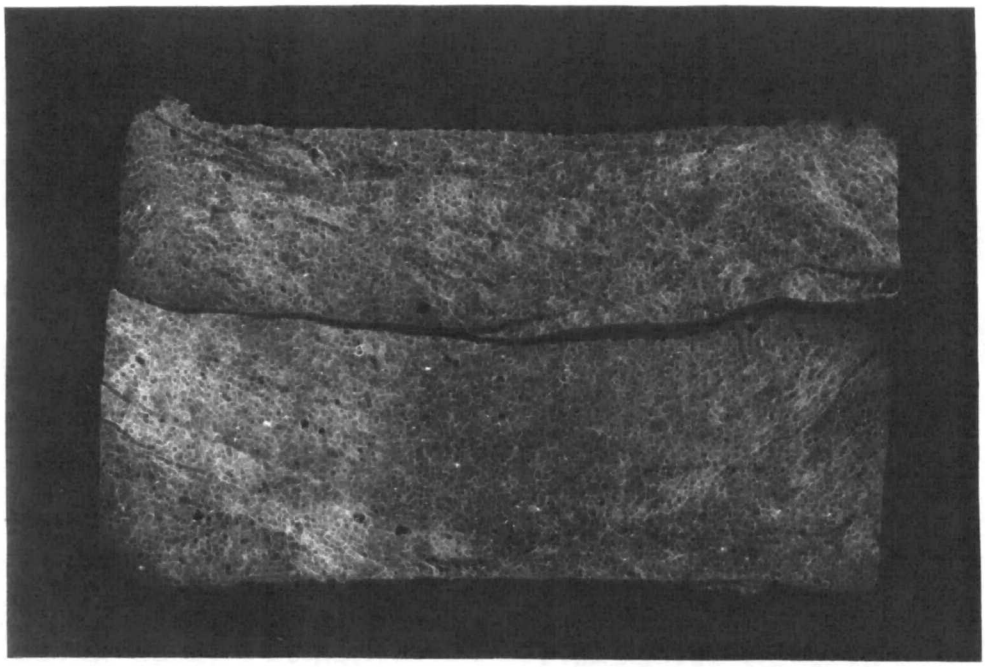


Fig. 5.TH7 Thin section from shear zone of normally consolidated, submerged, Cowden till sample (3-stage test)

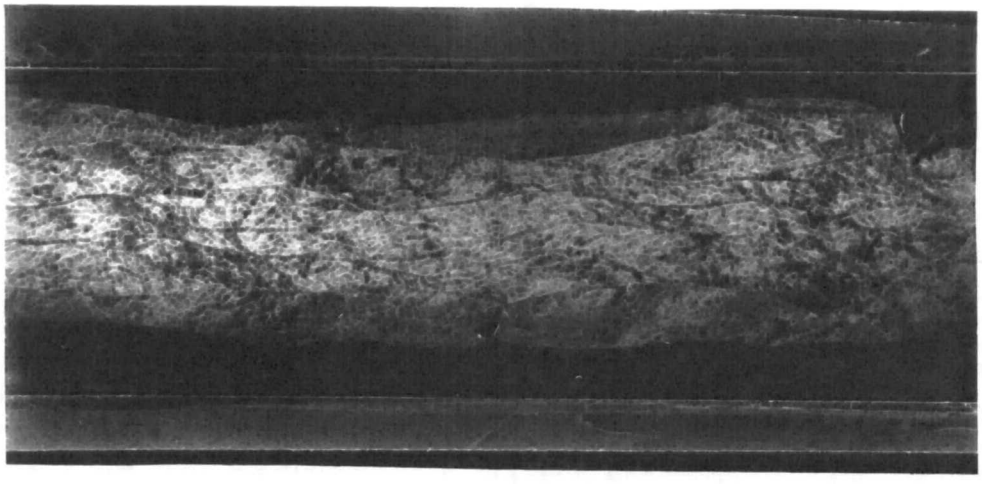


Fig. 5.TH8 Thin section from shear zone of kaolin sample

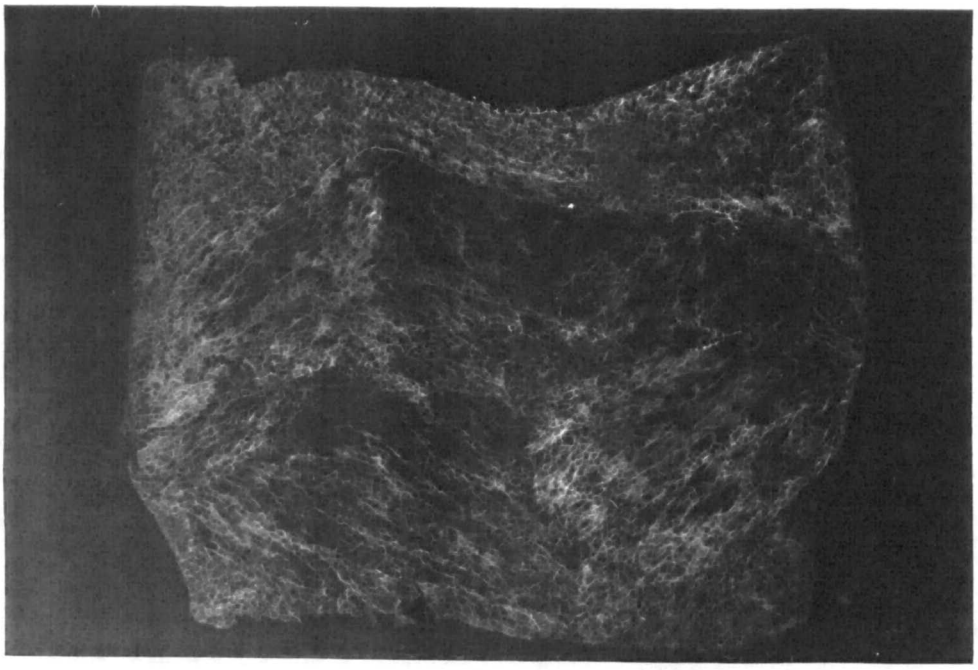


Fig. 5.TH9 Thin section from shear zone of kaolin sample

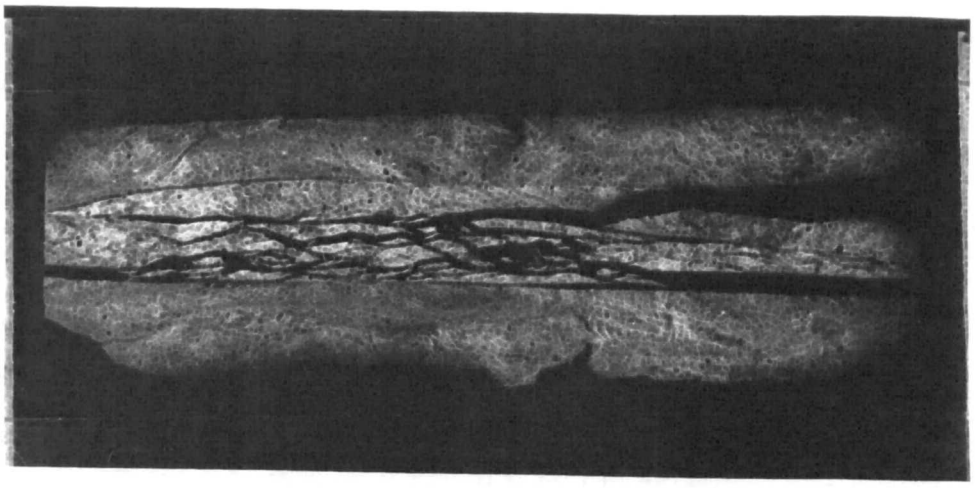


Fig. 5.TH10 Thin section from shear zone of over-consolidated, submerged brown London clay "intact" sample

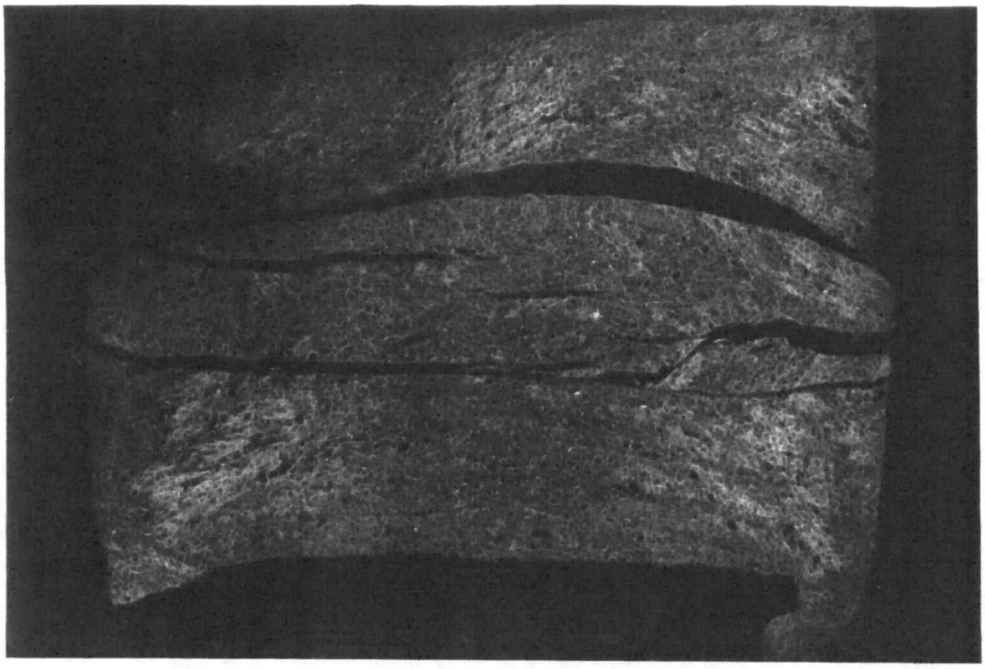


Fig. 5.TH11 Thin section from shear zone of over-consolidated, submerged brown London clay "intact" sample

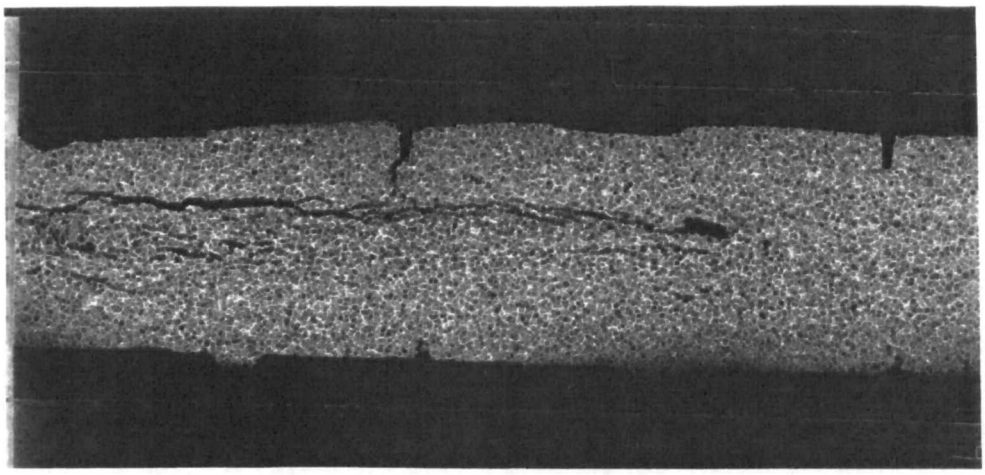


Fig. 5.TH12 Thin section from shear zone of normally consolidated, submerged Cowden till "intact" sample

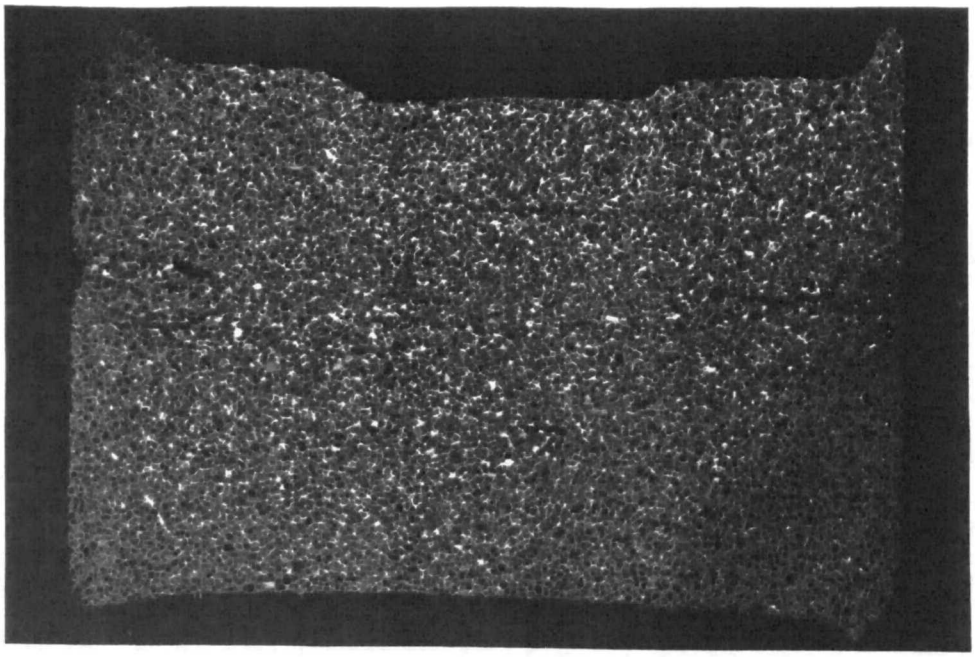


Fig. 5.TH13 Thin section from shear zone of normally consolidated, submerged Cowden till "intact" sample

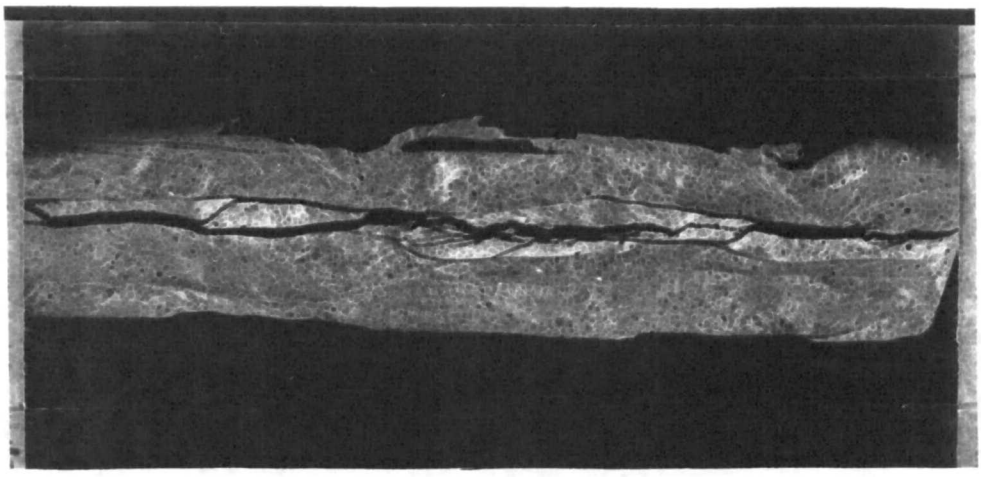


Fig. 5.TH14 Thin section from shear zone of over-consolidated, submerged brown London clay sample (reversed direction tests)

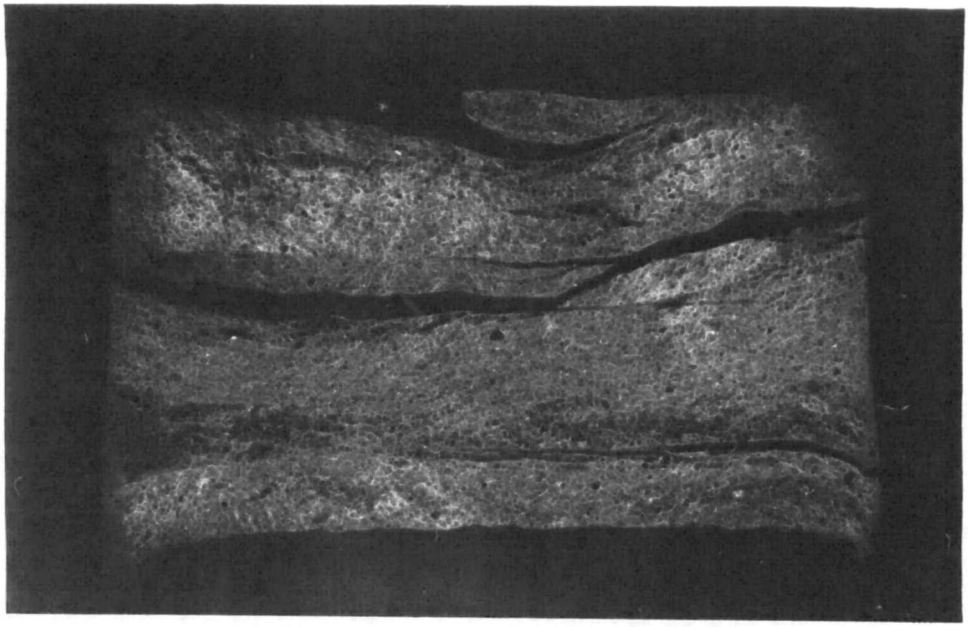


Fig. 5.TH15 Thin section from shear zone of over-consolidated, submerged brown London clay sample (reversed direction tests)

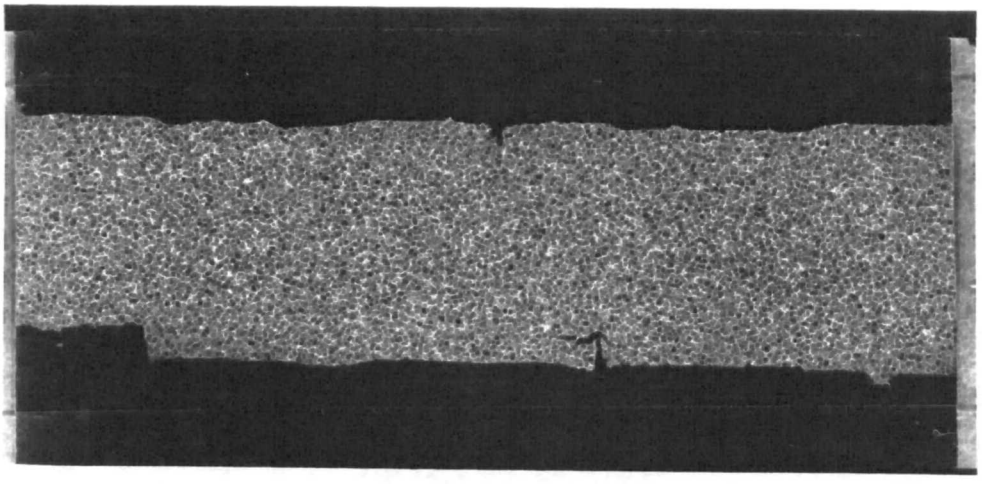


Fig. 5.TH16 Thin section from shear zone of normally consolidated, submerged Cowden till sample (reversed direction tests)

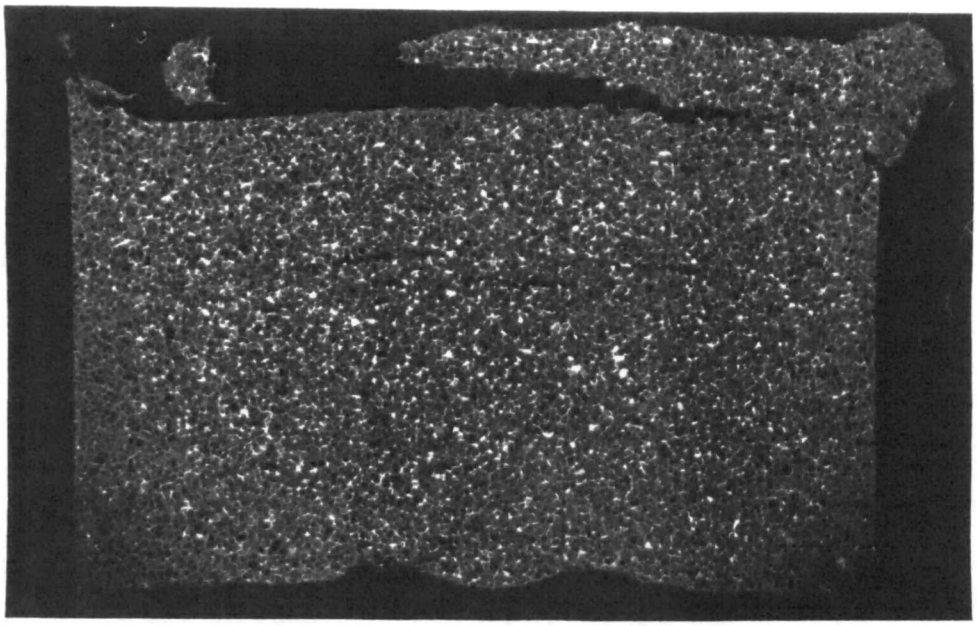


Fig. 5.TH17 Thin section from shear zone of normally consolidated, submerged Cowden till sample (reversed direction tests)

CHAPTER SIX

ANALYSIS AND DISCUSSION OF TEST RESULTS

6.1 Introduction

In this chapter the test results which have been summarised in Chapter 5 and are also presented in detail in Appendices IV-VI are divided into two main categories to facilitate analysis and discussion. Thus, in the first part of this chapter, the results from tests which have been conducted on plastic soils, *i.e.* brown London clay, Cowden till, and kaolin, will be analysed and discussed, while the second part will include the results from tests which have been conducted on non-plastic soils, *i.e.* pure silica, artificial sandy-silt, and sand. The analysis and discussion of the test results will concentrate mainly on the effects of the rate of displacement on the fast residual strength of soils. The concepts of the threshold strength and the post-threshold strength decrease are also discussed here.

6.2 Slow Residual Strength

The slow (drained) residual strength of soils has been investigated in detail by previous researchers since its fundamental significance in soil mechanics was postulated by Skempton (1964). In Chapter 2 the most important conclusions from previous investigations associated with the slow residual strength of soils were presented. It is well established that, while the slow residual strength is independent of the stress history, it depends on mineralogy, pore water chemistry, particle-size distribution and shape, normal stress, and, possibly, water content. In this investigation, the mobilisation of the slow residual strength is considered as the basis on which the rate effects were examined for different soil types. No further investigation was attempted on the factors which affect the slow residual strength.

The non-linearity of the slow residual strength envelope has been investigated by previous researchers. It has been concluded that increasing normal stress leads to a better orientation of the platy particles, a denser packing of the rotund and platy particles, a decrease in the granular void ratio, and an increase in the inter-locking between the massive grains. Therefore, for each soil type which has been tested in this investigation, the shape of its slow residual strength envelope [Figs. 6.2.1-6.2.4] should be viewed in the context of the particles which compose it.

For each soil type which was used in this investigation at least three identically prepared samples were tested at four normal stress levels. The slow residual strength of each sample measured at each normal stress level was plotted in order to form a strength envelope with all the available test results for each soil type [Figs. 6.2.1-6.2.4]. The average strength at each normal stress level was used to establish the slow residual strength envelope of each soil type. For consistency purposes, any reference which will be made, hereon, to the slow residual strength at any particular normal stress level should be interpreted as the "average" slow residual strength, as shown in these figures, and not necessarily as the strength which was actually measured during each test. All the analysis which will be presented in this chapter is associated with the "average" slow residual strength. Moreover, it must be noted that the soil strength in the calculation and analysis of results in this investigation is presented as the frictional angle of the soil. This may lead to high values of apparent soil strength at low normal stresses or when soils with angular particles are involved, but was adopted because it provides a comprehensible indication of the behaviour of soils.

The slow residual strength envelope of brown London clay [Fig. 6.2.1] appears, as expected for soils with platy particles, to be non-linear for normal stresses lower than 150 kPa. The slow residual strength decreases from 15.4° to 11.4° as the normal stress increases from 50 to 150 kPa. and remains, for all practical purposes, constant thereafter. In comparing the slow residual strengths of the normally consolidated, non-submerged sample with those of the normally consolidated, submerged sample, it becomes evident that the strengths of the non-submerged sample are consistently higher by 0.5° to 1.0°. This comes in agreement with previous findings (Cullen and Donald, 1971) which suggest that a decrease in water content may in fact lead to an increase in the measured slow residual strength of soils. Indeed, the water content of the non-submerged sample

was measured to be 2 % lower than that of the submerged sample [Table 5.2.1]. Furthermore, in comparing the slow residual strengths of the normally consolidated, submerged sample with those of the overconsolidated, submerged sample, it is suggested that the scatter associated with this set of data ought to be attributed to the testing procedure. The multistage testing technique may accelerate the production of results and insure an identical sample nature at all normal stress levels but the results from each slow shearing stage can be affected by the secondary shear surfaces and the non-planar shear zone which might have been developed during the preceding fast shearing stage. Nevertheless, the consistency of the slow residual strengths measured on brown London clay samples is considered, for all practical purposes, satisfactory.

The slow residual strength envelope of Cowden till [Fig. 6.2.2] also appears to be of a non-linear shape within the lower normal stress range. The slow residual strength decreases from 32.9° to 31.5° when the normal strength increases from 50 kPa to 100 kPa, and remains constant at 31.2° when the normal stress increases from 200 kPa to 400 kPa. This non-linearity is more pronounced with the results from the normally consolidated, non-submerged sample. Although the water content of the normally consolidated, non-submerged sample after testing is 3.5 % lower than that of the normally consolidated, submerged sample, the evidence that the slow residual strength increases with decreasing water content is not conclusive. The relatively high slow residual strength (35.4°) which was measured on the first test (50 kPa) conducted on the normally consolidated, non-submerged sample may be attributed to the fact that the ring shear apparatus has been often found by previous researchers (Lemos, 1986 and Tika 1989) not to perform reliably at very low normal stress levels. The slow residual strength values which are associated with tests on submerged samples show a decreased variation at low normal stresses and a satisfactory overall consistency.

In the case of the tests which were conducted on pure silica samples, the slow residual strengths did not exhibit the consistency and clear non-linearity [Fig. 6.2.3] that were observed in the two previous cases. The apparatus seamed to be affected by the nature of the soil, while the high slow residual strength which was exhibited by the soil prevented the continuation of the multistage testing procedure at high normal stress levels. Nevertheless, the slow residual strength appears to be decreasing from 47.3° to 46.5°, when the normal stress increases from 50 kPa to 100 kPa, and remains constant at

46.4° for normal stresses between 150 kPa and 300 kPa. The slow residual strength values from the normally consolidated, non-submerged sample appear to be lower than those from the submerged samples, do not exhibit a clear non-linearity, and remain nearly constant at all normal stresses. Furthermore, the slow residual strength values from the normally and overconsolidated, submerged samples are in close agreement at all normal stress levels. It is interesting to note that the water contents of the three samples after testing do not vary by more than 0.6 % [Table 5.2.1]. The wide scatter in the slow residual strengths measured at the lower normal stress levels may again be attributed to the inconsistency of the performance of the apparatus at such normal stress levels.

A more constant non-linear shape of the slow residual strength envelope was observed in the case of tests conducted on the artificial sandy-silt [Fig. 6.2.4]. The slow residual strengths decrease, in average, from 48.7° to 46.7° when the normal stress increases from 50 kPa to 100 kPa. Furthermore, the results indicate that the slow residual strength increases when the normal stress increases above 200 kPa. Considering the influence which the rotund sand particles have on the shearing resistance of these samples, the previous observation comes in agreement with the suggestion made by Lupini (1981) that the increasing normal stress will result in a denser packing of such particles and, therefore, in an increased strength. However, for the purposes of the analysis and discussion of these results and in order to avoid over-estimating the slow residual strength of the artificial sandy-silt, it will be considered to remain constant at an average value of 46.5°. Finally, it should be noted that while the water content of the normally consolidated samples decreased with testing, this was not the case for the overconsolidated sample [Table 5.2.1].

In the case of the tests which were conducted on medium-coarse, clean sand, the slow residual strength was not considered. It was assumed that shearing the same sample at increasing normal stresses leads to crashing of the rotund particles and continually changes the size distribution of the material within the shear zone. Therefore, it was concluded that comparing the slow residual strength of, essentially, different samples would not result in any meaningful conclusion and only the rate effects on these samples were examined.

6.3 Plastic Soils

6.3.1 Threshold Strength

The threshold strength has been defined (Chapter 2) as the soil strength which is required to initiate slippage or the strength which is associated with zero shear displacement. Moreover, it has been suggested that the threshold strength becomes noticeable for rates of displacement which exceed 10 mm/min. Tika (1989) presented evidence indicating that the threshold strength increases with decreasing normal stress and increasing rate of displacement, and decreases with increasing clay fraction.

The results from this investigation do not show systematic threshold strength variations with changes in normal stress and rate of displacement. The large scatter in the data associated with the threshold strength is in part attributed to apparatus effects and in part to the fact that, in order to achieve larger rates of displacement and large shear displacements during each fast shearing stage, the rate of displacement was increased rather suddenly and this might have influenced the results. It must be emphasised that the threshold strength of soils was not being examined systematically in this investigation, and so the testing procedure which was adopted in order to achieve the other targets may have affected the development of that strength.

Nevertheless, the results from tests on brown London clay suggest that the threshold strength decreases when the normal stress increases from 50 kPa to 100 kPa and does not change significantly at higher normal stresses [Fig. 6.3.1]. Furthermore, the threshold strength appears to increase when the rate of displacement increases up to 2.0 cm/sec and remain constant at higher rates of displacement [Fig. 6.3.2].

The results from tests on Cowden till also indicate, although there is some scatter in the data, that the threshold strength tends to decrease with increasing normal stress [Fig. 6.3.3]. Finally, it is evident that no clear conclusion about the relationship between the threshold strength and the rate of displacement can result from the available data [Fig. 6.3.4]. The considerable scatter which is associated with this set of data is thought to have been caused by the testing procedure associated with these tests. When the slow shearing stage of a test is terminated, the sample is unloaded in shear, the fast gear is engaged, and shearing at higher rates of displacement is restarted. The loading yoke which transmits the shear stress impacts suddenly on the load cells. It is speculated that

the initial shear stress measurement during each fast shearing stage, which is in fact associated with the threshold strength, may be affected to some extend by the sudden impact of the loading yoke on the load cells. Therefore, any quantitative analysis of the results from this investigation which are associated with the threshold strength of soils should be viewed with caution.

6.3.2 Post-threshold strength decrease

The post-threshold strength decrease is defined as the drop in strength which is observed immediately after the sample has reached its threshold strength and which normally occurs within the first 30 cm of the fast shearing stages. Insufficient evidence exists to suggest any correlation between the threshold or residual strengths and the post-threshold strength decrease, and it is impossible to predict whether the post-threshold strength decrease will be above or below the slow residual strength level for the normal stress and rate of displacement at which a sample is to be tested. However, the post-threshold strength decrease may be associated with the increase in sample height which has been observed during the early parts of the fast shearing stages.

It is speculated that the following three mechanisms may be responsible for the decrease in strength after it has reached its threshold level: dilation of the shearing matrix, development of a void between the rotating and stationary parts of the sample, and pore pressure increase. It is believed that the magnitude of the post-threshold strength decrease in each test may have been affected by the handicaps of the system in measuring the "absolute" threshold strength as has been discussed previously.

The post-threshold strength decrease which was measured in the tests conducted on brown London clay samples has been plotted against the ***immediate dilation*** [Fig. 6.3.5] which is defined as the increase in sample height observed immediately after the sample has reached its threshold strength. It is not clear from the existing data whether this increase in sample height ought to be attributed to the dilation of the shearing matrix or the increase in the void between the rotating and stationary parts of the sample and, therefore both mechanisms have to be considered. Taking into account the scatter in the data which, as it has been mentioned previously, is partly attributed to the measurements of the threshold strength, the post-threshold strength decrease increases with increasing

immediate dilation. It is suggested from the available data that the measured sample strength decreases, on average, by 0° to 15° as the sample height increases from 0 to 0.5 mm during the initial part of the fast shearing stages. Further increase in the sample height (between 0.5 and 1.0 mm) appears to have a less significant effect on the post-threshold strength decrease for the brown London clay samples.

In the case of the Cowden till samples, the post-threshold strength decrease [Fig. 6.3.6] appears to follow the same general trend established from the brown London clay samples. The post-threshold strength decrease is greater with increasing immediate dilation, though it appears that immediate dilation above 1.5 mm has no significant effect on the strength. The existing evidence suggests that the post-threshold strength decrease for the Cowden till samples typically varies from 0° to 20° as the immediate dilation increases from 0 to 1.5 mm. Further increases of the immediate dilation between 1.5 and 3.0 mm have no significant effect on the post-threshold strength decrease, which remains around 20°.

The three mechanisms which have been considered to be associated with the post-threshold strength decrease can only be evaluated indirectly because the existing equipment does not allow the direct measurement of the volume changes or the pore pressure increases at the shearing surface. An increase in pore pressure during the initial part of the fast shearing stages has been observed by Yagi *et al* (1991). They suggested that the pore pressure may increase considerably during the first 10 cm of a fast shearing stage [Fig. 2.4.9]. This pore pressure decreases as shearing continues and appears to dissipate within the first 40 cm of shear displacement. Although it is not possible to measure the pore pressure which is generated during fast shearing in the IC/NGI ring shear apparatus, it seems that any pore pressure increase must be affected by the immediate dilation of the sample. Nevertheless, it is possible that some increase in pore pressure takes place immediately after the sudden application of the fast rates of displacement and, therefore, this factor may be partly responsible for the post-threshold strength decrease.

The increase in sample height which has been observed at the early parts of each fast shearing stage and was defined as the immediate dilation of the sample may be due to either dilation of the shearing matrix or increase in the void between the rotating (lower) and stationary (upper) parts of the sample. Although both mechanisms oppose

the increase in pore pressures, it may be possible that the pore pressure increase actually takes place before the sample height begins to increase. Therefore, the two opposing mechanisms may be operating at different stages during fast shearing.

It seems unlikely that the increase in sample height can be attributed entirely to dilation of the shearing matrix. The immediate dilation was often measured to be greater than 0.5 mm, and when one considers that the thickness of the shear zone is usually less than 1.0 mm, it becomes difficult to conceive how the increase in sample height is induced entirely by volume changes within the shearing matrix. The most plausible explanation for the post-threshold strength decrease and the immediate dilation of the sample must be the increase in the void (or gap) between the rotating and stationary parts of the sample. It has been shown in Chapter 5 that several secondary shear surfaces develop during shearing and also that the shape of the principal shear zone in usually non-planar or, even, undulating. During the slow shearing stage of each test, the strength over the whole principal shear zone is mobilised but when the fast rates of displacement are suddenly applied the secondary shear zones may be activated and the irregularities of the principal shear zone may give rise to voids between the two parts of the sample. Thus, the sample height appears to increase only a part of the shear strength is mobilised and the measured strength decreases. As the test progresses the shear zone may settle to a nearly planar shape or develop an undulating or non-planar one. Those mechanisms will affect the measured strength in opposite manner as it will be described later in this chapter.

It may be considered that all three mechanisms which have been presented above operate at different parts during each fast shearing stage. However, it would have been impossible to address the mater reliably using the existing laboratory infrastructure. The modifications which, in the opinion of this researcher, need to be considered before undertaking such task will be presented in Chapter 7.

6.3.3 Rate-Dependent Characteristics

The rate-dependent characteristics of soils have been described by previous researchers as ***viscous*** or ***positive rate*** effects. A series of tests which were conducted by means of a *cone and plate viscometer* on a clayey siltstone, kaolin, and a glycerol-silicon

oil mixture (Tika, 1989) gave some indication of the rate dependency of these materials. In the case of the clayey siltstone [Fig. 6.3.7], the rate dependency seemed to be affected by the water content of the samples whose shear stress appeared to be increasing over the lower band of the rates of displacement under which they were tested. However, with the exception of the sample with the lower water content, when the rate of displacement was increased further, the measured shear stress decreased. In the case of the tests which were conducted on kaolin samples [Fig. 6.3.8], the shear stress of the sample with the lower water content increased by approximately 20 % when the rate of displacement was increased by a thirty-fold, while the wetter samples did not seem to respond to similar increases in the rate of displacement. Finally, the viscometer tests on the glycerol-silicon oil mixture [Fig. 6.3.9] indicated a clear decrease in viscosity within a wide range of increasing rates of displacement, while the shear stress appeared to be increasing during the higher band of that range.

Previous investigations of the effect of increasing rate of displacement on the residual strength of soils have resulted in a rather complicated picture of the behaviour of soils upon fast shearing. Tika (1989) concluded that there are five types of this behaviour which are controlled by the shear mode of each soil during slow shearing and the nature of each soil. However, it is believed that these investigations have been impeded by the shortcomings of the IC/NGI ring shear apparatus which have been discussed in Chapter 4. The modifications of the original design of the apparatus which have been implemented for the purposes of this investigation enable the operator to sustain high rates of displacement for a period sufficient to allow the measured strength to stabilise and the soil to reach a steady condition. The steady sample strength measured during the fast stages of each test will be referred to, hereon, as the ***fast residual strength*** of the sample and the figures which are presented in this chapter, and are associated with this strength, have been prepared by selecting points along the graph of each fast shearing stage which are considered to represent a steady measured strength for each corresponding level of rate of displacement. The scatter that may be associated with these figures must be attributed to the non-uniformities of the shear zone which arise during the large shear displacements which resulted from each test. In each figure, the average slow residual strength which corresponds to each sample at a particular normal stress, as it has been discussed earlier in this chapter, is also presented as a basis for

comparison. Finally, in discussing the results from each test, references will be made to the shape of the shear zone because it is thought that it has a considerable influence on the behaviour of each sample.

The results from the normally consolidated, submerged and non-submerged samples (Tests IV.1-9) and the second overconsolidated, submerged sample (Tests VI.1-2) of brown London clay indicate that the fast residual strength increases with increasing rate of displacement [Figs. 6.3.10-6.3.20]. Apart from the test on the normally consolidated, submerged sample when it was tested under the normal stress of 100 kPa (Tests IV.8) [Fig. 6.3.17], all tests indicated that the fast residual strength remains above the corresponding slow residual strength. Furthermore, there is evidence to suggest that the fast residual strength reaches a maximum value when the soil is sheared at rates of displacement around 1.5 and 2.0 cm/sec [Figs. 6.3.12-16 and 6.3.18-20]. The scatter which is associated with some of these figures is mainly attributed to the fact that the rate of displacement was sustained at a particular level for a considerable period and it was varied through the same level at several points during the course of each fast shearing stage. Therefore, the non-uniformities of the shear zone along the periphery of each sample have resulted in variations in the shear stress measurements which corresponded to one particular rate of displacement. Nevertheless, the aforementioned scatter does not appear to affect the general picture which is associated with the behaviour of this soil during fast shearing. However, two tests deserve individual attention.

The first is the test which was conducted on the normally consolidated, non-submerged sample under the normal stress of 50 kPa (Test IV.1). The scatter in the associated data [Fig. 6.3.10], which is not considered acceptable, has to be attributed primarily to the poor performance of the apparatus under low normal stresses and secondarily to the increased non-uniformities of the shear zone of the sample [Figs. 5.4.1.1, 5.4.1.2, and IV.1.2]. The shear zone has an undulating shape with an initial amplitude of 0.62 mm which decreased to 0.13 mm as the test progressed [Table 6.1]. Furthermore, at one point along the periphery of the sample the shear zone increased substantially in thickness, thus leading to periodic increases of the measured shear stress which also might have been affected by contamination of the confining rings, although this seems unlikely at this early part of a multistage test.

The second is the test which was conducted on the normally consolidated, submerged sample under the normal stress of 100 kPa (Test IV.8). It appears from the results [Fig. IV.8.2] that an undulating shear surface was formed at the early stages of this test with an initial amplitude of 0.3 mm which decreased progressively to 0.1 mm [Table 6.1] before it became nearly planar. Supporting evidence is also given by the shape and position of the shear zone of that same sample after it was tested under the higher normal stress of 200 kPa [Fig. 5.4.1.3, 5.4.1.4]. Although the fast residual strength of this sample when it was tested under the normal stress of 100 kPa stabilised below the slow residual strength, it appeared to increase with increasing rates of displacement. This behaviour does not follow the general trend and it must be viewed as the product of two opposing mechanisms, the undulating shape of the shear zone, which will be discussed in detail later in this chapter, and the rate dependent characteristics of the soil.

In conclusion, the non-submerged sample exhibited a positive rate effect, *i.e.* the fast residual strength was higher than the slow residual strength, and its fast residual strength appeared to be rate dependent although its shear zone was not always planar. The normally and overconsolidated, submerged samples which exhibited a positive rate effect had a nearly planar shear zone and their fast residual strength was rate dependent until it reached a maximum value when the rate was around 1.5 and 2.0 cm/sec.

Similar behaviour was observed in the tests which were conducted on the normally consolidated, non-submerged sample (Tests IV.13-16) and the first two tests on the overconsolidated, submerged Cowden till sample (Tests IV.21-22) [Figs. 6.3.21-26]. The normally consolidated, non-submerged sample exhibited clear positive rate effects and its fast residual strength was consistently higher that its slow residual strength and appeared to be rate dependent. The shear zone of the sample was undulating [Figs. 5.4.2.1 and 5.4.2.2] with an initial amplitude of 0.14 to 0.12 mm which decreased to 0.03 and 0.01 mm as the tests progressed [Table 6.2], however, this did not affect the behaviour of the soil during the fast shearing stages. Furthermore, when the rate of displacement was increased above 1.0 cm/sec, the fast residual strength remained around a constant maximum level. The scatter in the data which is associated with the first test conducted on this sample (Test IV.13) [Fig. 6.3.21] is attributed to the apparatus effects,

while the scatter in the data which is associated with the remaining tests [Figs. 6.3.22-24] is attributed to the non-uniformities of the shear zone.

A positive rate effect was again observed in the first two tests which were conducted on the overconsolidated, submerged sample [Figs. 6.3.25-26]. The evidence concerning the shape of the shear zone of this sample from the sample height curves of each test [Figs. IV.21.2, IV.22.2] indicate that, although it appears to be undulating, the maximum amplitude of these undulations is 0.09 mm and 0.05 mm for the tests under the normal stresses of 100 kPa and 200 kPa, respectively [Table 6.2]. Therefore, it seems unlikely that these undulations should have any effect on the behaviour of the soil during the fast stages of shearing.

Similar behaviour was observed when the kaolin sample was tested, normally and overconsolidated, submerged, under the normal stress of 200 kPa (Tests VI.13-14). The fast residual strength remained consistently above the slow residual strength, appeared to be rate dependent, and reached a maximum value when the sample was being sheared at rates of displacement around 1.2 cm/sec while the shear zone of the sample was not undulating [Figs. VI.13.2, VI.14.2].

The results from all the tests which were conducted on brown London clay and Cowden till samples and have shown a positive rate effect have been summarised in Figures 6.3.27-6.3.30. It is indicated from these results that the fast residual strength decreases with increasing normal stress. The effect of the increase in the rate of displacement, within a certain range (0.2-2.0 cm/sec), on the fast residual strength also decreases with increasing normal stress. Therefore, it is concluded that fast residual strength of plastic soils becomes less rate dependent as the normal stress increases. Furthermore, there is enough evidence to suggest that the fast residual strength of plastic soils reaches a maximum value when the rate of displacement is around 1.5 cm/sec. The same observations and conclusions that were made in relation to the fast residual strength of plastic soils hold true for the positive rate effect when this is defined as the difference between the fast and the slow residual strengths and applies only in cases where the first is greater than the second [Figs. 6.3.31-34].

6.3.4 Pore-Pressure Effects

The decrease of the measured strength below its slow residual value upon fast shearing has been defined by previous researchers (Lemos, 1986, and Tika, 1989) as the ***negative rate effect*** and it was associated with the nature of the soil and its shear mode upon slow shearing. However, in the light of the present work it seems unlikely that previous tests conducted with the IC/NGI ring shear apparatus in its original form would have resulted in an unambiguous measurement of the fast residual strength of soils because fast rates of displacement could not be sustained for a sufficient duration. Thus, it appears that the decrease in measured strength which was observed in tests which have been conducted in the past by means of this apparatus should be associated with the *post-threshold strength decrease* described earlier in this chapter. A more comprehensive picture of the negative rate effects which are associated with the fast residual strength of soils arises from the test results which have been presented in Chapter 5.

The first indications of a genuine negative rate effect were observed in the tests which were conducted on the first overconsolidated, submerged brown London clay sample (Tests IV.10-12). The fast residual strength stabilised within a range below the slow residual strength and it was not affected significantly by any further changes of the rate of displacement. Meanwhile, an undulating shear zone appeared to have formed with an initial amplitude of 0.6 and 0.3 mm which decreased to 0.18 and 0.08 mm, as the tests under the normal stresses of 100 and 200 kPa, respectively, progressed [Figs. IV.10.2, IV.11.2 and Table 6.1]. It is important to note that, during the latter part of the test which was conducted under the normal stress of 200 kPa, the shear zone appeared to become more planar and this was associated with a progressive increase in the fast residual strength. When the same sample was tested under the higher normal stress of 300 kPa, the measured shear stress gave clear evidence of the existence of an undulating shear zone but this cannot be detected on the sample height curve, as it has usually been the case, because of the large scale of that curve [Fig. IV.12.2].

However, when the sample was removed from the confining rings and the size and position of its shear zone on the outer face of the sample were measured, there were clear indications of the existence of an undulating shape [Fig. 5.4.1.6]. Moreover, it was observed that the shearing soil matrix had a much softer texture and, apparently, greater

porosity than the rest of the sample, and that secondary shear surfaces appeared to have developed adjacent to the principal shear zone at various points, thus introducing non-uniformities in the shearing mechanism, and allowing voids between the shear zone and the rest of the sample to develop [Fig. 5.TH5].

A second test on an identically prepared brown London clay sample was conducted using the same testing procedure. The idea was to investigate the negative rate effect by duplicating the conditions under which it was originally observed. However, Tests VI.1-2 which were conducted on this second sample did not exhibit a negative rate effect. The shear zone was planar and horizontal with nearly uniform width and the secondary shear surfaces did not appear to join the principal shear zone at any point along the outer face of the sample [Figs. 5.4.1.7, 5.4.1.8, VI.1.2 and VI.2.2].

The scatter in the data associated with the tests in which the soil exhibited a negative rate effect [Figs. 6.3.35-37] is attributed to the undulating shape of the shear zone. Therefore, the observed variations of the fast residual strength should not be viewed as a general indication of soil behaviour during fast shearing but rather as a result of the type of shearing mechanism which develops in ring shear tests.

Evidence from the available data indicates that the average magnitude of the decrease in fast residual strength below slow residual increases with increasing amplitude of the undulating shear zone [Fig. 6.3.38]. This suggests that the shearing mechanism involved in annular samples which exhibit negative rate behaviour is significantly affected by a periodic "pumping action" which is induced by the shape of the shear zone.

Similar indications of a negative rate effect were observed in the tests conducted on the normally consolidated, submerged Cowden till samples (Tests IV.17-20 and VI.3-4) [Figs. 6.3.39-44] and the last test conducted on the overconsolidated, submerged sample of the same soil (Test IV.23) [Fig. 6.3.45]. The presence of an undulating shear zone appeared to influence the behaviour of samples during the fast shearing stages of each test. Test IV.17 which was conducted on the normally consolidated, submerged sample under the normal stress of 50 kPa indicated that the fast residual strength stabilised above the slow residual in the first part of the test, while the shear zone did not appear to be undulating. However, as the test progressed, the shear zone developed a clear undulating shape and the fast residual strength eventually stabilised below the slow residual in the latter part of the test [Fig. IV.17.2]. A clearer picture of the effect of the

undulating shear zone on the fast residual strength of this soil emerged from the remaining tests which were conducted on the same sample. The fast residual strength stabilised below the slow residual and was independent of any changes in the rate of displacement. The amplitude of the undulating shear zone was nearly constant, around 0.53 mm, under the normal stress of 100 kPa, whilst it appeared to decrease from 0.5 mm to 0.2 mm towards the latter part of the test which was conducted under the normal stress of 200 kPa [Figs. IV.18.2, IV.19.2]. This decrease in amplitude of the undulating shear zone was followed by an increase in the fast residual strength as with the brown London clay samples. Similar behaviour was observed with the same sample under the normal stress of 300 kPa [Fig. IV.20.2]. It appears that the increase in fast residual strength observed during the latter part these last two tests is not so much a result of the increasing rate of displacement but rather is a consequence of the decreasing amplitude of the undulating shear zone of this sample.

Inspection of this sample after it was extruded from the confining rings was made difficult because its upper and lower parts split so that the size and position of the shear zone could not be measured. Nevertheless, the shearing soil matrix had a softer texture than the remaining parts of the sample and also appeared to have greater porosity. Further evidence of the nature of the shear zone is provided by photographs of air-dried fragments of the sample [Fig. 5.4.2.3]. The concave shape of the surface which occurs between the shear zone and the lower part of the sample indicates that the shear zone is oval-shaped and its size varies throughout the width of the sample.

A second identically-prepared sample of the same soil was tested under identical conditions (Tests VI.3-4). Since it appeared that the negative rate effect is a result of increasing pore pressures, a second slow shearing stage was included following the fast shearing stage. Thus, if pore pressures did, indeed, increase during the fast shearing stages, they should gradually dissipate as the sample was subsequently sheared at a drained rate of displacement, and the measured strength should eventually return to its slow residual value. Both tests which were conducted on this second normally consolidated, submerged Cowden till sample showed similar behaviour during fast shearing to that observed during the tests on the first sample. The fast residual strength stabilised below the slow residual, and was independent of any changes of the rate of displacement [Figs. 6.3.43-44]. The scatter of the data is attributed to the effect which

the presence of the undulating shear zone has on the shearing mechanism. Furthermore, the undulating shear zone which developed had an amplitude which varied between 0.5 and 0.4 mm during the test conducted under 100 kPa normal stress and remained nearly steady, around 0.3 mm, during the test at 200 kPa. [Figs. VI.3.2, VI.4.2 and Table 6.2]. Finally, when the rate of displacement was decreased to its drained level (0.09 mm/min), the measured strength gradually increased and stabilised around its slow residual value, thus, supporting the argument that the negative rate effect is associated with increased pore pressures within the shearing soil matrix.

Further evidence of the undulating shape and irregular thickness of the shear zone of this sample was observed upon measuring its size and position and visually inspecting the sample [Fig. 5.4.2.5]. The photographs which were taken from the sample after it was impregnated by Carbowax [Fig. 5.4.2.4] show the highly variable thickness of the shear zone and the presence of secondary shear surfaces which appear to join the principal shear zone, thus introducing additional irregularities in the shearing mechanism.

The negative rate effect was also encountered in the third test which was conducted on the overconsolidated, submerged Cowden till sample under the normal stress of 300 kPa. The fast residual strength stabilised below the slow residual [Fig. 6.3.45] and was not dependent on any changes in the rate of displacement whilst an undulating shear zone appeared to have developed with an amplitude which remained nearly constant at about 0.54 mm throughout this fast shearing stage [Fig. IV.23.2 and Table 6.2]. Once again the sample split when an attempt was made to separate it from the porous stones and a softer, in comparison with the rest of the sample, texture of the shearing soil matrix was observed.

It has been shown that the negative rate effect is observed when the shear zone is undulating. Moreover, the magnitude of the negative rate effect appears to be related to the amplitude of the undulating shear surface [Fig. 6.3.46]. The available data [Table 6.2] indicate that, when the amplitude of the undulating shear zone is less than 0.1 or 0.2 mm, the fast residual strength does not drop below the slow residual and the rate effect is positive. However, when that amplitude increases above 0.2 or 0.25 mm the fast residual strength falls below the slow residual. The magnitude of the drop of the fast residual strength below the slow residual value increases with increasing amplitude of the

undulating shear zone and appears to reach a maximum level around an amplitude of 0.6 mm. Further investigation on this subject is required.

6.3.5 "Intact" Samples

In order to investigate the effect which the initial slow shearing stage of each test and the development of the residual structure have on the behaviour of the plastic soils which have been used in this research programme, a different testing procedure was adopted for a limited number of tests. In this, the sample was consolidated and then sheared immediately at fast rates of displacement. Two tests were conducted on an overconsolidated, submerged brown London clay sample and two on a normally consolidated, submerged Cowden till sample which were prepared by the same methods as previously.

The tests on brown London clay (Tests VI.15-16) [Figs. VI.15.1, VI.16.1] indicate a similar behaviour upon fast shearing to that observed earlier. The fast residual strength stabilised above the corresponding slow residual strength, and appeared to reach a maximum when the rate of displacement is about 1.0 cm/sec [Figs. 6.3.47-48]. Although the shear zone during the first test (100 kPa) appeared initially to be undulating, the undulations progressively decreased in amplitude. It is thought that the irregular shape of the shear zone at the early stages of the first test may be attributed to the sudden application of the fast rates of displacement and the development and mobilisation of secondary shear surfaces which progressively lead to the formation of a principal shear zone and a more steady shearing mechanism [Fig. 5.6.1]. In the second test (200 kPa) the principal shear zone which had been formed previously was mobilised immediately and the behaviour of the sample indicated a positive rate effect. The shape of the shear zone appeared to be undulating with an amplitude of around 0.15 mm but this, as has been suggested previously, did not affect the behaviour of the sample.

The tests conducted on the Cowden till sample (Tests VI.17-18) also did not provide further information about the behaviour of soils during fast shearing [Figs. VI.17.1, VI.18.1]. The fast residual strength stabilised below the corresponding slow residual value and was independent of any further changes in the rate of displacement [Figs. 6.3.49-50]. Although during the first test the shear zone appeared to undulate

slightly, during the second test the amplitude of this undulation increased considerably. After the test was terminated the sample was extruded from the confining rings and its water content was measured at its top and bottom parts. Also, the water content of the shear zone was measured [Table 5.2.2]. It is interesting to note that, although the water content at the top and bottom parts of the sample was measured to be below the plastic limit of the soil, the water content of the shear zone approached the liquid limit.

6.3.6 Completely Dry Samples

Finally, the tests which were conducted on completely dry, powdered samples of brown London clay (Tests VI.5-8) and Cowden till (Tests VI.9-12) indicate insignificant rate effects in the absence of water within the soil mass. One sample of each soil was tested normally and overconsolidated under the normal stresses of 100 and 200 kPa. The fast residual strength measured during each test will be compared with the slow residual strength which was measured during the slow shearing stage of that same test. The range of normal stresses used was not sufficient to establish the complete failure envelope.

The fast residual strength of the normally consolidated brown London clay sample stabilised slightly above the slow residual strength and was not affected by any changes in the rate of displacement [Figs. 6.3.51-52]. The fast residual strength of the same sample when overconsolidated stabilised slightly below the slow residual strength and was, also, not affected by any further changes in the rate of displacement [Figs. 6.3.53-54]. The fast residual strength of the normally and overconsolidated Cowden till sample generally stabilised slightly below its slow residual strength and appeared independent of any variation of the rate of displacement [Figs. 6.3.55-58].

The behaviour of the dry samples during fast shearing (with the exception of the tests which were conducted on the normally consolidated brown London clay) suggests a decrease in strength with increasing rates of displacement. This may be attributed to the fact that, during fast shearing, the shearing mode of the particles changes from sliding to turbulent, thus leading to an increased porosity of the shearing matrix, a better ability of the particles to move relative to each other and a decrease in measured strength. Nevertheless, the rate effect on dry samples is considered relatively insignificant and the behaviour of such samples insensitive to variations of the rate of displacement.

6.4 Non-Plastic Soils

6.4.1 Threshold Strength

The tests which were conducted on the non-plastic soils used in this investigation, *i.e.* pure silica (Tests V.1-11) and the artificially mixed sandy-silt (Tests V.12-20), did not lead to any clear conclusions associated with the threshold strength and the factors which affect it. Lemos (1986) concluded that the threshold strength measured in ring shear tests should be considered as an intercept to the failure envelope rather than an actual increase of the shearing resistance of the sample. The results from this investigation suggest that the threshold strength measurements must be influenced by apparatus effects which cannot be quantified.

In the tests on pure silica samples the threshold strength did not appear to follow any particular trend associated with the normal stress or the rate of displacement [Figs. 6.4.1-2]. Although a wide range of rates of displacement and normal stresses were employed in these tests, the evidence does not indicate any systematic change in the threshold strength with changing rates of displacement and varying normal stresses. Similar results were obtained from the tests conducted on the sandy-silt samples [Figs. 6.4.3-4].

It is thought that the nature of these soils influences the behaviour of the apparatus and, indeed, of the samples at the starting point of each fast shearing stage. The threshold strength measurements are partly influenced by the testing procedure, as has been discussed earlier in this chapter, and partly by a stick-slip mechanism involving soil particles inside the gap of the confining rings. Once the shear stress is removed from the sample, in order to engage the fast gear, the soil inside the gap ceases to move and jams in the rims of the upper and lower of the confining rings. When shearing is re-started, a part of the measured shear resistance must be attributed to the force required to overcome this stick-slip mechanism and re-establish the rotation of the confining rings. Once shearing has re-started the material within the gap moves continuously and exits slowly from the rings. This contributes to the gap friction which is measured at the end of each test as described in Chapter 4. Thus, it may be concluded that the threshold strength is affected by various factors which operate at the start of each fast shearing stage for which allowance cannot easily be made.

6.4.2 Post-threshold Strength Decrease

The behaviour of the non-plastic soils immediately after they had reached their threshold strength was similar to that of the plastic soils. This post-threshold strength decrease is associated with the immediate dilation of the sample. However, unlike the results from tests conducted on plastic soils, the tests on non-plastic soils do not indicate the existence of any relationship between the post-threshold strength decrease and the normal stress or the rate of displacement [Figs. 6.4.5-8]. Moreover, although the results from almost all tests on pure silica and sandy-silt samples (presented in detail in Appendix V) indicate an increase in sample height at the beginning of each test, no correlation between the immediate dilation and the post-threshold strength decrease can be seen [Figs. 6.4.9-10]. Nevertheless, the post-threshold strength was in all tests lower than, or equal to, the slow residual strength, despite the magnitude of the immediate dilation of the sample.

6.4.3 Negative Rate Effect

The ultimate response of the non-plastic soil samples which were tested in this investigation was rather uniform. The fast residual strength stabilised, on average, below the slow residual strength and was not affect by any further changes in the rate of displacement [Figs. 6.4.11-30]. This type of behaviour is expected from soils with a high slow residual strength and is associated with an increase in the porosity of the shearing matrix. Similar conclusions concerning the behaviour of silts or silty soils with high slow residual strength (associated with turbulent or transitional shear modes) have been reached by previous researchers (Lupini, 1981; Lemos, 1986; Tika, 1989).

The negative rate effect in this case may be associated with the *flow structure* theory (Casagrande, 1975) which involves the continuous rotation of soil particles relative to each-other and ceases to exist once shearing is halted. It may be suggested that upon the application of the fast rate of displacement the soil particles are forced to rearrange themselves, the porosity of the soil matrix which is associated with shearing increases -and this may be related to the immediate dilation of the sample which has been described previously- and the particles are able to rotate more easily relative to each

other. This mechanism would lead to a decrease in shearing resistance. Therefore, the fast residual strength stabilises below the slow residual strength and it is not affected by any further changes in the rate of displacement. Furthermore, the results from this investigation indicate that, when the sample was unloaded in shear and shearing was restarted, the measured strength reached a new threshold value before it stabilised to the previously established level below the slow residual strength. This is in agreement with the flow structure theory which suggests that such structure collapses, when the shearing process stops, and the shearing matrix becomes denser resulting in an increase in measured strength, when shearing is restarted.

The results from the tests on pure silica and sandy-silt indicate that the fast residual strength stabilises 2° to 6° below the slow residual strength [Figs. 6.4.11-30] which could be translated in a 4 % to 12 % decrease in strength. No further correlation between the fast residual strength and the normal stress could be deducted from the available data.

6.4.4 Coarse Uniform Clean Sand

The purpose of the tests which were conducted on dry (Tests V.21-23), or moist (Tests V.24-27), clean-coarse sand [Figs. V.21.1-V.27.1] was to investigate the rate dependency of their fast residual strength and not to compare it with their slow residual strength. The data indicate clearly that the fast residual strength of clean-coarse sand is rate independent [Figs. 6.4.31-32] regardless of whether the sample is dry or moist. Due to the fact that the same sample was used for the tests at all normal stress levels and particle crushing has taken place during each shearing stage, the particle size distribution has altered. As a consequence, no composite residual strength envelope was developed from these results; nor was any correlation attempted.

6.5 Summary

The results from this investigation have been analysed and discussed in this chapter. The behaviour of soils during fast shearing and the effects of high rates of

displacement on the fast residual strength have been divided into two principal types and two hybrid conditions [Fig. 6.5.1]:

Type I behaviour [Fig. 6.5.1.a] is the ***positive rate effect***, where the *post-threshold strength* and the *fast residual strength* stabilise above the slow residual strength. This behaviour is observed with plastic soils, submerged (with planar shear zone) or non-submerged (regardless of the shape of the shear zone). The fast residual strength decreases with increasing normal stress, reaching its maximum value when the rate of displacement is around 1.5 cm/sec.

Type Ia behaviour [Fig. 6.5.1.b] occurs when there is a positive rate effect, and the *fast residual strength* stabilises above the slow residual strength, but the *post-threshold strength* falls below slow residual. This behaviour occurs with the same soil types which have been listed in behaviour Type I.

Type II behaviour [Fig. 6.5.1.c] is the ***negative rate effect***, where the *post-threshold strength* and the *fast residual strength* stabilise below the slow residual level. It occurs with non-plastic soils (submerged or non-submerged) and plastic soils (submerged with undulating shear zone). In the case of the non-plastic soils, the negative rate effect is considered to be an intrinsic property, whereas in the case of the plastic soils it is attributed to increased pore pressures which are induced by the undulating shear zone. The fast residual strength is independent of the rate of displacement and is not affected by normal stress variations.

Type IIa behaviour [Fig. 6.5.1.d] is a composite behaviour, where the *post-threshold strength* falls below the slow residual strength, and the *fast residual strength* initially stabilises above the slow residual strength but, as shearing progresses, falls below slow residual. This type of behaviour occurs with submerged plastic soils with undulating shear zone which gradually increases in amplitude.

The general conclusions which can be drawn from this investigation will be presented in the following chapter.

test type	test number	σ_n (kPa)	rate of displace-ment (cm/sec)	immediate dilation (mm)	threshold strength (°)	post-threshold strength decrease (°)	slow residual strength (°)	shear zone shape	undulation amplitude (mm)	rate effect (°)
NC NS	IV.1	50	0.72	0.72	25.8	16.1	15.6	undulating	0.62/0.13	3.3
	IV.2	100	0.12	0	14	0	11.7	planar	-	2.3
	IV.2	100	1.4	0	14	0	11.7	planar	-	10.8
	IV.3	200	0.05	0	-	0	11.3	planar	-	0.2
	IV.3	200	1.29	0.42	20.8	9.3	11.3	planar	-	15
	IV.4	400	0.04	0	-	0	12.4	non-planar	-	3.7
	IV.4	400	1.38	0.53	21.3	9.1	12.4	non-planar	-	2.4
NC S	IV.6	50	0.63	0.15	29.8	16.7	14.5	planar	-	11
	IV.7	400	1.31	0.05	20.3	6.3	11.3	planar	-	5.1
	IV.8	100	0.35	0.01	19.8	1.3	11.3	-	-	7.2
	IV.8	100	1.71	0.21	22.8	9.5	11.3	undulating	0.3/0.1	-5.4
	IV.9	200	0.11	0.03	16.1	2.1	11.9	planar	-	2.1
	IV.9	200	1.7	0	-	0	11.9	planar	-	11.4
OC S	IV.10	100	1.76	0.88	36.9	24.3	15.5	undulating	0.6/0.18	-9.8
	IV.11	200	3.77	0.85	24.7	19.6	11.4	undulating	0.3/0.08	-6.6
	IV.12	300	7.8	0.48	24.6	13.3	12.3	undulating	-	-3.9
OC S 3Stg	VI.1	100	1.5	0.2	27	14.2	10.9	planar	-	10.1
	VI.2	200	1.25	0	21.5	3.1	11	planar	-	10
NC DRY	VI.5	100	1.26	-	43	12.7	37.4	planar	-	2.3
	VI.6	200	1.27	-	42.3	1.1	37.6	planar	-	4.4
OC DRY	VI.7	100	1.36	-	44.7	4.2	42.1	planar	-	-2.1
	VI.8	200	1.15	-	41.9	-	41.5	planar	-	-1.5
OC S INT	VI.15	100	1.29	0.52	49.5	23.5	-	non-planar	-	-
	VI.16	200	1.4	-	20.3	-	-	non-planar	-	-

Table 6.1 Tests on brown London clay

test type	test number	σ_n (kPa)	rate of displace-ment (cm/sec)	immediate dilation (mm)	threshold strength (°)	post-threshold strength decrease (°)	slow residual strength (°)	shear zone shape	undulation amplitude (mm)	rate effect (°)
NC NS	IV.13	50	1.7	0.48	47.3	13.2	35.4	undulating	0.14/0.03	7.9
	IV.14	100	0.09	0	37.7	0	31.4	-	0.12/0.01	6.3
	IV.14	100	1.68	0	41.3	0	31.4	undulating	0.12/0.01	11.3
	IV.15	200	1.78	0.01	39.5	6.6	31.4	planar	-	7.1
	IV.16	400	1.69	0.34	38.7	13.1	31.4	non-planar	-	3.4
NC S	IV.17	50	1.29	1.8	45.3	21.8	31.8	undulating	0.24/0.6	-9
	IV.18	100	1.3	2.9	39.7	18.1	31.7	undulating	0.91/0.65	-22.7
	IV.19	200	1.25	0.35	40.4	17.2	31.4	undulating	0.53	-24.4
	IV.20	300	1.35	0.6	52	22.3	31.7	undulating	0.5	-20.4
OC S	IV.21	100	1.68	0.22	40.6	5.4	33.8	undulating	0.09	7.3
	IV.22	200	1.7	0.3	34.7	8.8	31.1	undulating	0.05	4.4
	IV.23	300	1.71	1.09	33.8	14	31.2	undulating	0.54	-20.7
NC S 3Stg	VI.3	100	1.39	0.89	41	14	30.7	undulating	0.5/0.4	-15.4
	VI.4	200	1.29	0.3	47.1	21.5	30.9	undulating	0.3	-16.9
NC DRY	VI.9	100	1.72	0	48	7.5	42	planar	-	-3
	VI.10	200	1.69	0	42	0.6	41.4	planar	-	-3.4
OC DRY	VI.11	100	1.71	0.02	44.3	4.2	42.6	planar	-	-4.1
	VI.12	200	1.7	0.02	42.9	3.1	37.8	planar	-	-2
NC S INT	VI.17	100	1.26	-0.42	34.2	9	-	non-planar	-	-
	VI.18	200	1.28	-0.04	28.8	5.4	-	undulating	0.48/0.57	-

Table 6.2 Test on Cowden till

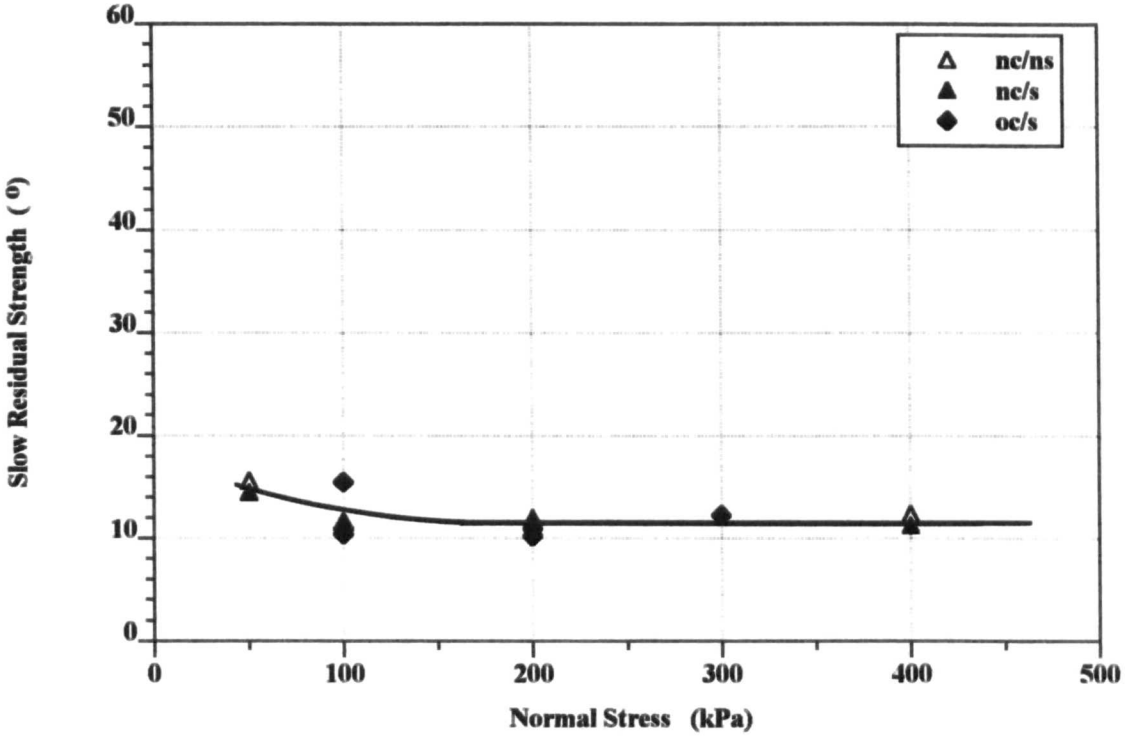


Fig. 6.2.1 Slow residual strength envelope for brown London clay

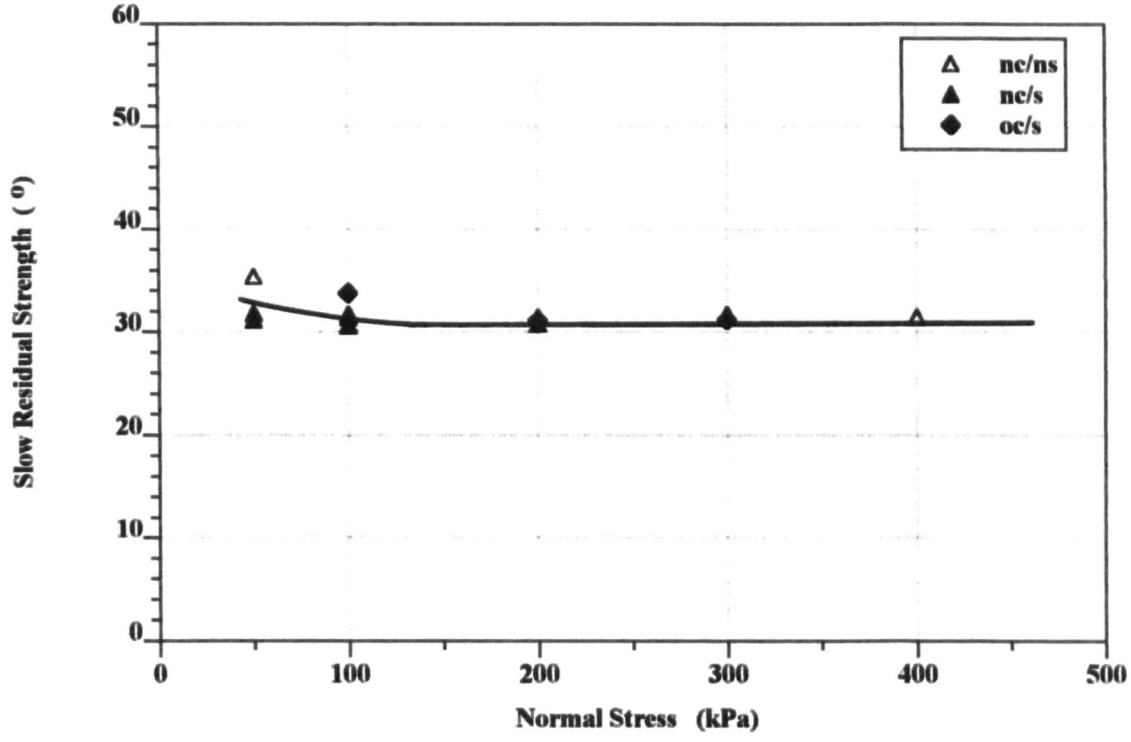


Fig. 6.2.2 Slow residual strength envelope for Cowden till

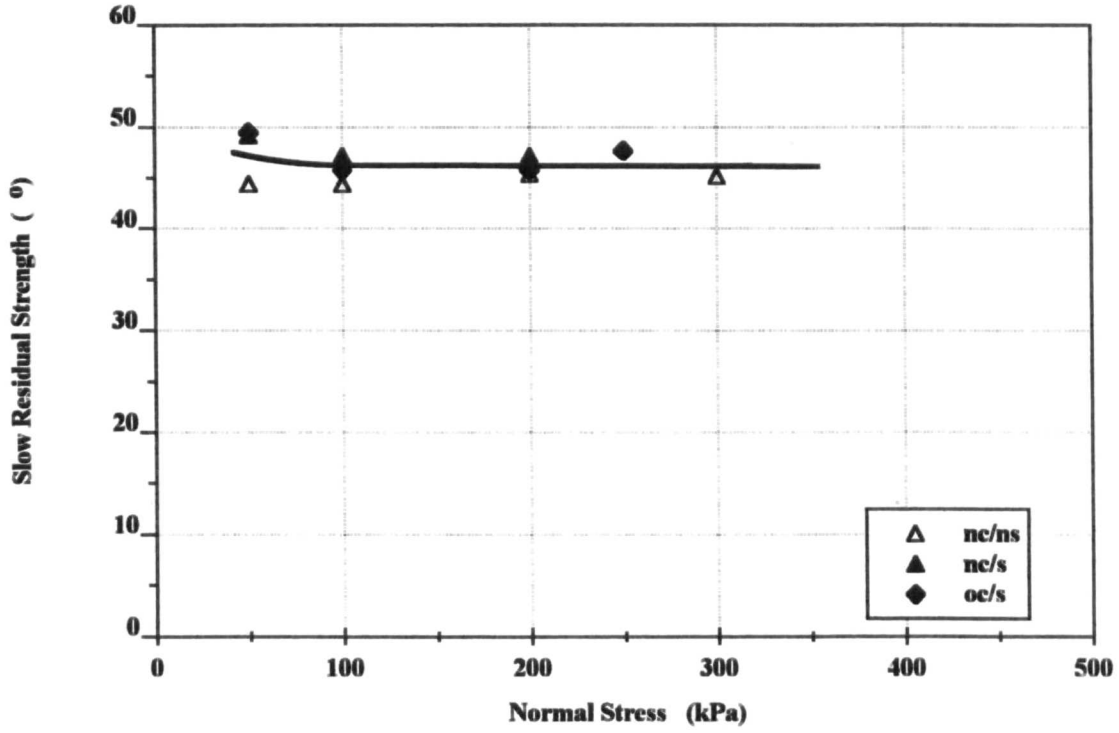


Fig. 6.2.3 Slow residual strength envelope for pure silica

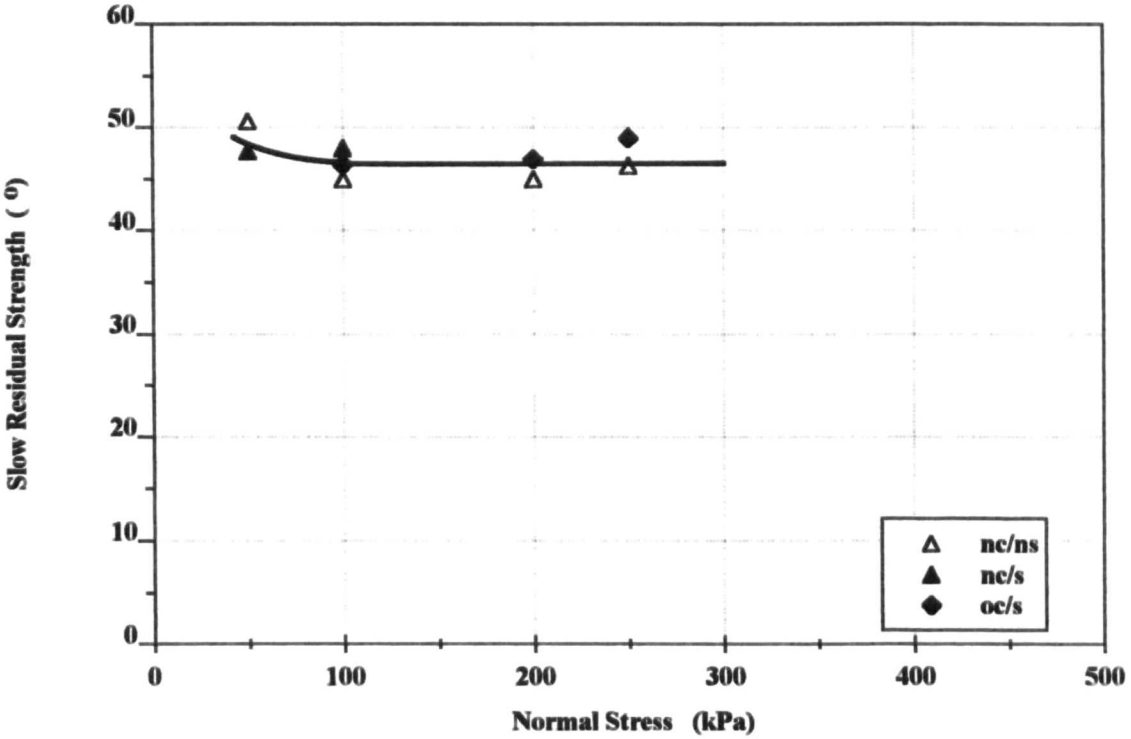


Fig. 6.2.4 Slow residual strength envelope for artificial sandy-silt

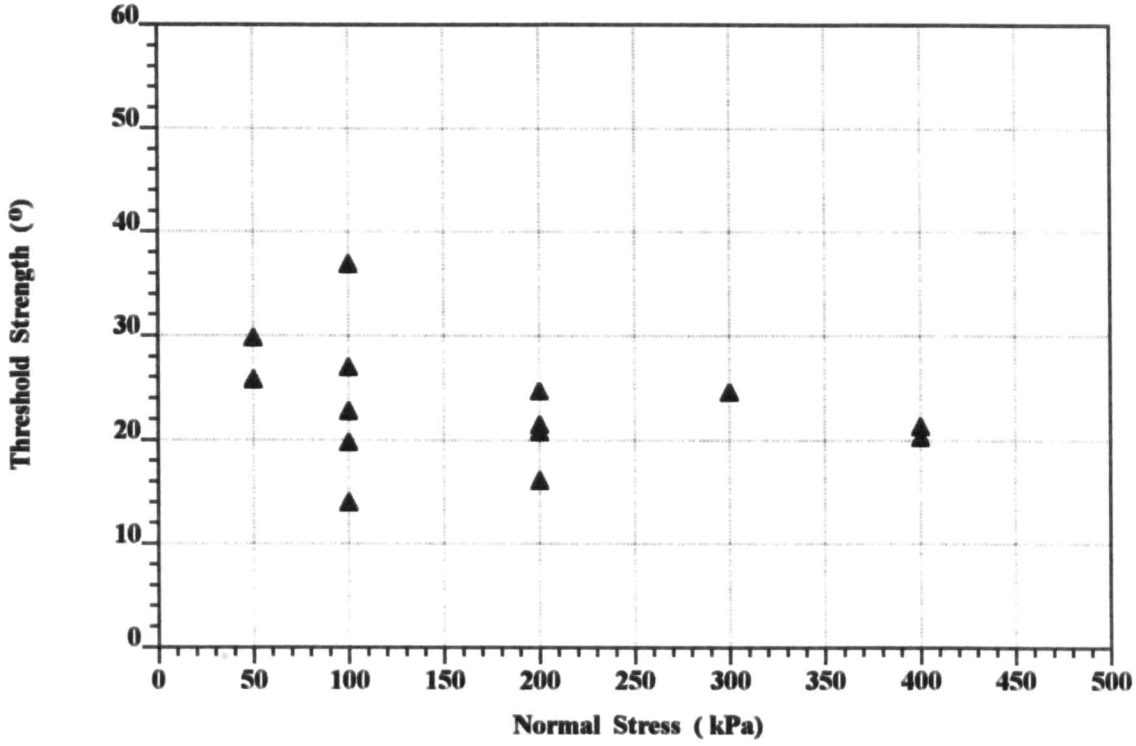


Fig. 6.3.1 Threshold strength versus normal stress for brown London clay

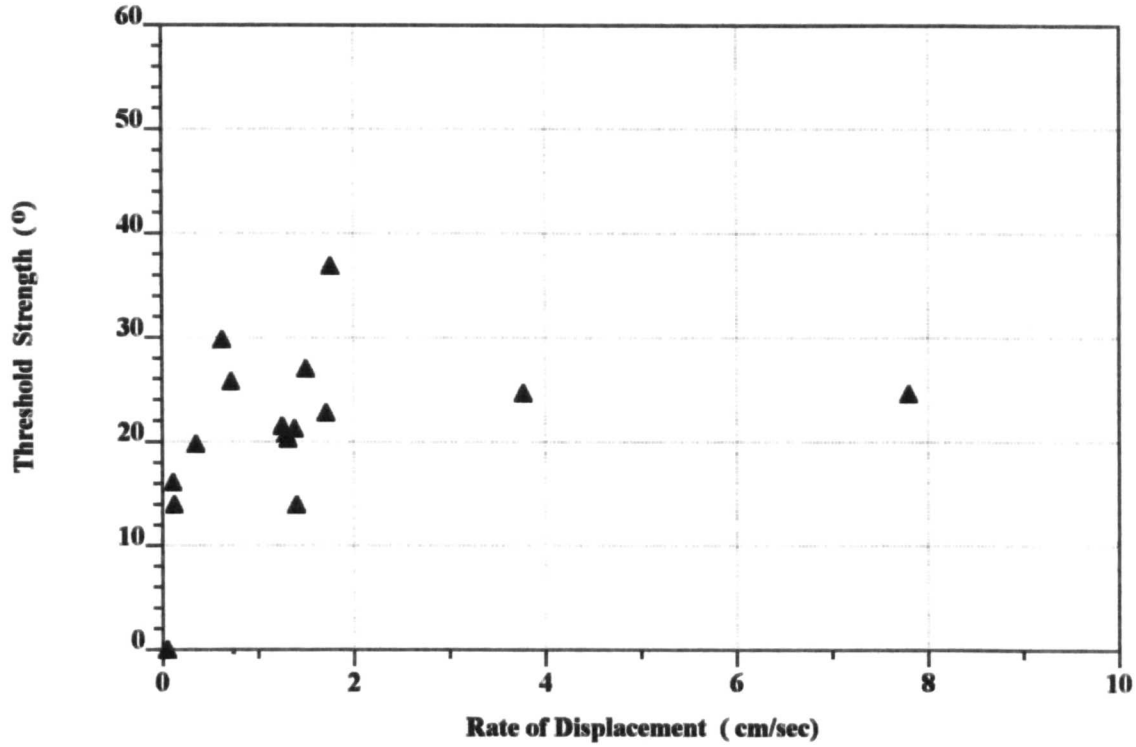


Fig. 6.3.2 Threshold strength versus rate of displacement for brown London clay

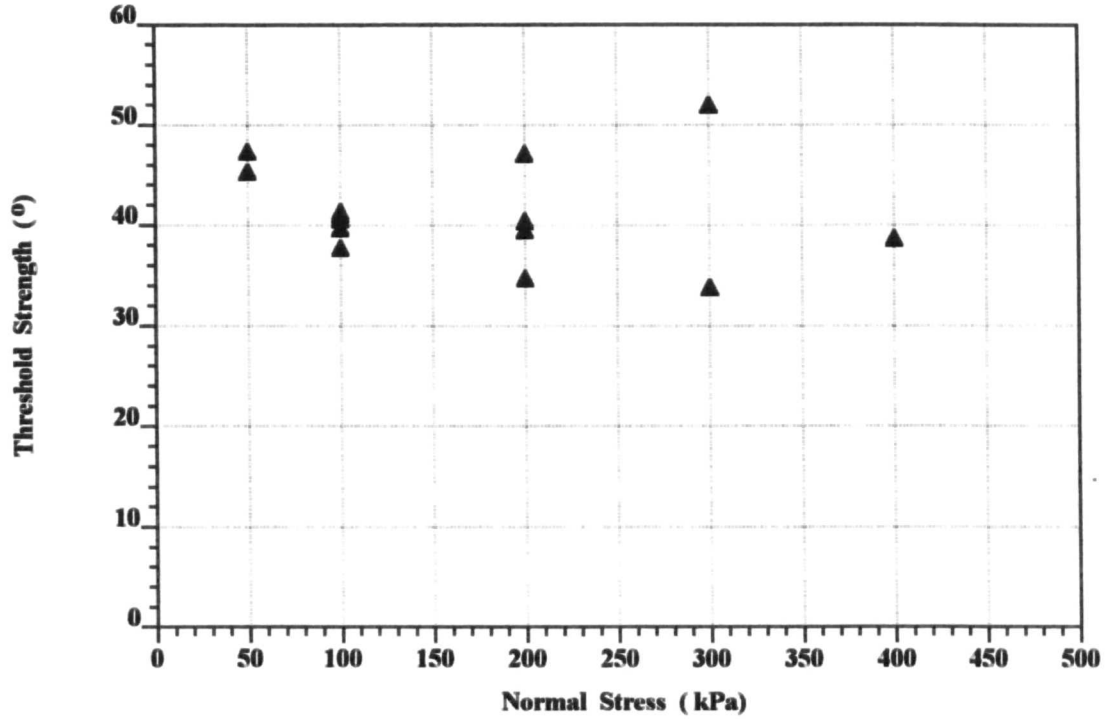


Fig. 6.3.3 Threshold strength versus normal stress for Cowden till

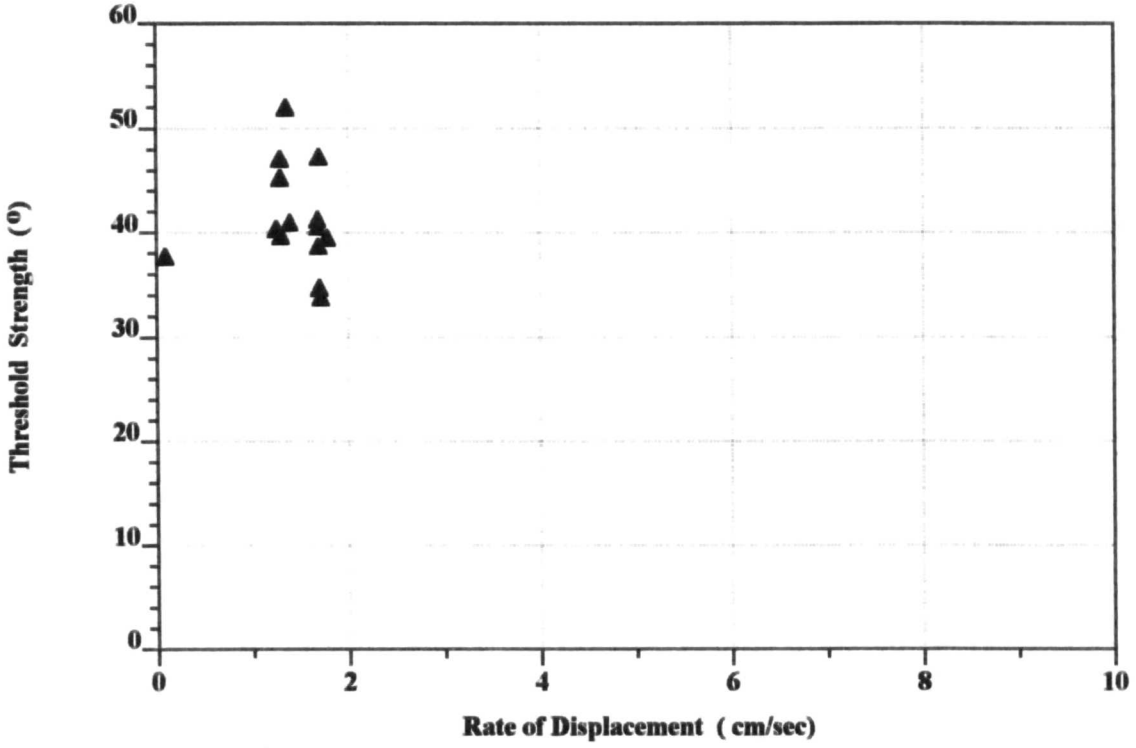


Fig. 6.3.4 Threshold strength versus rate of displacement for Cowden till

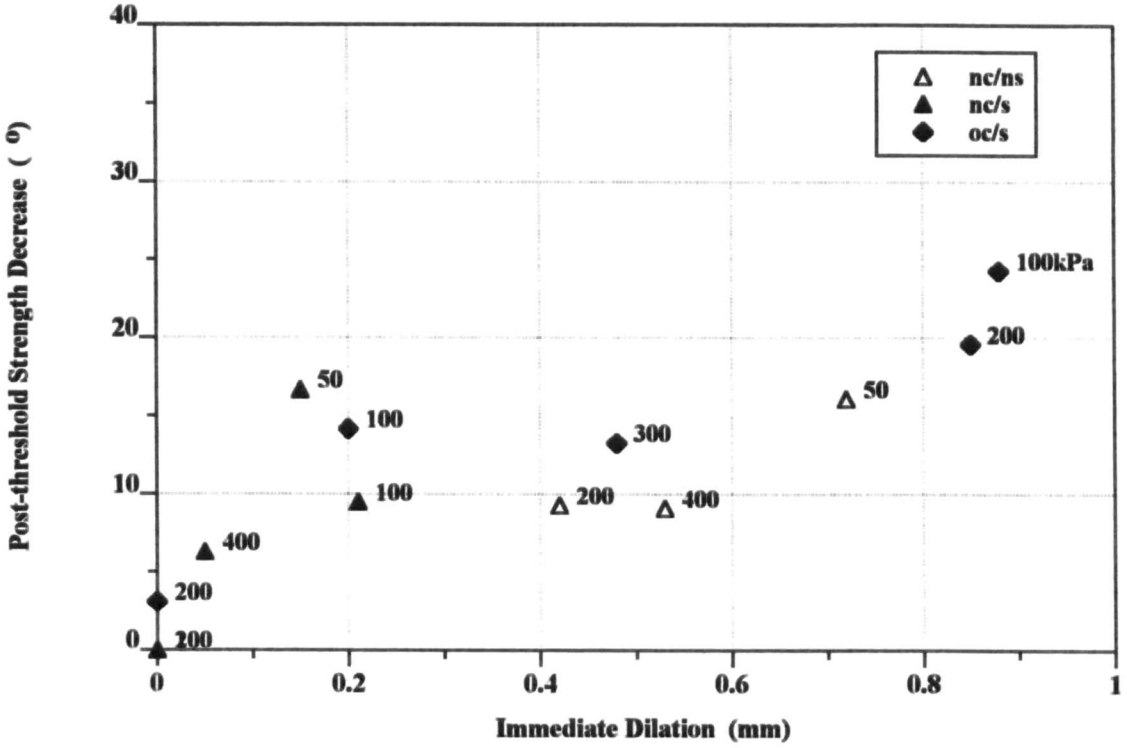


Fig. 6.3.5 Post-threshold strength decrease for brown London clay

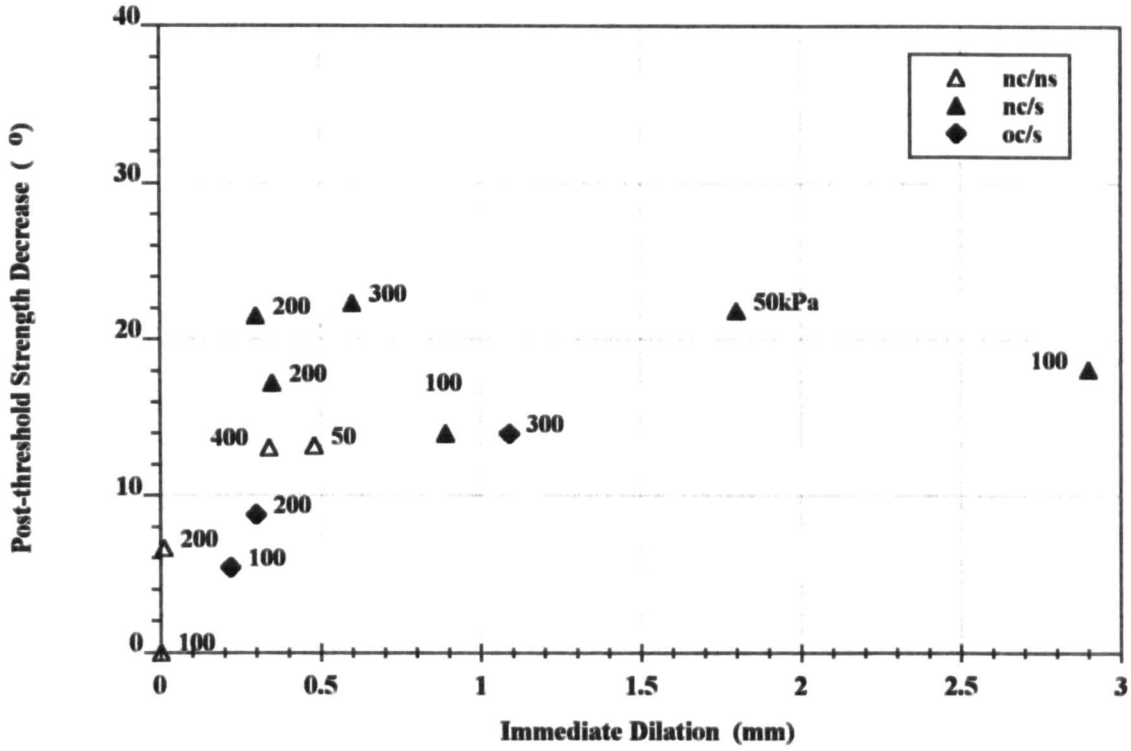


Fig. 6.3.6 Post-threshold strength decrease for Cowden till

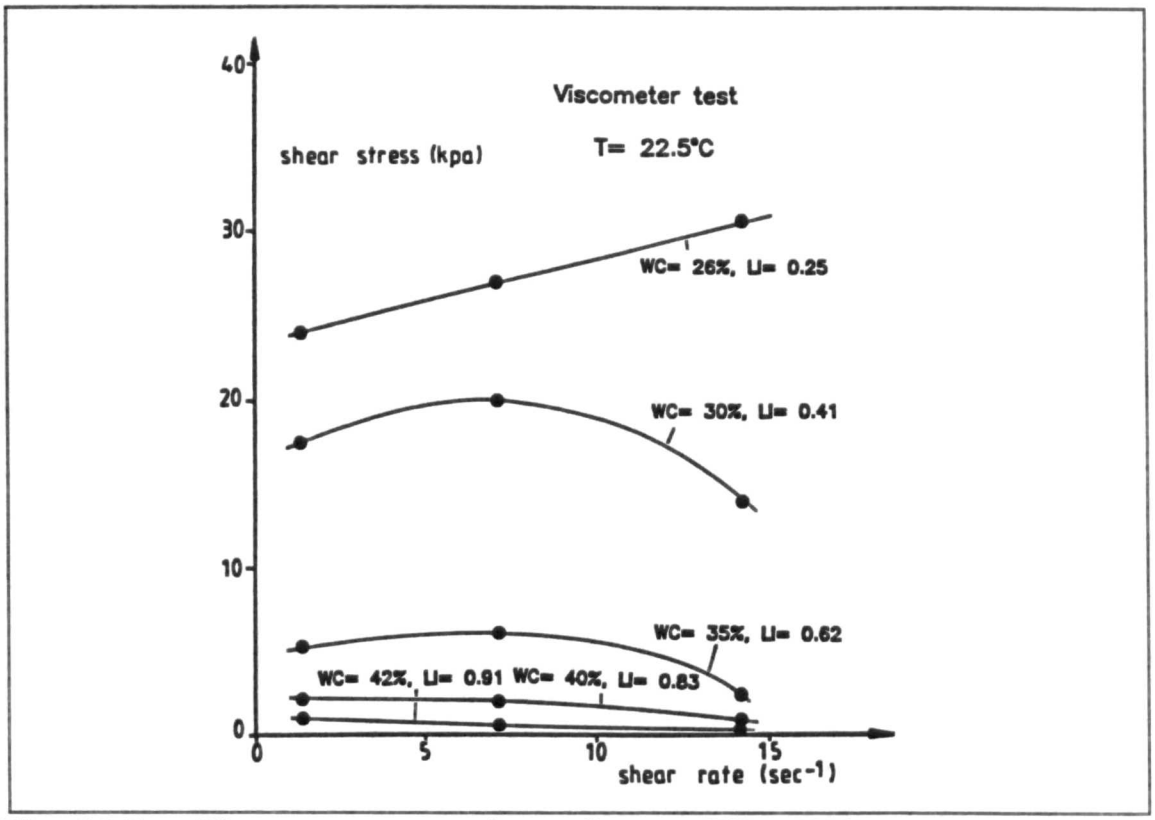


Fig. 6.3.7 Viscometer tests on clayey siltstone (Tika, 1989)

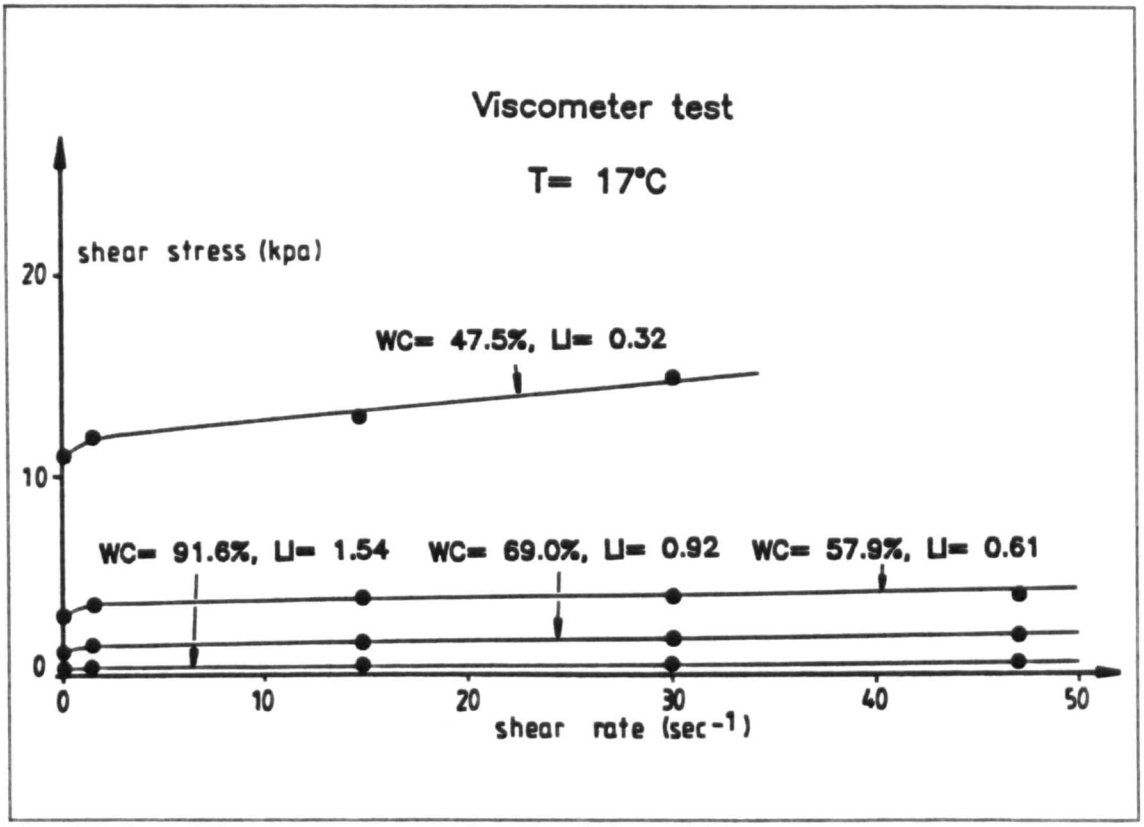


Fig. 6.3.8 Viscometer tests on kaolin (Tika, 1989)

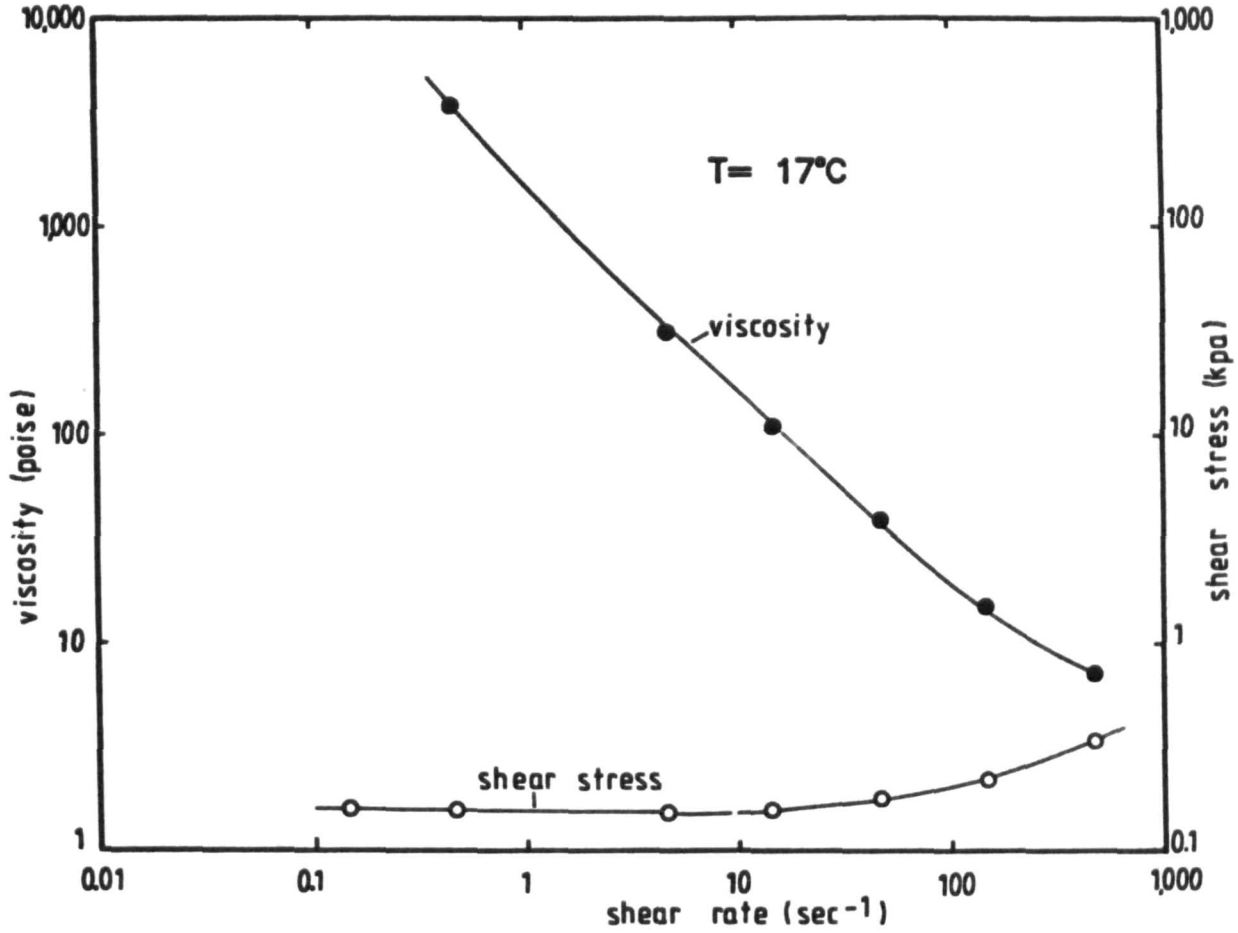


Fig. 6.3.9 Viscometer tests on glycerol (Tika, 1989)

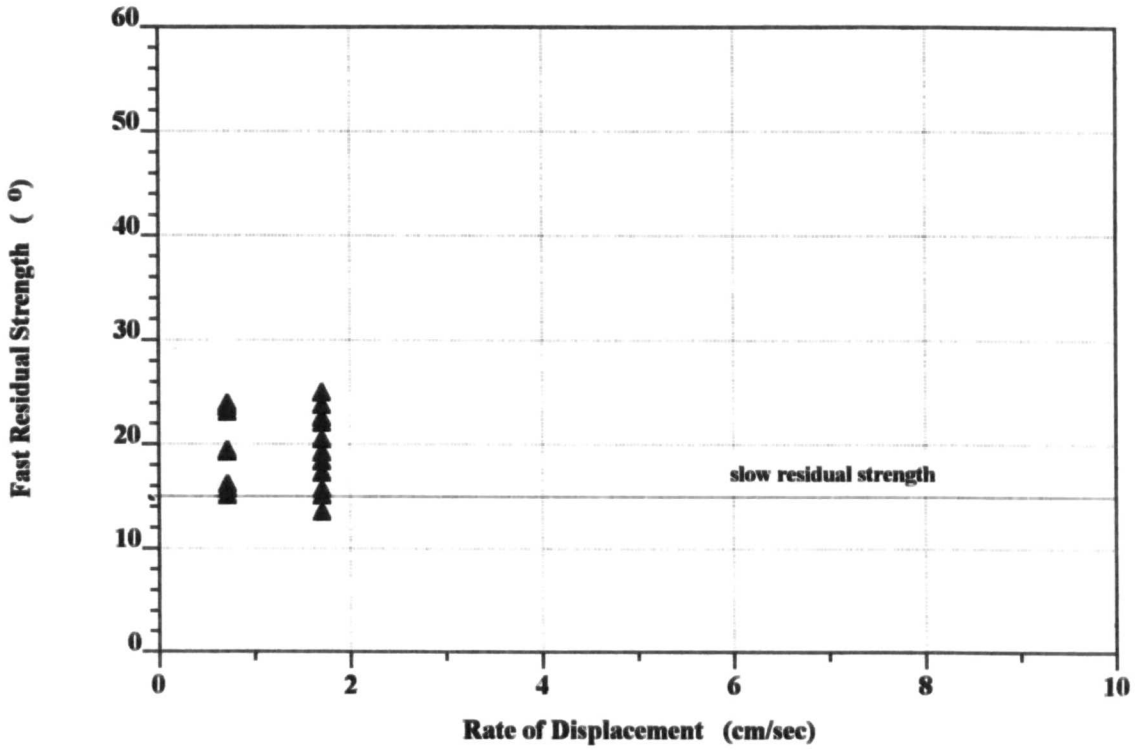


Fig. 6.3.10 Rate dependent characteristics of normally consolidated, non-submerged, brown London clay (Test IV.1)

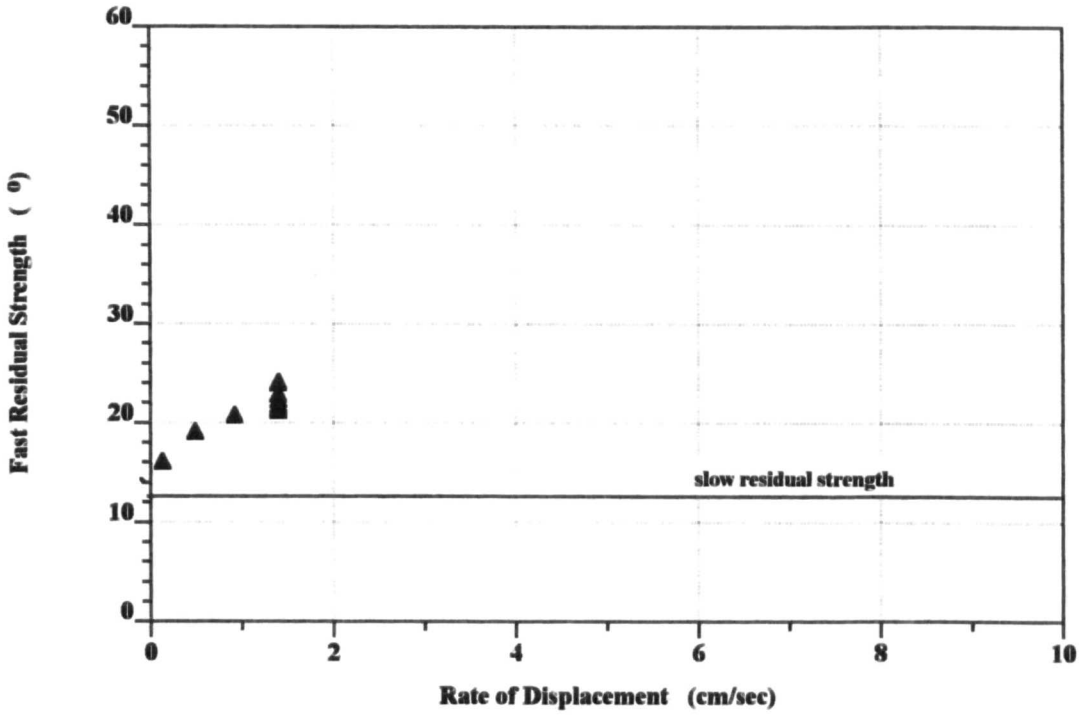


Fig. 6.3.11 Rate dependent characteristics of normally consolidated, non-submerged, brown London clay (100 kPa)

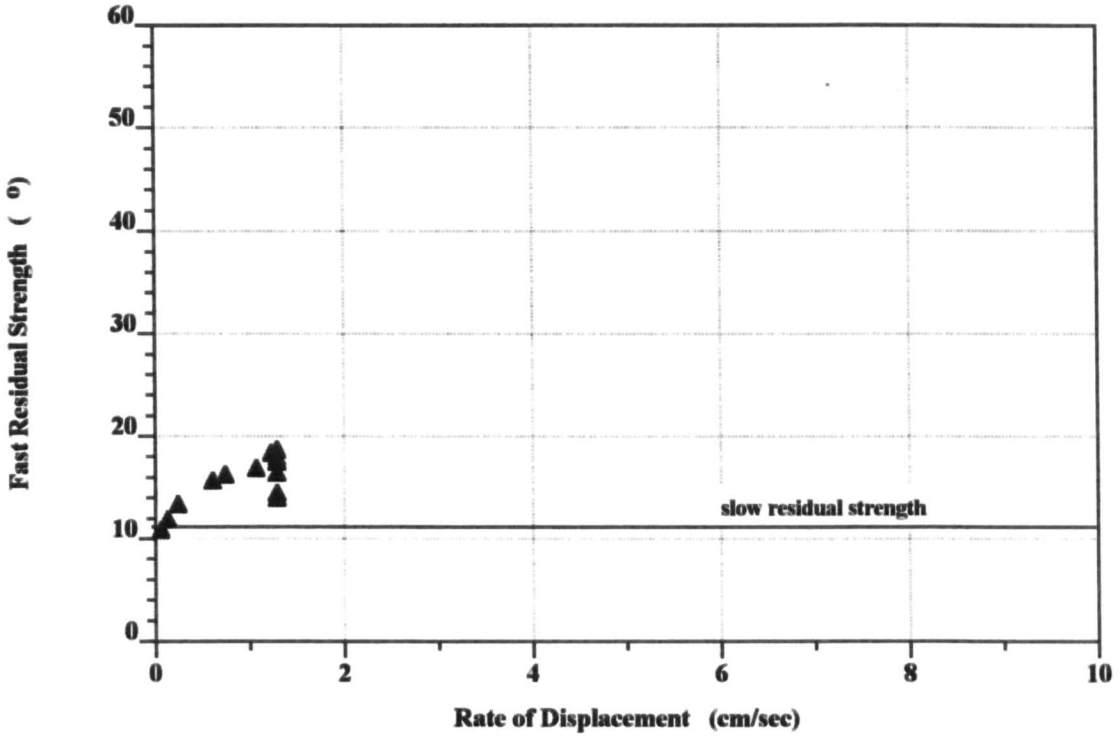


Fig. 6.3.12 Rate dependent characteristics of normally consolidated, non-submerged, brown London clay (200 kPa)

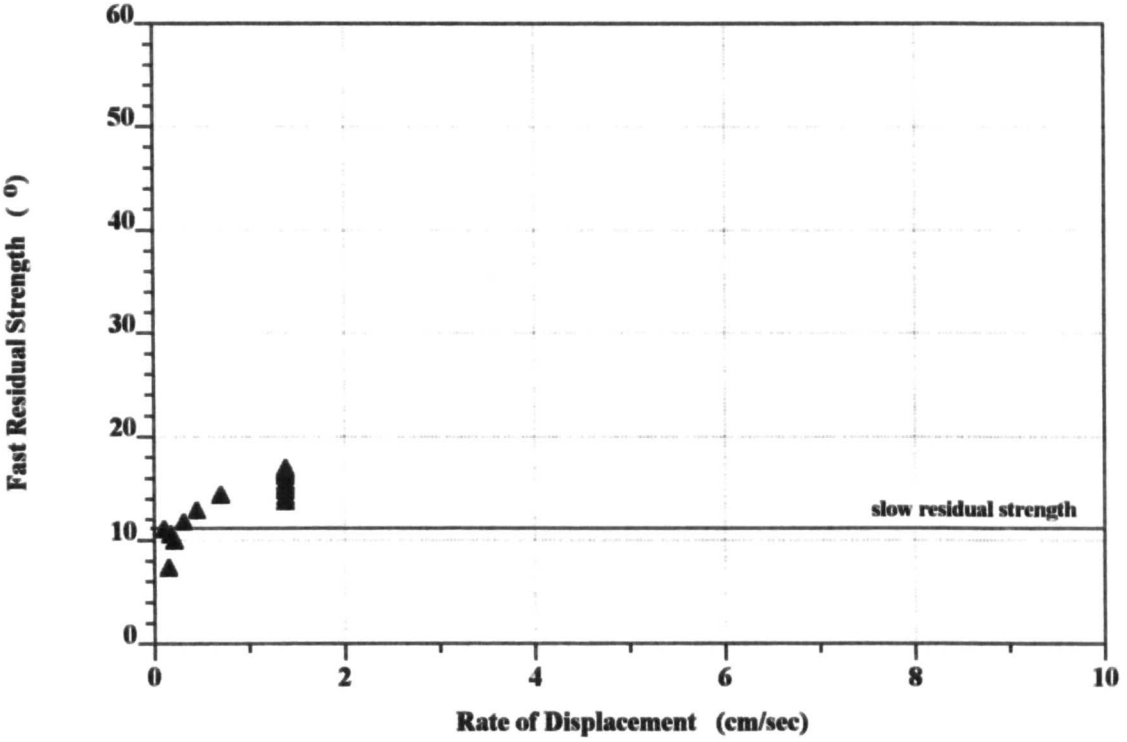


Fig. 6.3.13 Rate dependent characteristics of normally consolidated, non-submerged, brown London clay (400 kPa)

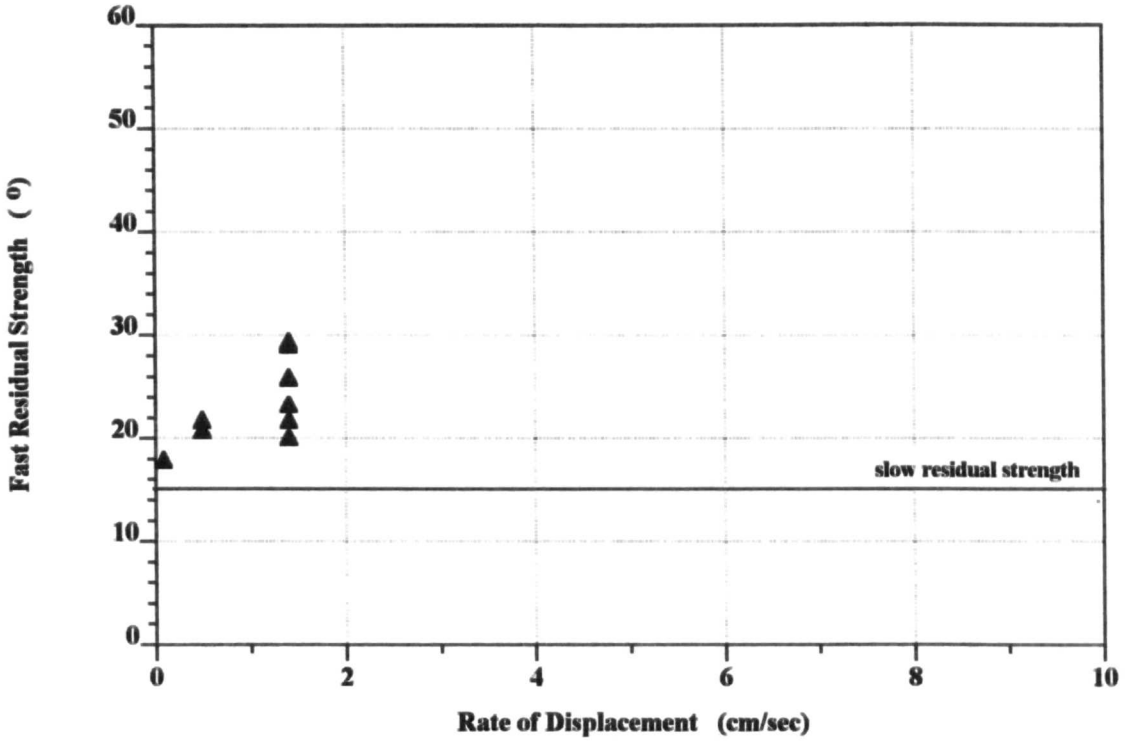


Fig. 6.3.14 Rate dependent characteristics of normally consolidated, non-submerged, brown London clay (Test IV.5)

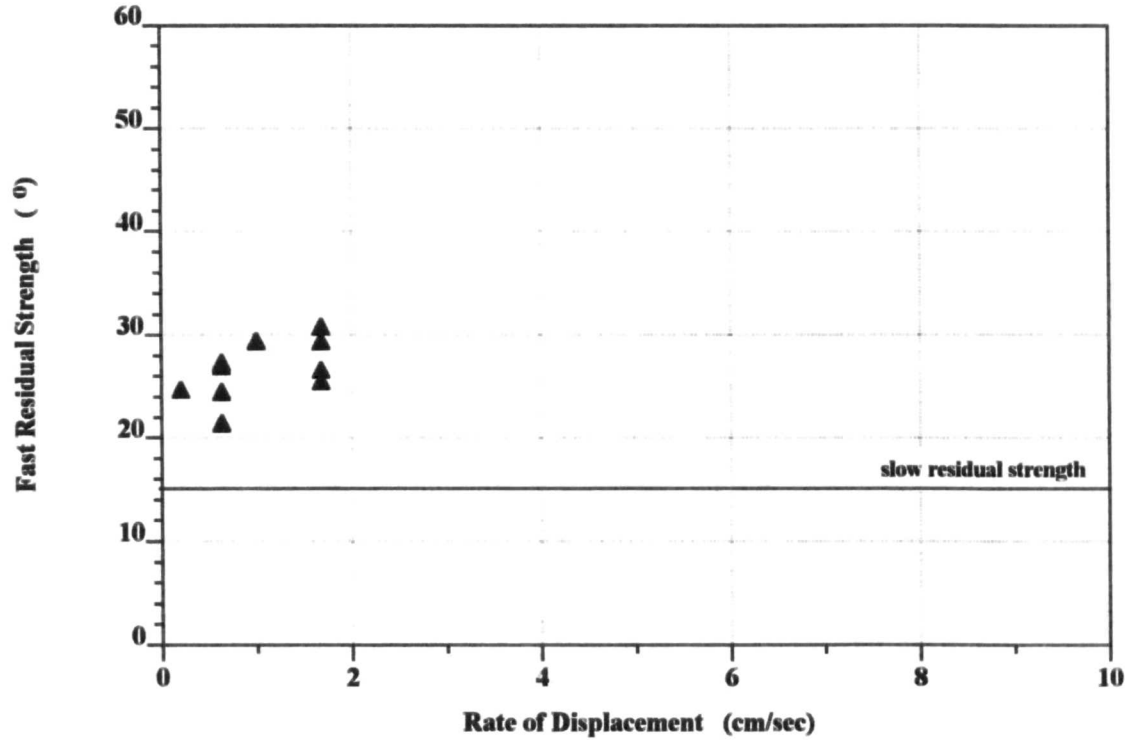


Fig. 6.3.15 Rate dependent characteristics of normally consolidated, submerged, brown London clay (50 kPa)

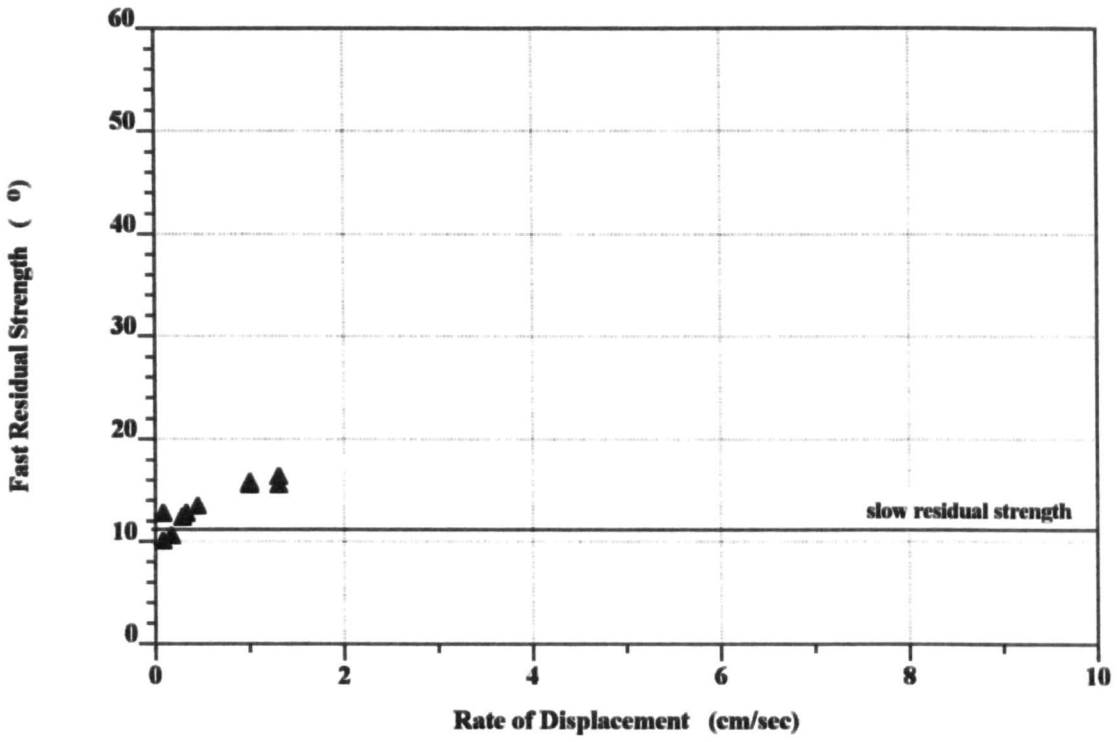


Fig. 6.3.16 Rate dependent characteristics of normally consolidated, submerged brown London clay (400 kPa)

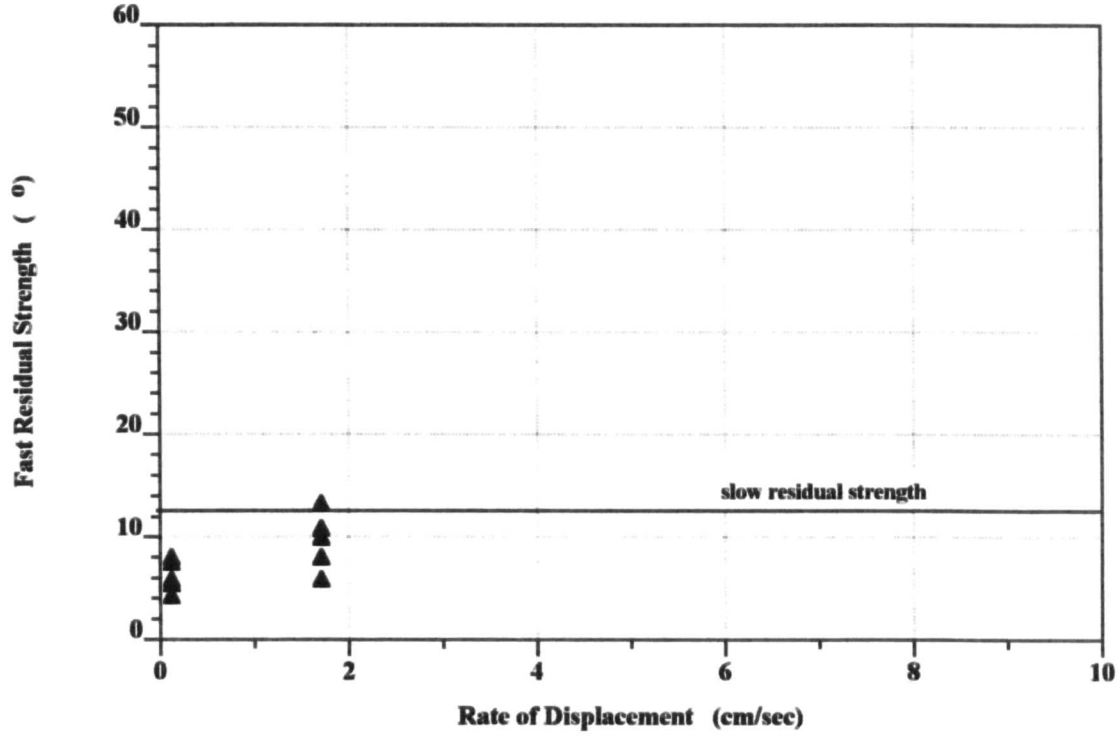


Fig. 6.3.17 Rate dependent characteristics of normally consolidated, submerged brown London clay (100 kPa)

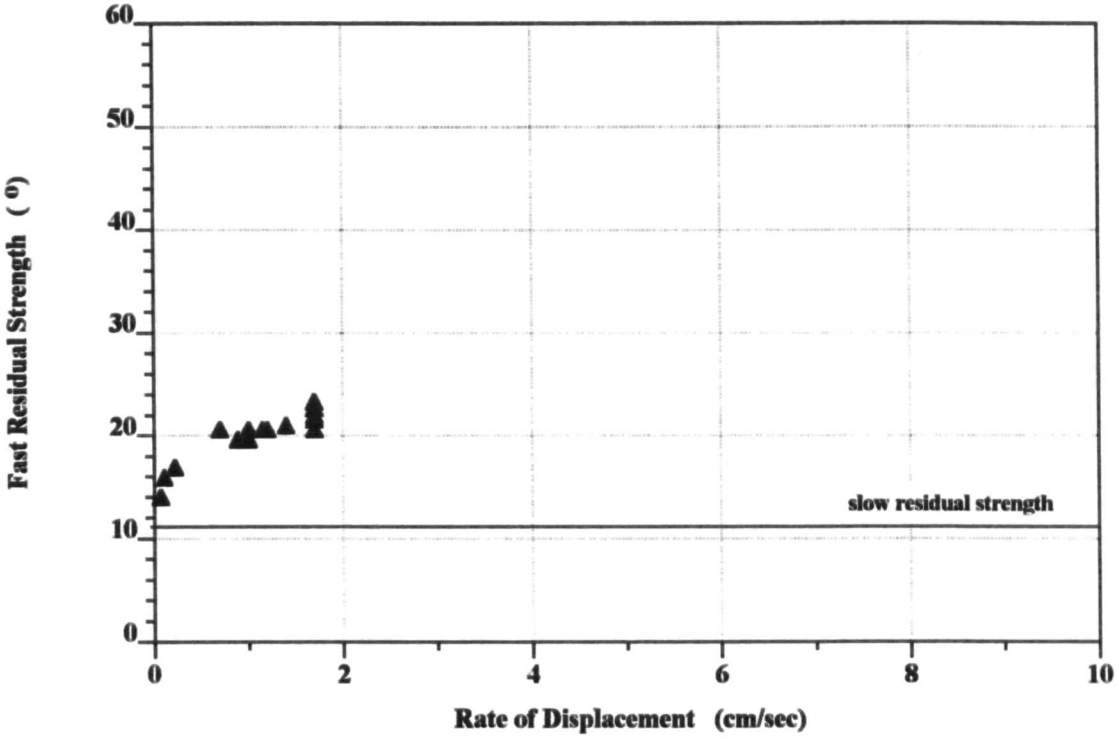


Fig. 6.3.18 Rate dependent characteristics of normally consolidated, submerged, brown London clay (200 kPa)

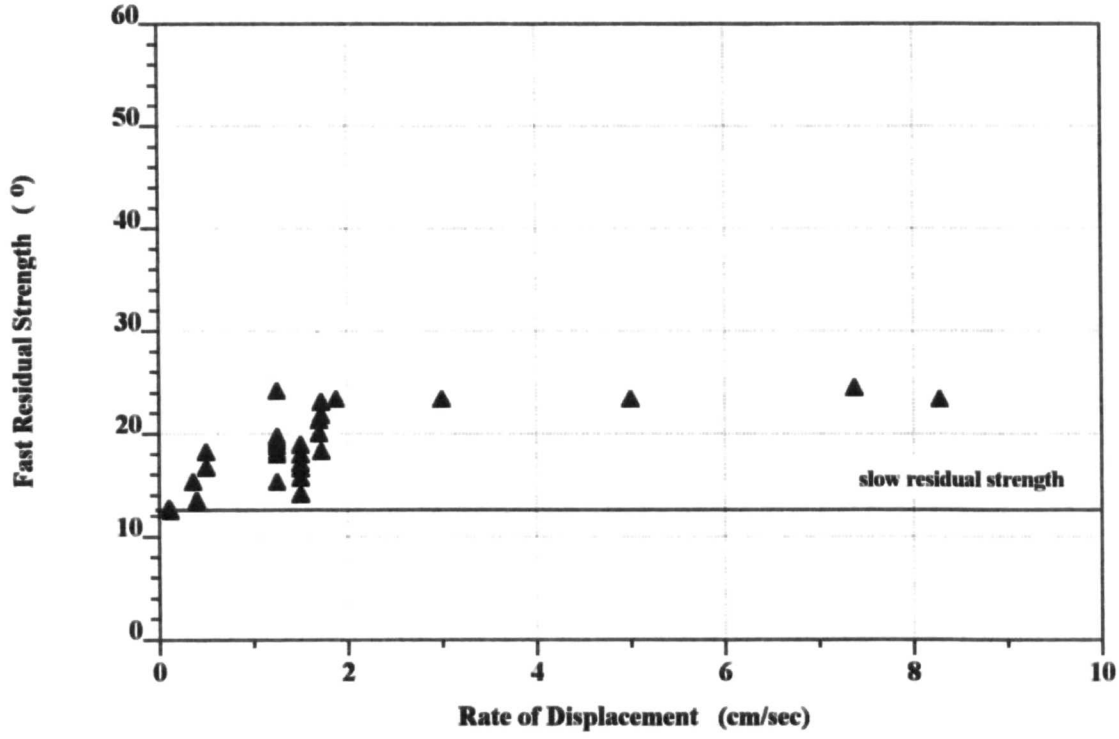


Fig. 6.3.19 Rate dependent characteristics of over-consolidated, submerged, brown London clay (3-stage test, 100 kPa)

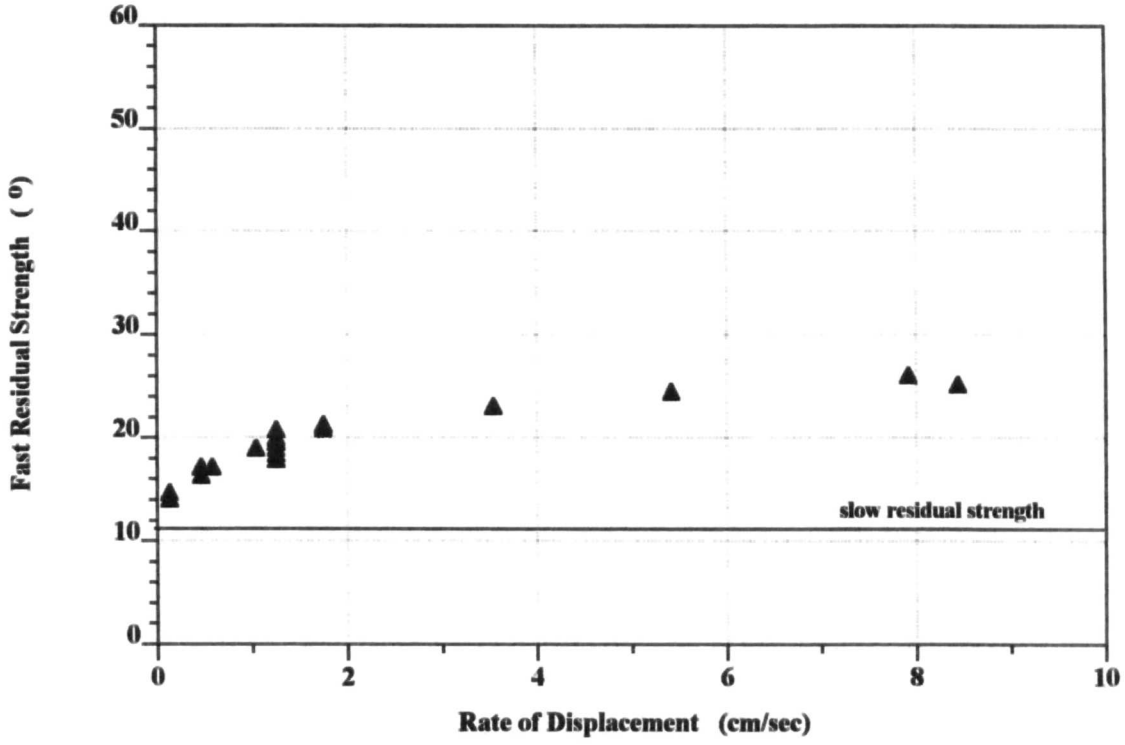


Fig. 6.3.20 Rate dependent characteristics of over-consolidated, submerged, brown London clay (3-stage test, 200 kPa)

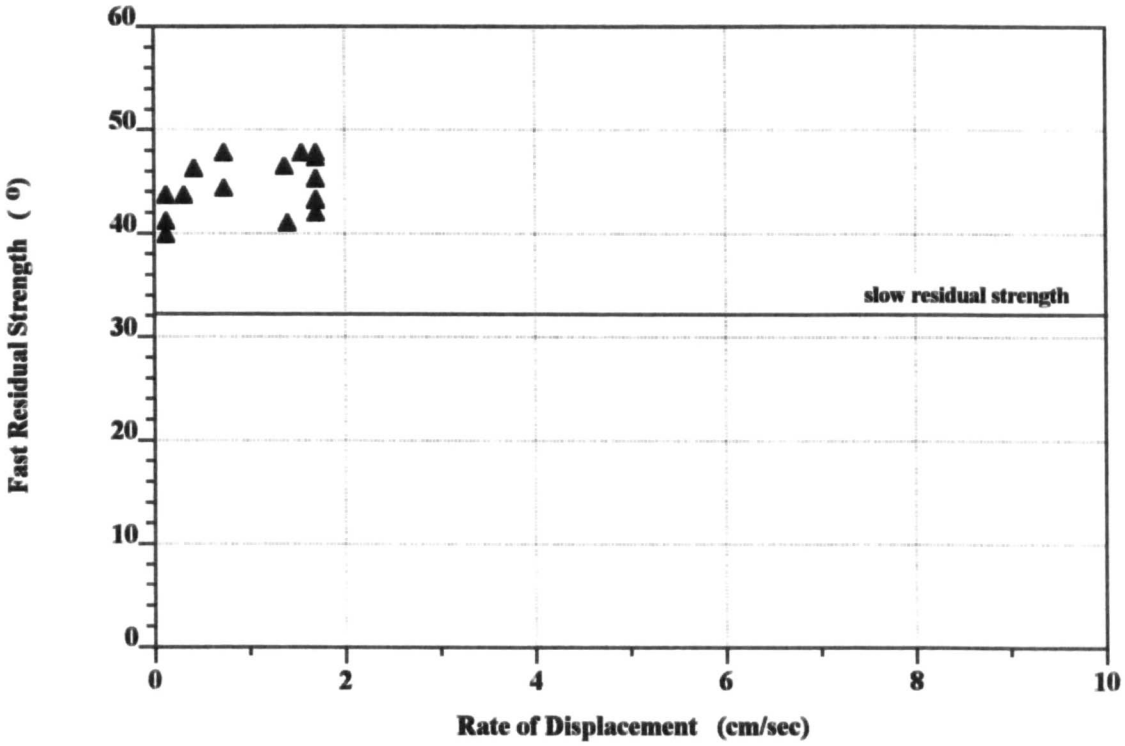


Fig. 6.3.21 Rate dependent characteristics of normally consolidated, non-submerged, Cowden till (50 kPa)

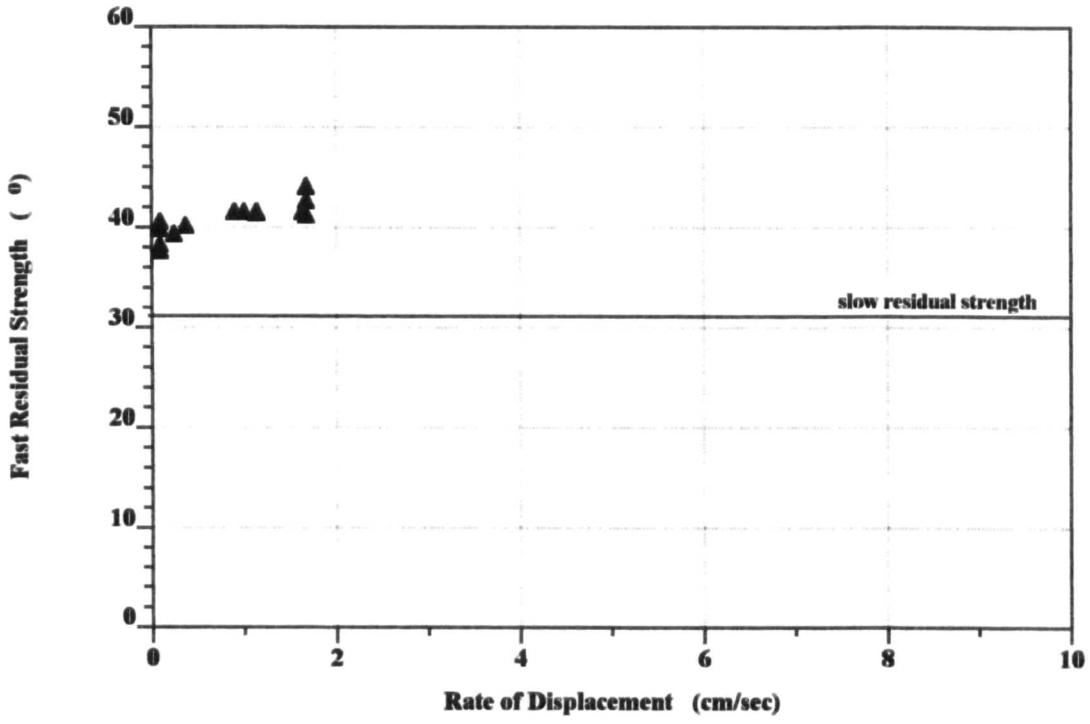


Fig. 6.3.22 Rate dependent characteristics of normally consolidated, non-submerged, Cowden till (100 kPa)

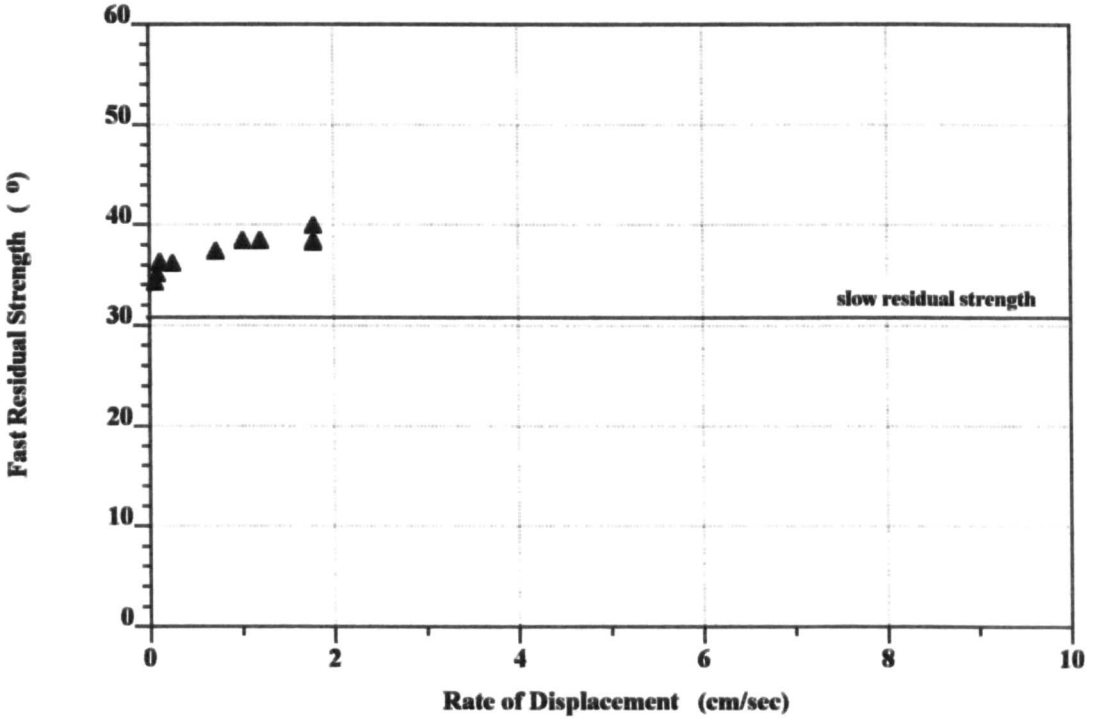


Fig. 6.3.23 Rate dependent characteristics of normally consolidated, non-submerged, Cowden till (200 kPa)

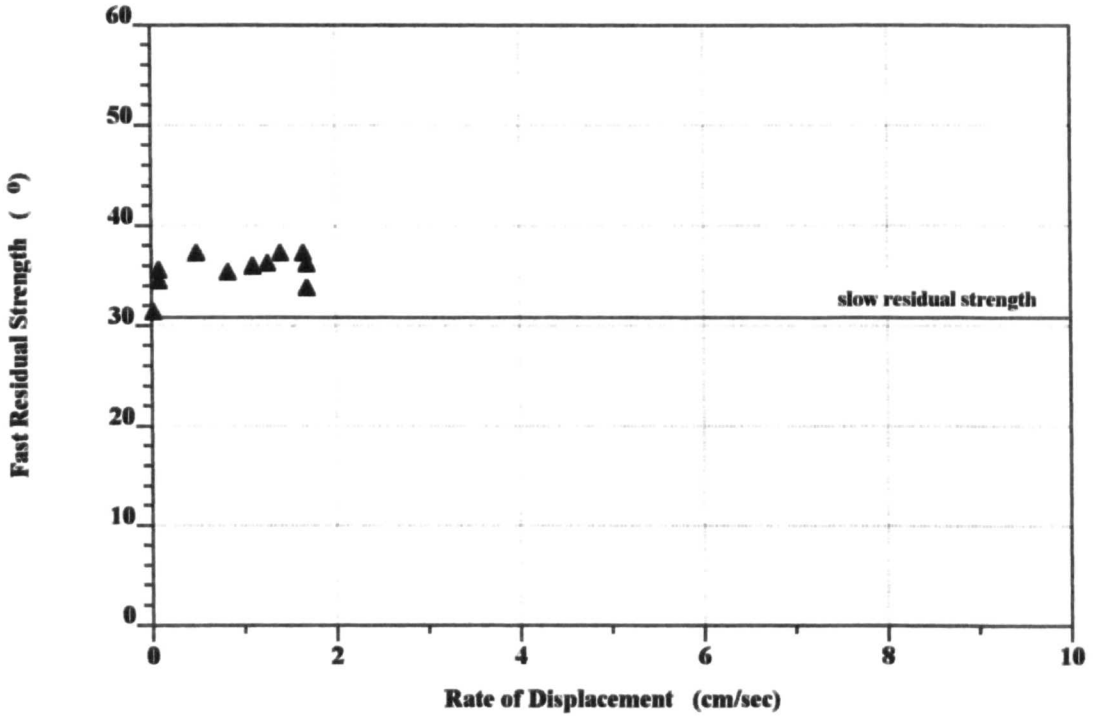


Fig. 6.3.24 Rate dependent characteristics of normally consolidated, non-submerged, Cowden till (400 kPa)

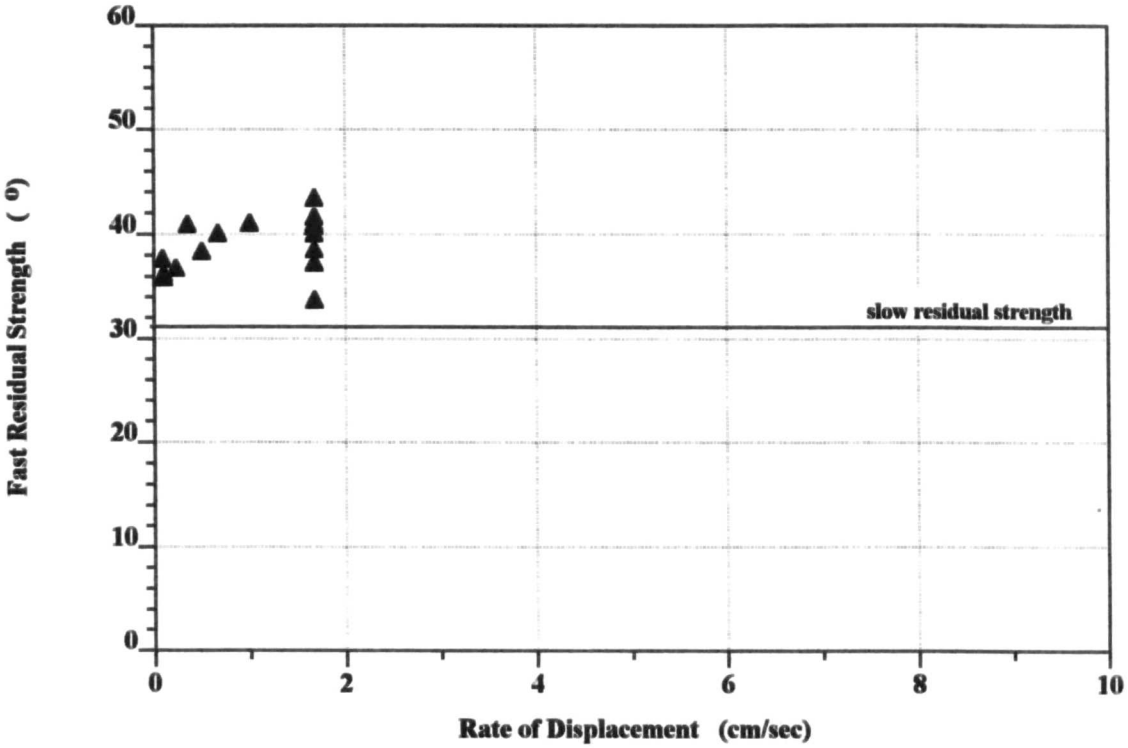


Fig. 6.3.25 Rate dependent characteristics of over-consolidated, submerged, Cowden till (100 kPa)

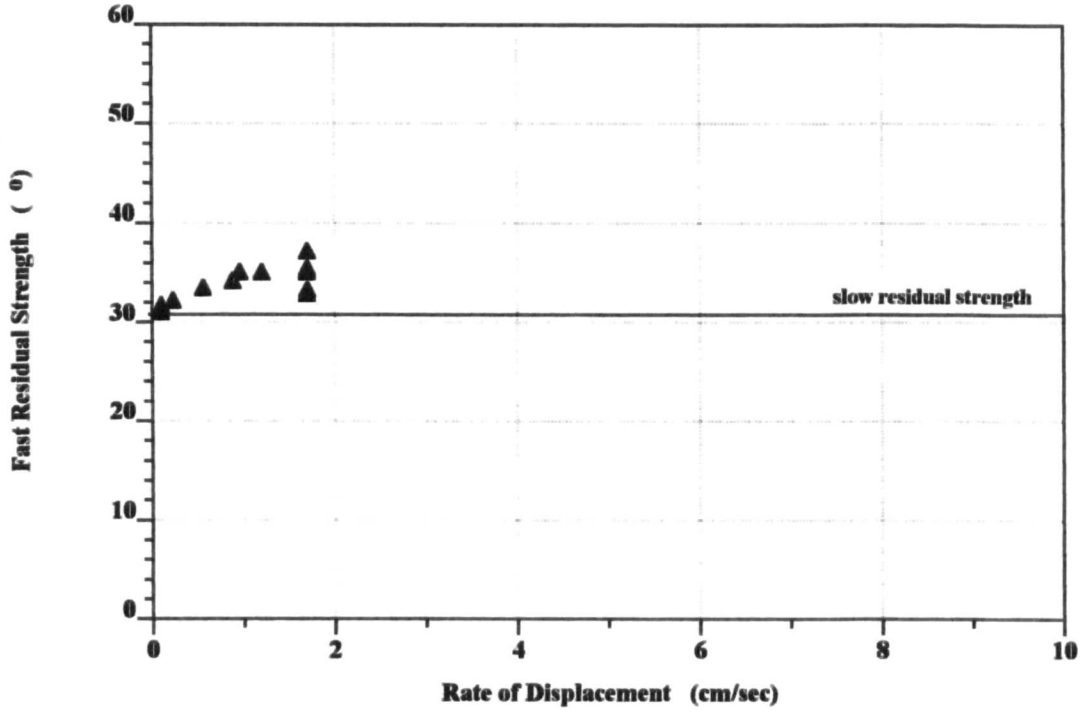


Fig. 6.3.26 Rate dependent characteristics of over-consolidated, submerged, Cowden till (200 kPa)

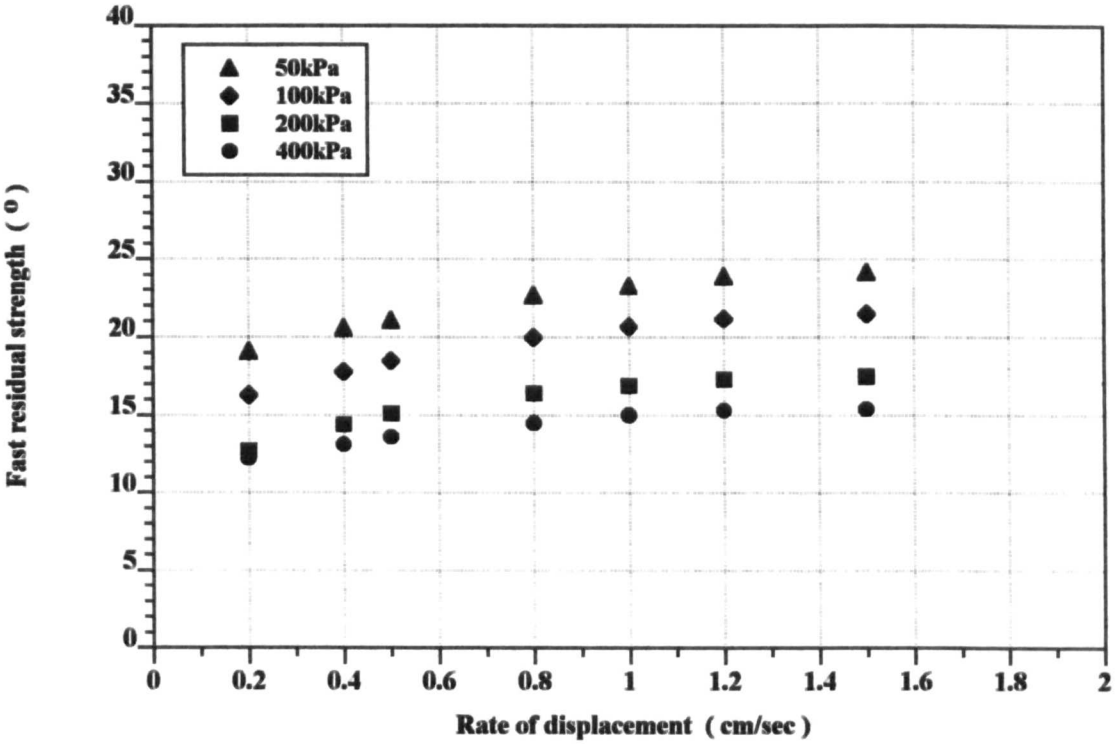


Fig. 6.3.27 Normal stress effect on the fast residual strength of normally consolidated, non-submerged brown London clay

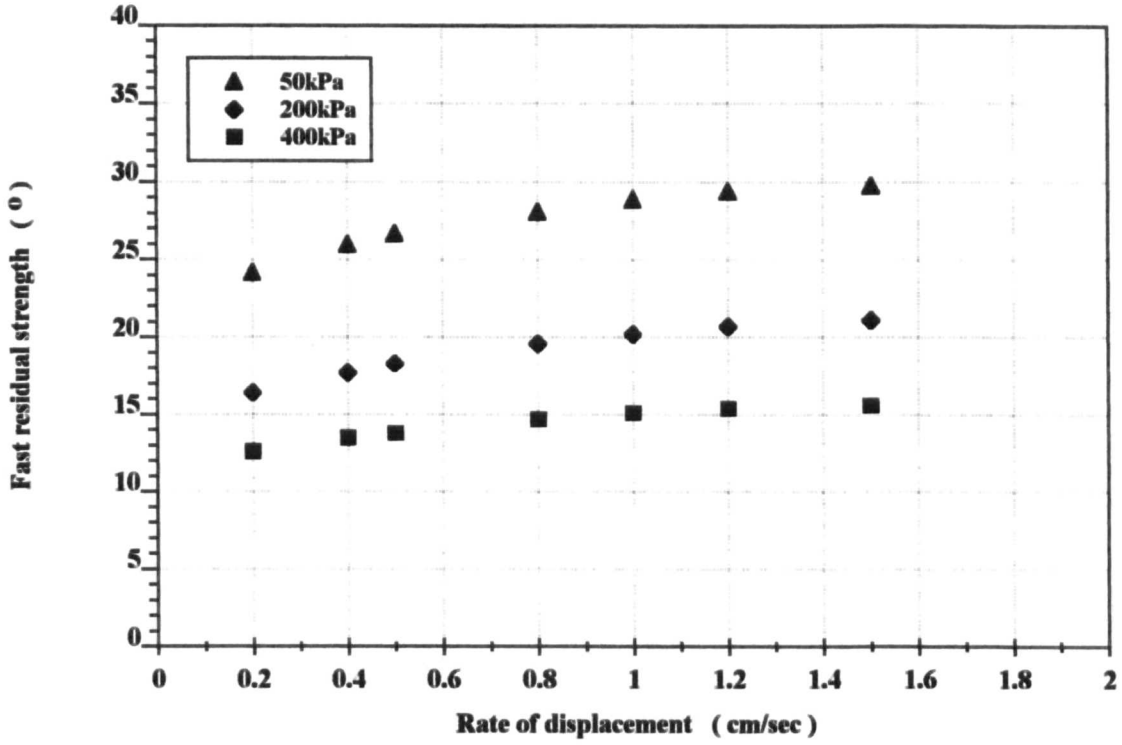


Fig. 6.3.28 Normal stress effect on the fast residual strength of normally consolidated, submerged brown London clay

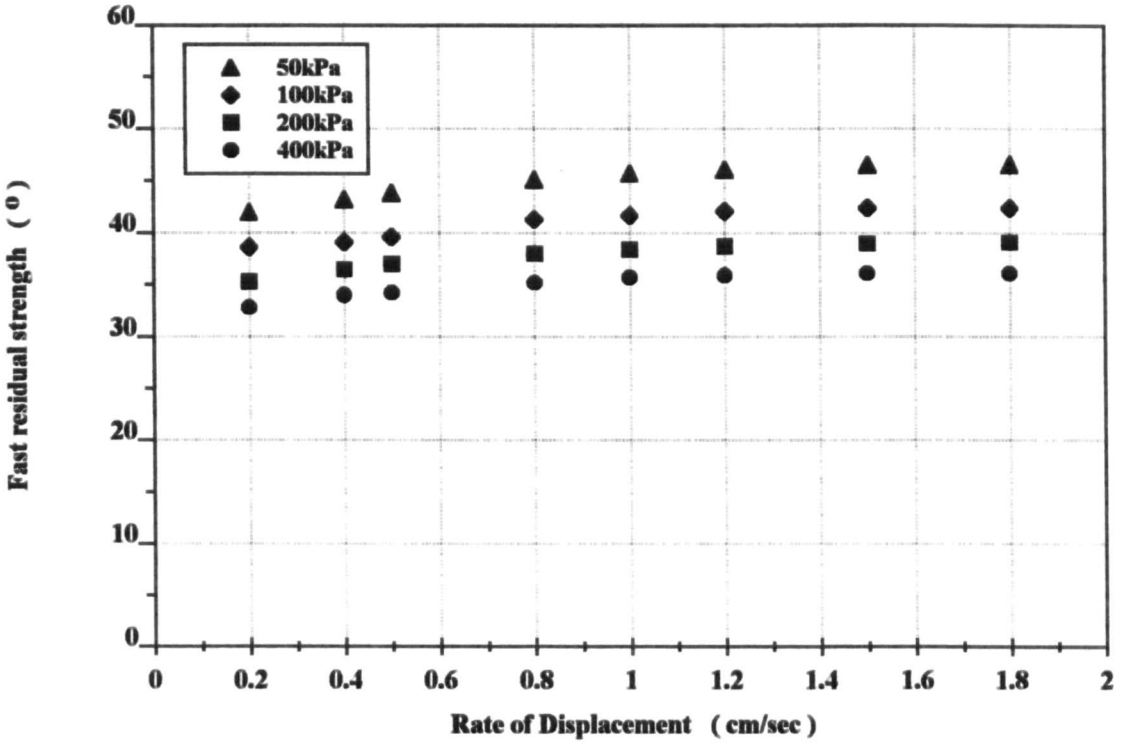


Fig. 6.3.29 Normal stress effect on the fast residual strength of normally consolidated, non-submerged Cowden till

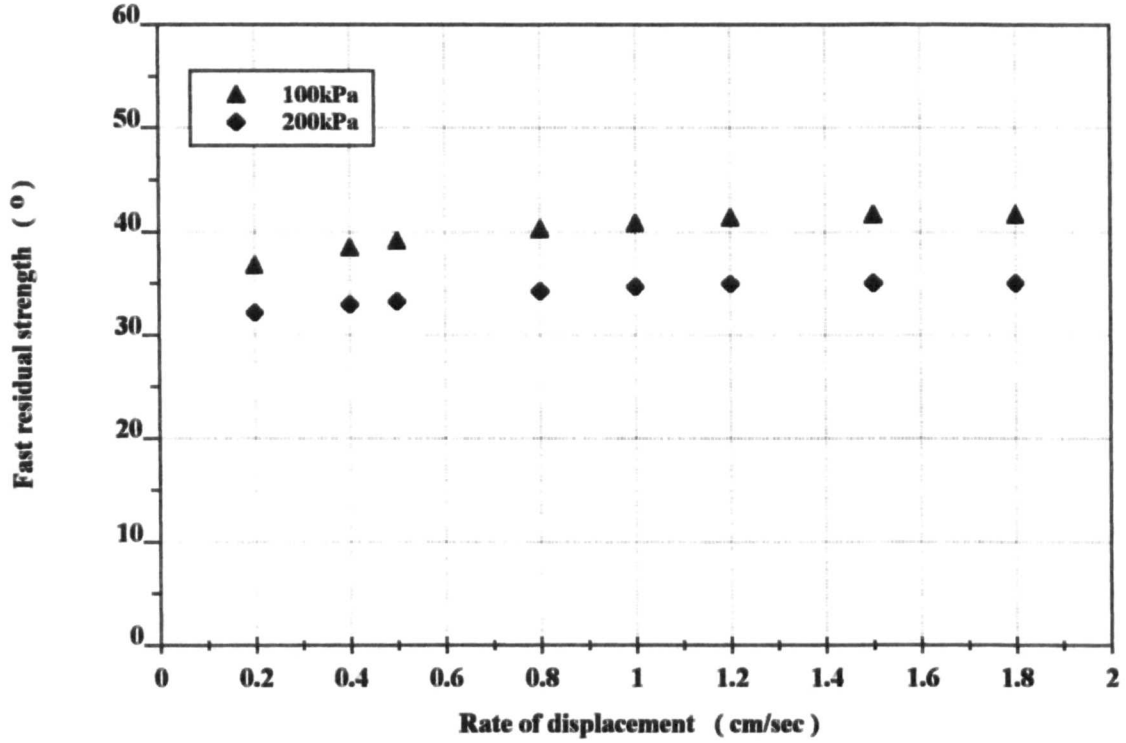


Fig. 6.3.30 Normal stress effect on the fast residual strength of over-consolidated, submerged Cowden till

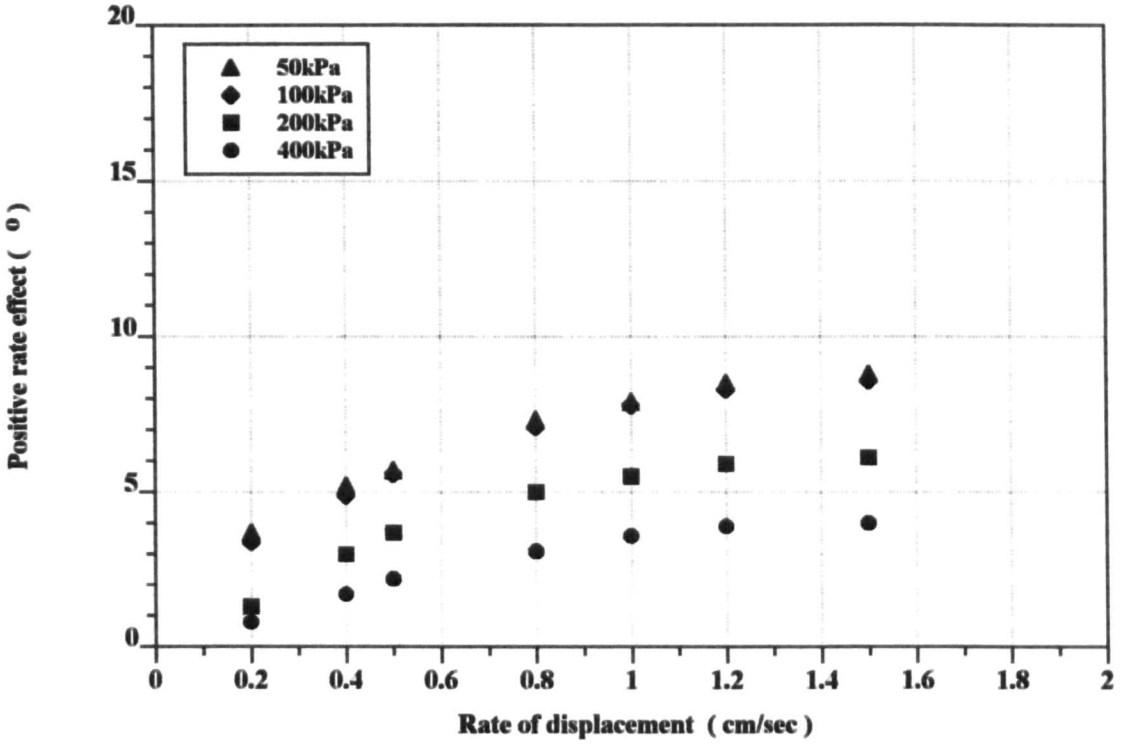


Fig. 6.3.31 Normal stress variations of the positive rate effect on normally consolidated, non-submerged brown London clay

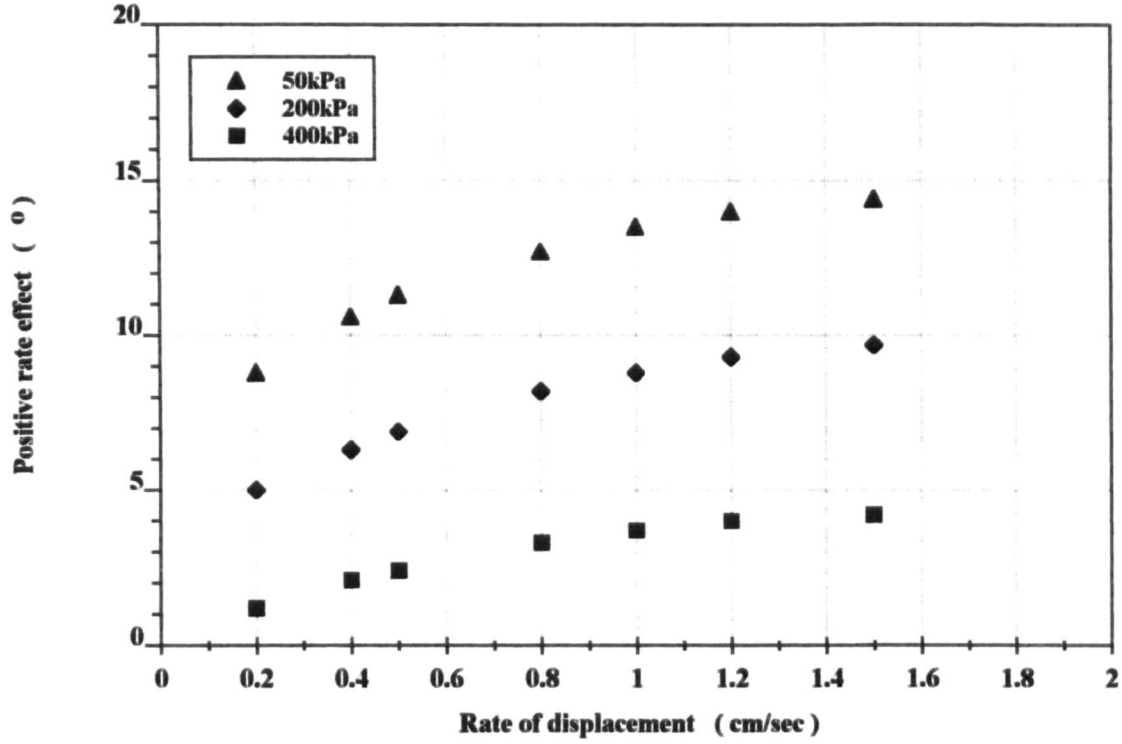


Fig. 6.3.32 Normal stress variations of the positive rate effect on normally consolidated, submerged brown London clay

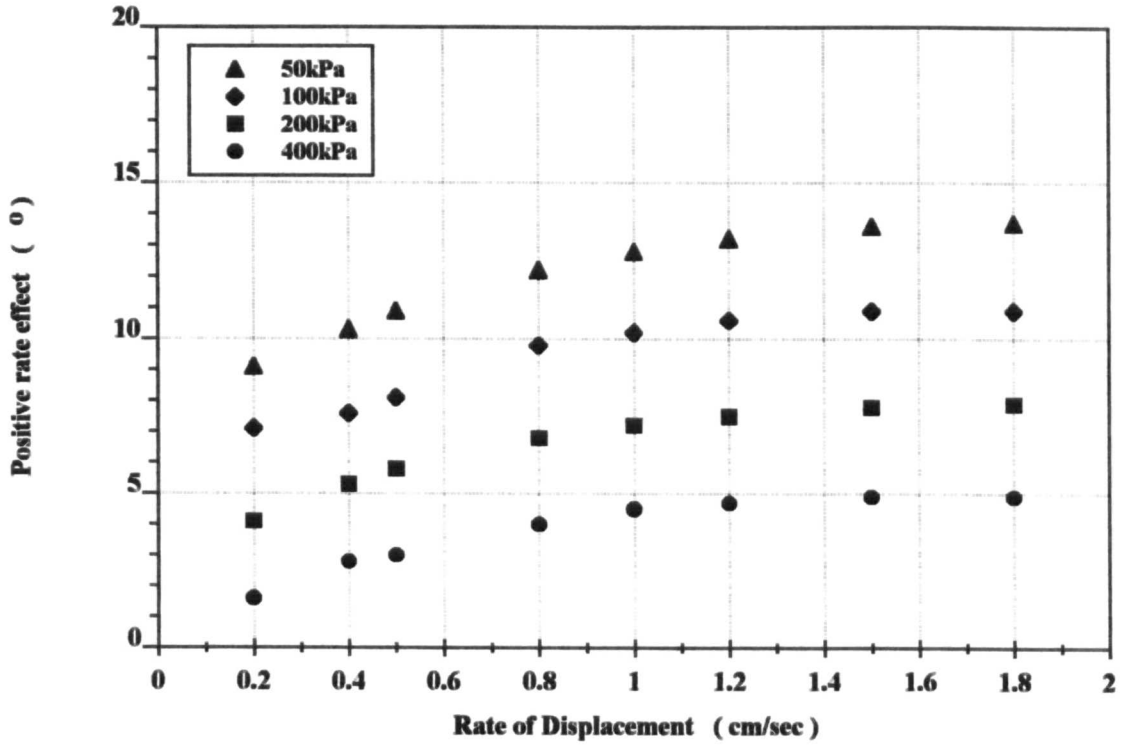


Fig. 6.3.33 Normal stress variations of the positive rate effect on normally consolidated, non-submerged Cowden till

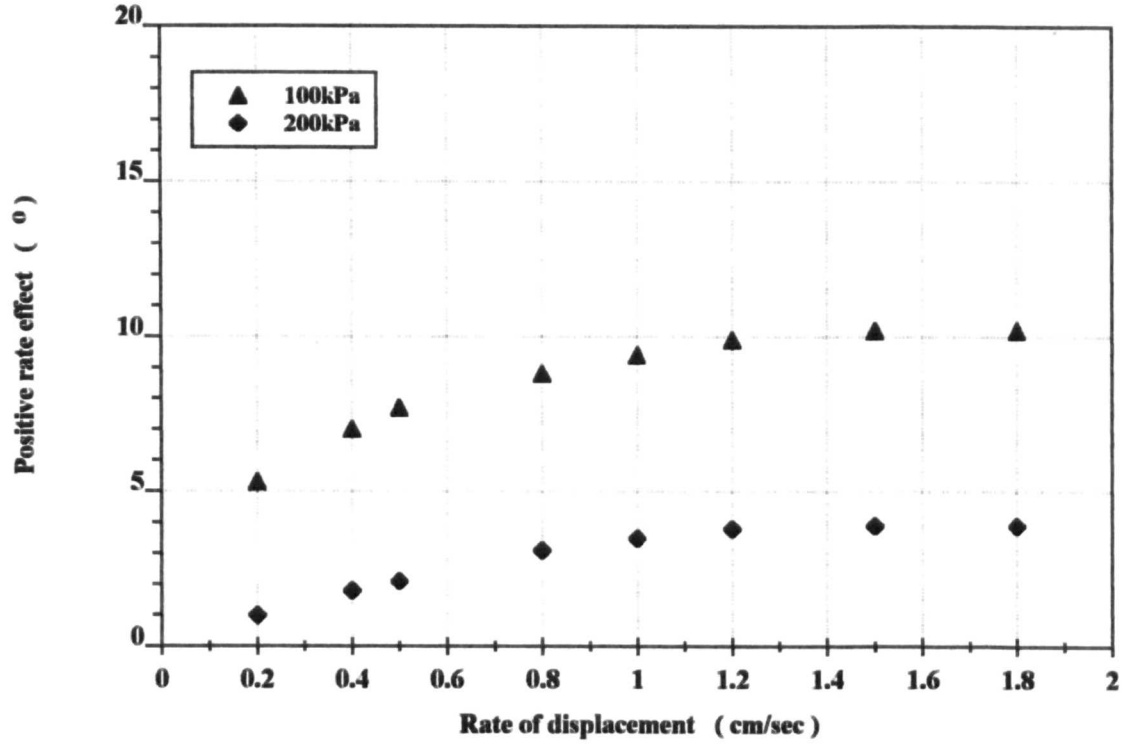


Fig. 6.3.34 Normal stress variations of the positive rate effect on over-consolidated, submerged Cowden till

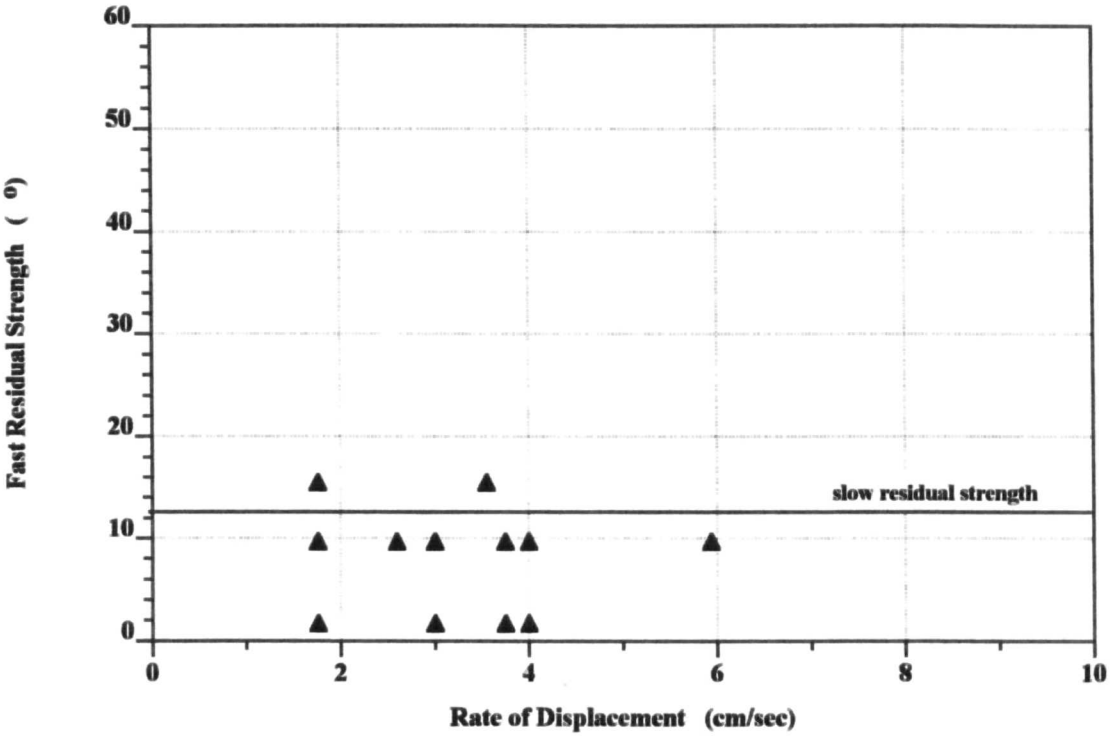


Fig. 6.3.35 Rate dependent characteristics of over-consolidated, submerged, brown London clay (100 kPa)

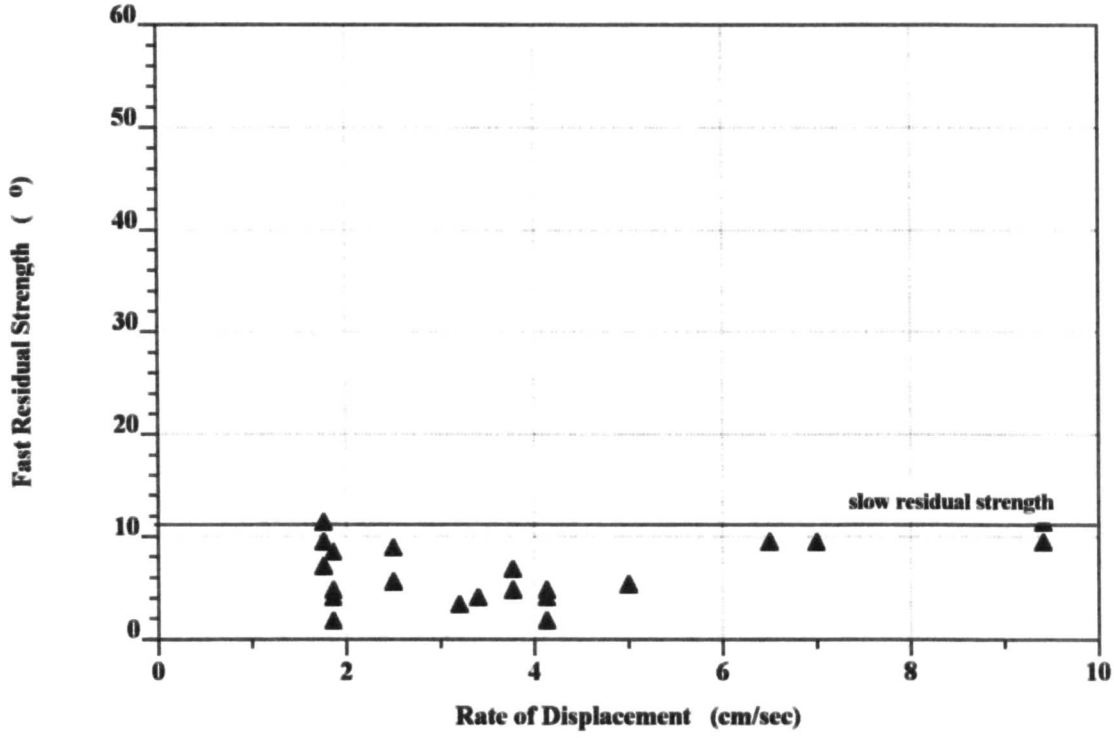


Fig. 6.3.36 Rate dependent characteristics of over-consolidated, submerged, brown London clay (200 kPa)

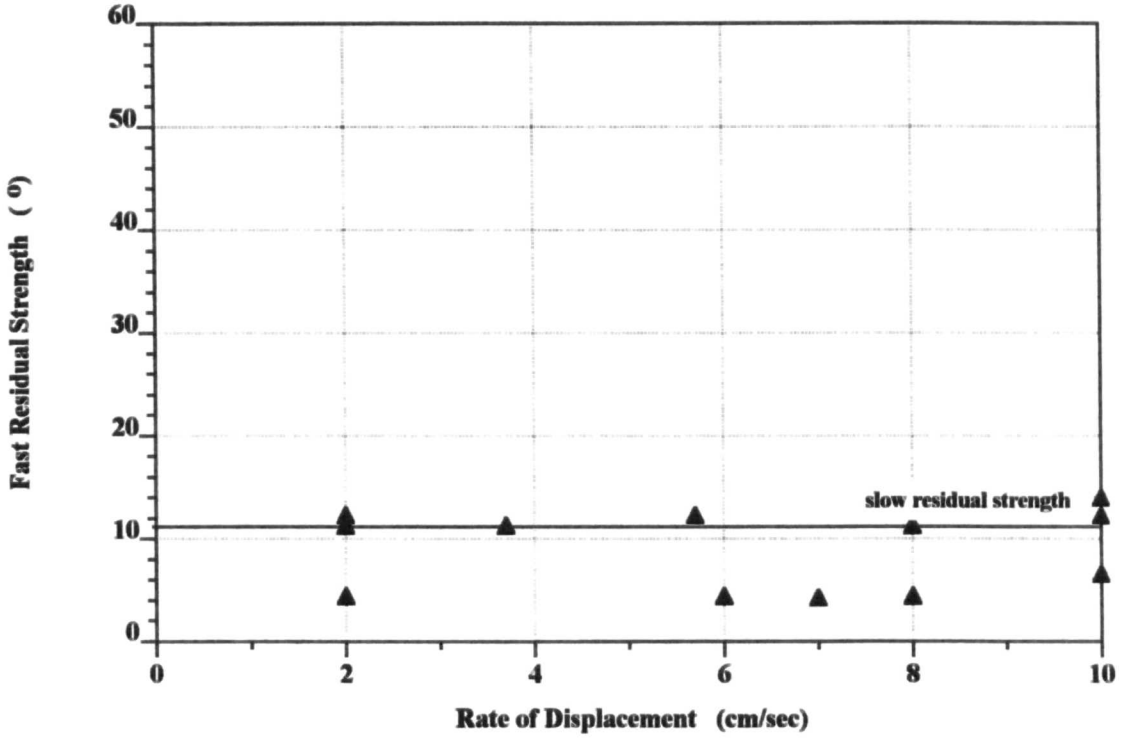


Fig. 6.3.37 Rate dependent characteristics of over-consolidated, submerged, brown London clay (300 kPa)

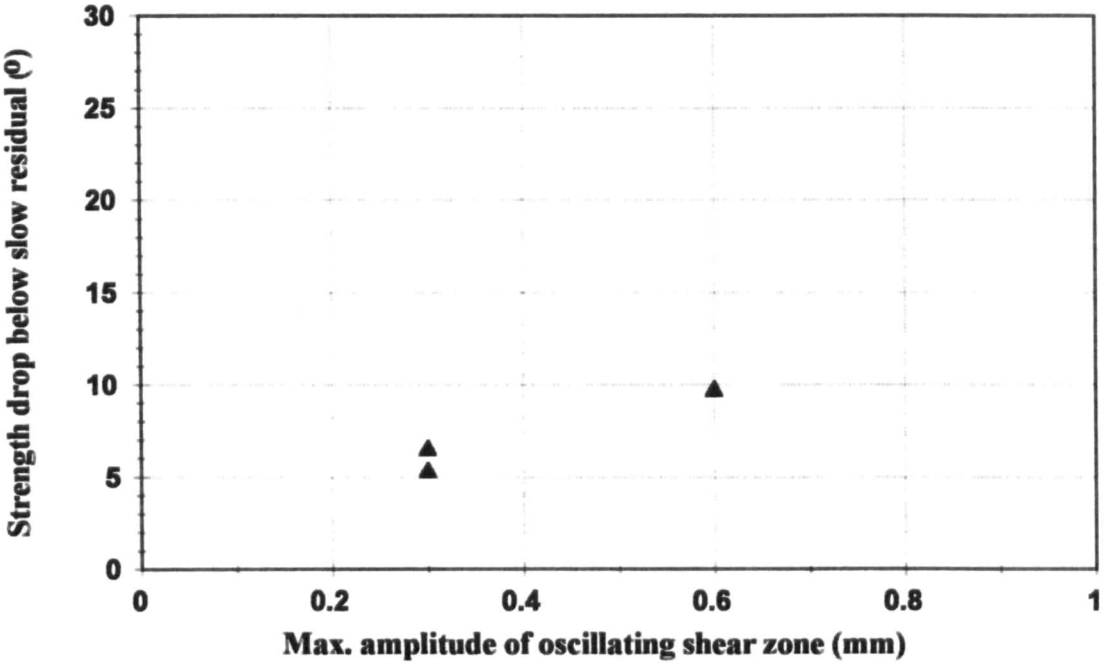


Fig. 6.3.38 Amplitude of oscillating shear zone effect on fast residual strength drop below slow residual strength for brown London clay

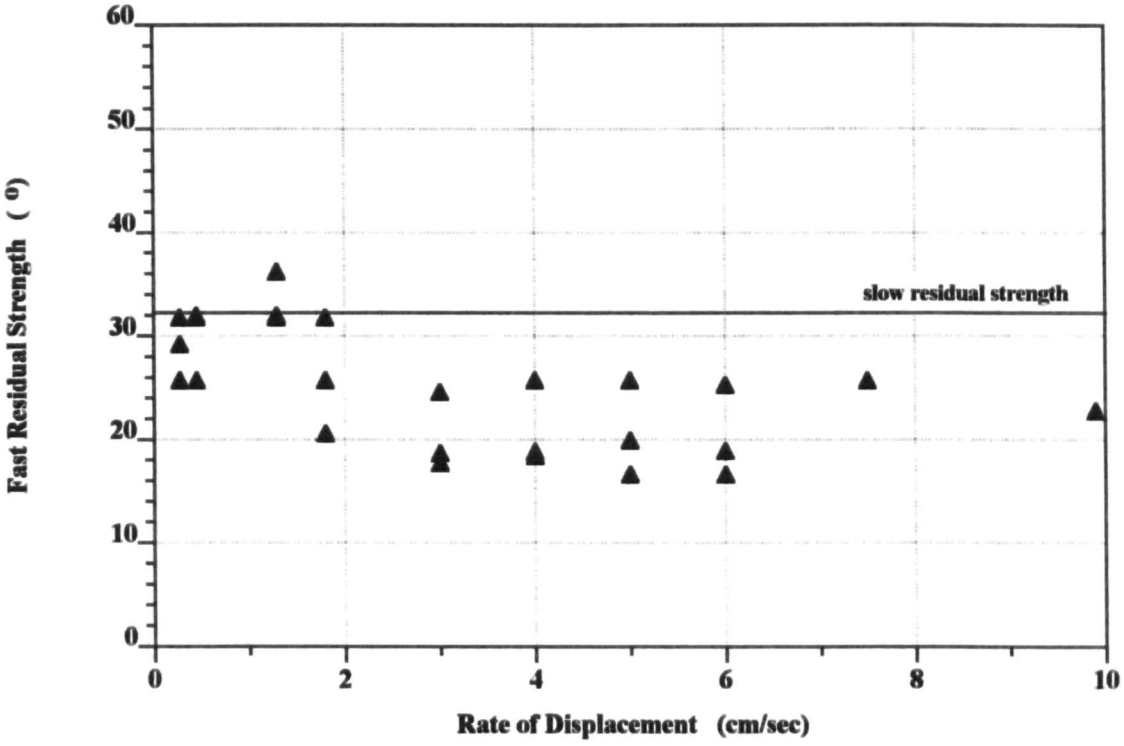


Fig. 6.3.39 Rate dependent characteristics of normally consolidated, submerged, Cowden till (50 kPa)

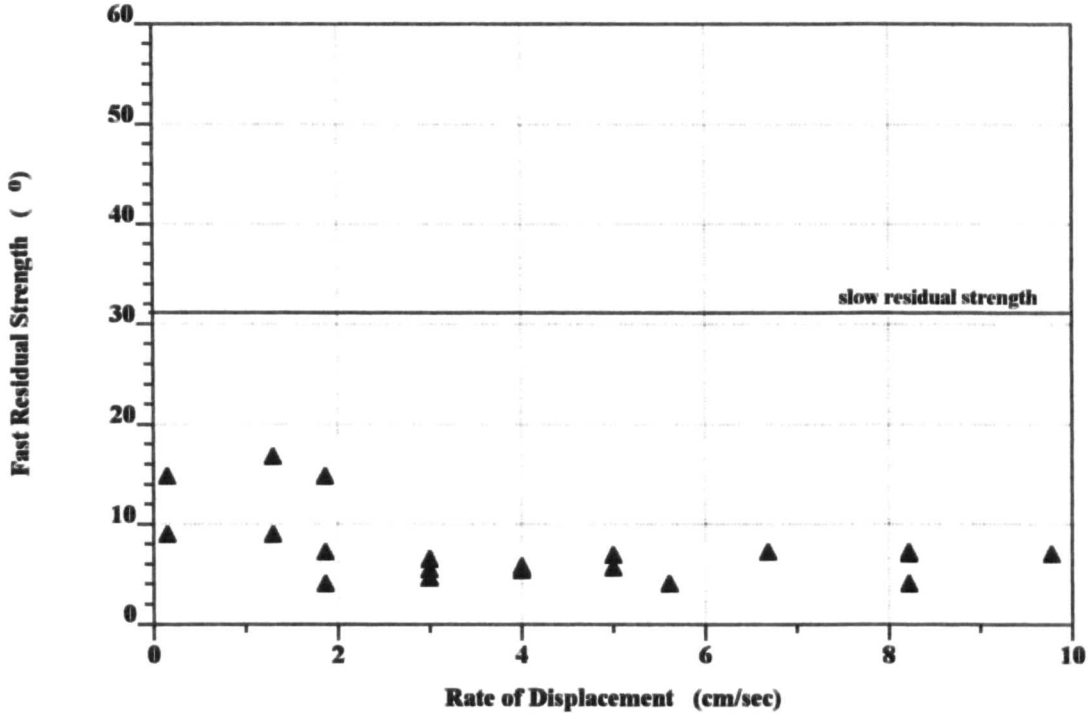


Fig. 6.3.40 Rate dependent characteristics of normally consolidated, submerged, Cowden till (100 kPa)

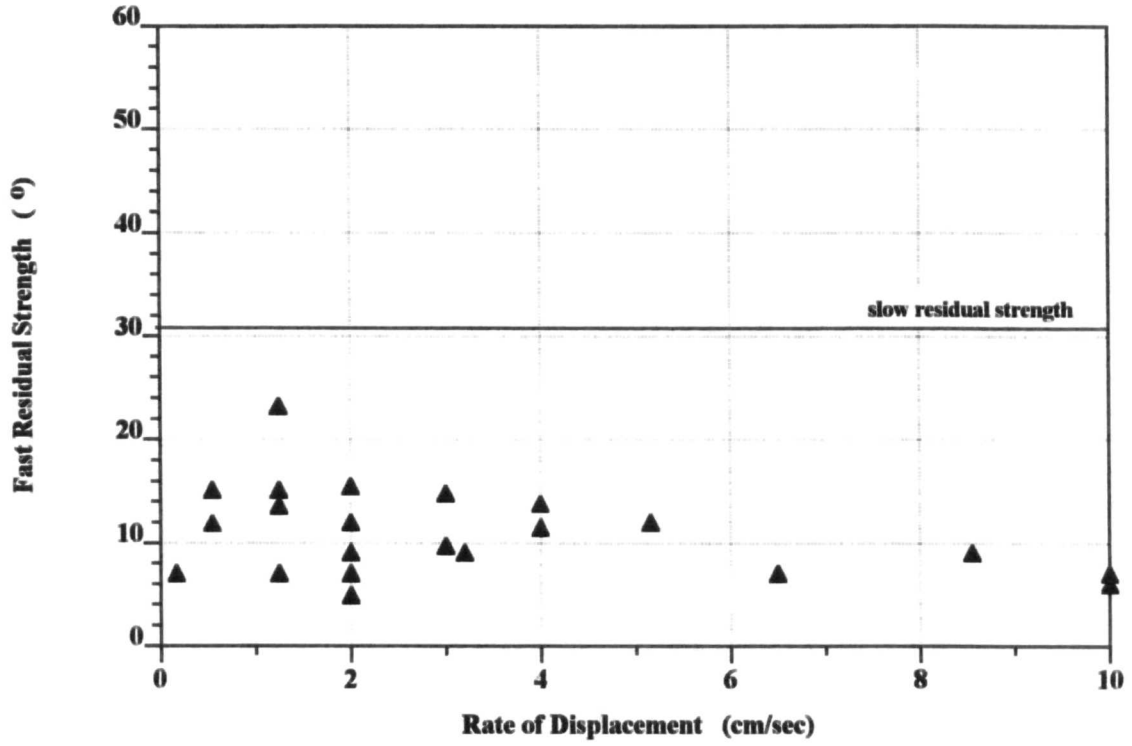


Fig. 6.3.41 Rate dependent characteristics of normally consolidated, submerged, Cowden till (200 kPa)

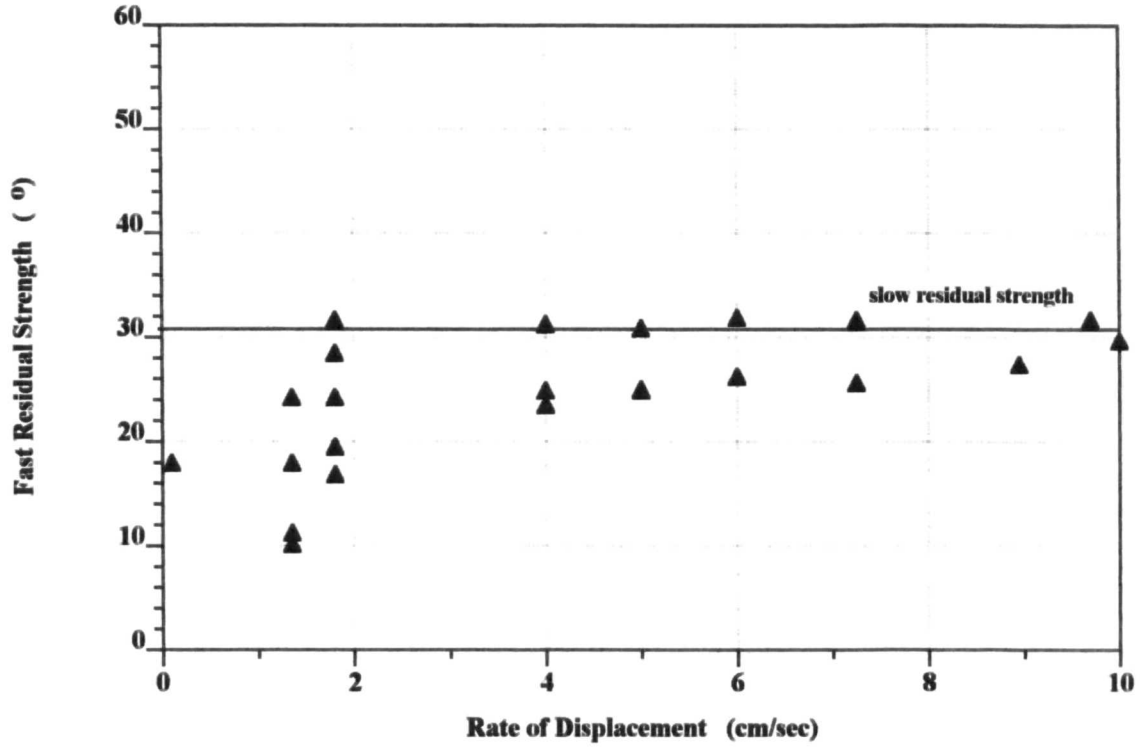


Fig. 6.3.42 Rate dependent characteristics of normally consolidated, submerged, Cowden till (300 kPa)

rate effects were observed as the rate of displacement was gradually varied. When the rate of displacement was decreased suddenly, the strength of the sample dropped accordingly.

TEST IV.4

σ_n= 400 kPa

$\phi_{residual}$= 12.4°

$\phi_{threshold}$= 21.3°

(Figures IV.4.1-2)

This sample had already been sheared at lower normal stresses, and thus the particle orientation associated with its residual state had been mainly completed. Immediately after the beginning of shearing at 0.09 mm/min the sample strength reached its slow peak value of 12.7°, and after 2.3 mm of shear displacement it reached its slow residual value of 12.4°. The test was continued for 3000 mins and the sample was sheared for 27 cm in order to observe its behaviour after more than half rotation at a drained rate of displacement and also to evaluate the effect which the contamination of the rings and the gap friction have on the measured strength. The initial 0.02 mm of decrease in sample height should be attributed to both sample loss and particle orientation as the sample approached its state of residual. However, the further 0.43 mm decrease in sample height, before this slow shearing part of the test was terminated, should be attributed to sample lost through the gap.

The rate of displacement was increased to 0.037 cm/sec for the next 2.9 cm of shearing while the measured strength increased to 13.9° and remained constant until the rate of displacement was suddenly increased to 1.32 cm/sec. Following this further increase in the rate of displacement, the strength reached its threshold value of 21.3° before it started to decrease while the sample dilated. After 28.4 cm of shearing the sample had dilated to 0.52 mm above its height at the end of the slow shearing stage and its strength had dropped to 12.5°. Further shearing caused a decrease in sample height, either due to sample contraction or soil loss, and after a full rotation of the former had decreased to its level at the end of the slow shearing part of this test and the measured strength had increased to 15.6°. For the next 55 cm of shearing at 1.32 cm/sec the strength of the sample remained nearly steady. When the rate of displacement was decreased gradually to 0.13 cm/sec, the strength of the sample dropped to 7.2°. The sample exhibited positive rate characteristics when the rate of

$\phi_{residual}$= 47.2°

$\phi_{threshold}$= 68.3°

(Figures V.7.1-2)

Contamination of the rings with soil which had been extruded through the gap during the previous stages of this procedure apparently affected the shape of the stress ratio curve for the slow shearing part of this test. Nevertheless, the measured strength reached its slow residual value of 47.2° after 57.5 mm of shearing while the sample height increased by 0.086 mm. The slow shearing part of this test was terminated after a displacement of 117.8 mm .

The rate of displacement was increased suddenly to 1.0 cm/sec and the strength reached immediately its threshold value of 68.3°. While the rate of displacement continued to be increased until it reached the level of 1.25 cm/sec, the strength decreased to 46.7° after 20 cm of shearing. Further shearing at the same rate of displacement led to further decrease in strength which had stabilised between 42.5° and 44.9° before the sample had been sheared for 226.9 cm at rates which ranged between 0.12 cm/sec and 1.25 cm/sec. At that point, the rate of displacement was increased suddenly to 1.82 cm/sec and the measured strength reached the value of 51.3° at zero associated displacement and then decreased to 44.9°. The rate of displacement was varied, gradually and suddenly, from 1.82 cm/sec to 6.08 cm/sec and 9.5 cm/sec before the test was terminated, but no effect on the strength of the sample was observed. The test was terminated after 640 cm of shearing.

Overconsolidated, Submerged

Test V.8

σ_n= 50 kPa

OCR= 8

$\phi_{residual}$= 49.5°

$\phi_{threshold}$= 53.8°

(Figures V.8.1-2)

This sample was consolidated to 400 kPa and unloaded to 50 kPa before it was sheared at 0.09 mm/min. The measured strength reached its slow peak value of 55.8° after 3.14 cm of shearing while the sample height increased by 0.22 mm. The sample continued to dilate as its structure relaxed and the strength reached its

measured strength increased to 23.3° at zero associated displacement. Throughout the remaining part of this test the rate of displacement was varied, gradually or suddenly, from 1.74 cm/sec to 10.0 cm/sec, but no effect on the strength of the sample was observed.

Overconsolidated, Submerged

Test VI.14

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 16.6°

$\phi_{threshold}$= 29.3°

(Figures VI.14.1-2)

The sample which was used in the previously described test was consolidated to 400 kPa and then unloaded to 200 kPa before it was sheared at 0.09 mm/min. The measured strength reached its slow peak value of 22.8° while the sample appeared to be dilating. The sample height increased to 0.0425 mm, and the measured strength started to decrease. After the sample had been sheared for 223.2 mm, its height had dropped back to its initial level and continued to decrease because of sample loss while its measured strength had reached its residual value of 16.6°. This slow shearing part of the test was terminated after the sample had been sheared for 399.0 mm.

The rate of displacement was increased suddenly to 1.30 cm/sec and the measured strength reached immediately its threshold value of 29.3°. Shearing at the same rate of displacement was continued until the sample had been sheared for 94.8 cm while the measured strength had stabilised within a range between 28.8° and 30.5°. The rate of displacement was then decreased gradually from 1.30 cm/sec, to 0.069 cm/sec, and to 0.057 cm/sec while the strength of the sample also decreased from 29.3°, to 24.3°, and to 21.6° respectively. After the sample had been sheared for 113.4 cm, the rate of displacement was suddenly increased to 1.30 cm/sec and the strength also increased and stabilised within a range between 30.3° and 31.9°. However, no significant increase in strength was observed when the sample was sheared at rates of displacement higher than 1.0 cm/sec. After the sample had been sheared for 223.3 cm, the rate of displacement was decreased from 1.30 cm/sec, to 0.16 cm/sec, and to 0.1 cm/sec and the strength appeared to decrease from 31.9°, to 25.4°, and to 24.3°, respectively. The rate of displacement was gradually increased to

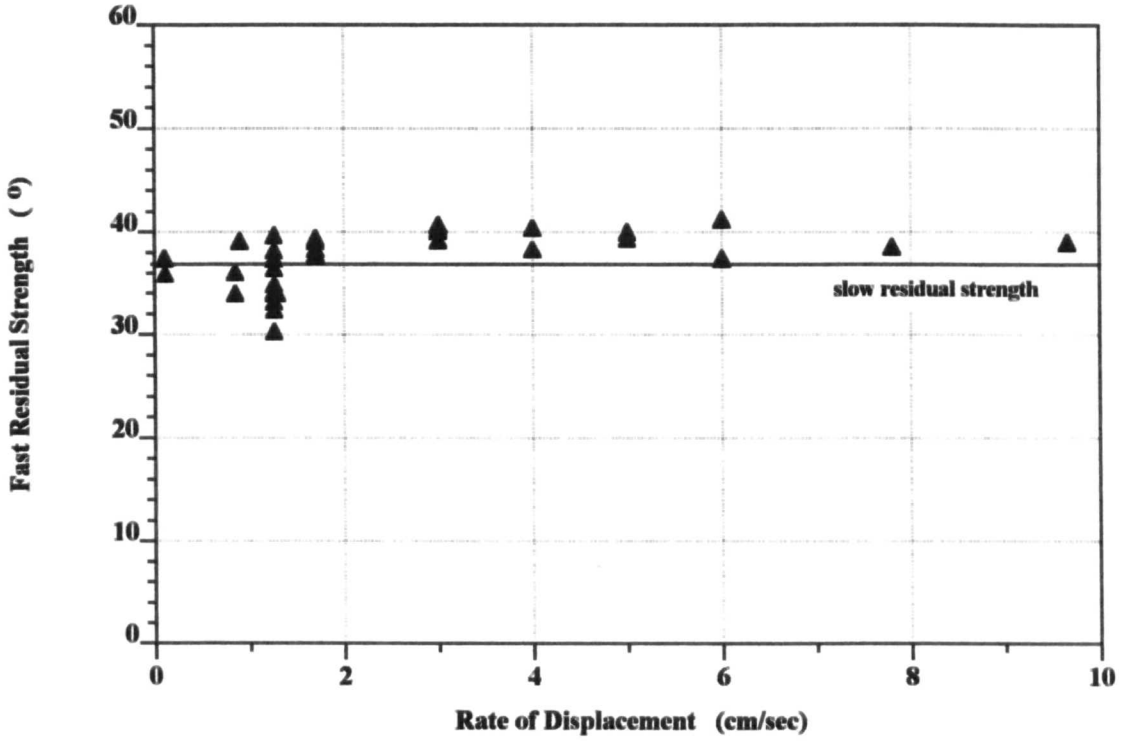


Fig. 6.3.51 Rate dependent characteristics of normally consolidated, completely dry, brown London clay (100 kPa)

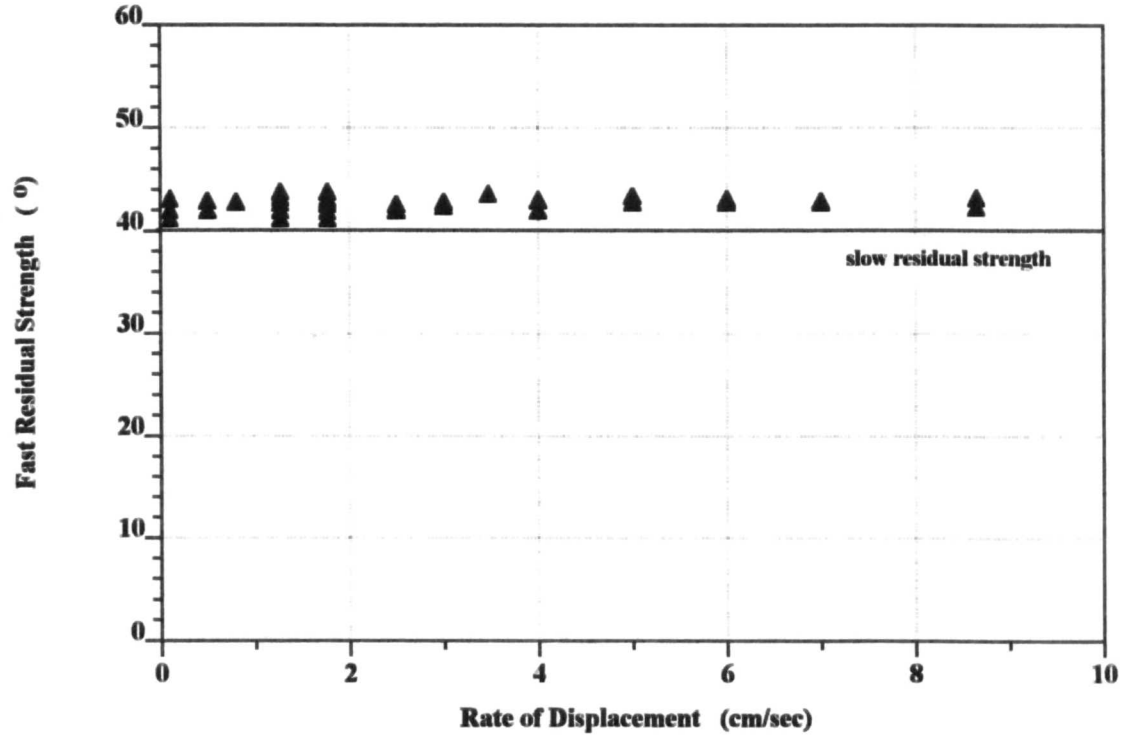


Fig. 6.3.52 Rate dependent characteristics of normally consolidated, completely dry, brown London clay (200 kPa)

formed with an amplitude of 0.3 mm and a wave-length of 39.4 cm. Apparently, pore pressures induced by the shape of the shear surface led to a decrease of the strength to 5.9° after 116.2 cm of cumulative shearing. When the rate of displacement was decreased to 0.12 cm/sec for the following 38.2 cm, the measured strength appeared to be increasing to 7.58°, probably because of the pore pressure dissipation. The rate of displacement was again increased to 1.71 cm/sec and the measured strength also increased to 10° for the next 28.2 cm of shearing but then decreased and stabilised at 8.05° until the test was terminated after 350 cm of cumulative displacement.

TEST IV.9

σ_n= 200 kPa

$\phi_{residual}$= 11.9°

$\phi_{threshold}$= 16.1°

(Figures IV.9.1-2)

The fact that this sample had been sheared at the previous normal stress of 100 kPa was reflected on its stress ratio/displacement curve when it was sheared to residual under 200 kPa normal stress. After 0.95 mm of shearing the sample had reached its slow peak strength of 16.7°, and, as shearing continued, the strength dropped until it had reached its slow residual value of 11.9° after a displacement of 156.8 mm. Due to particle reorientation and soil loss through the gap the sample height after 156.8 mm of shearing had decreased by 0.35 mm. Shearing at 0.09 mm/min was terminated after a displacement of 266 mm at which point the sample height had decreased further by 0.12 mm probably solely due to soil loss.

The rate of displacement was increased to 0.11 cm/sec for the next 7.26 cm of shearing. Upon the first application of fast shearing the strength of the sample increased to 16.1° at zero associated displacement, and, while approaching 7.26 cm of displacement, decreased to 14.0° as the sample height appeared to be increasing by 0.029 mm. The rate of displacement was increased further to 1.70 cm/sec for the following 26.44 cm of shearing and this was associated with an immediate increase in strength to 23.8°. After the rate of displacement had stabilised at 1.70 cm/sec the measured strength of the sample also stabilised at 22.7°. When the rate of displacement was decreased to 0.11 cm/sec, after 97.3 cm of shearing, the strength dropped to 15.9°. There were four cycles of increasing/decreasing of the rate of displacement in this test which eventually led to a shear displacement of 410 cm

while the sample height dropped due to sample loss. The strength fluctuated between 41.1° and 45° until the cumulative shear displacement had reached 150 cm and then stabilised between 35.0° and 41.1° until the test was terminated at 460cm. The rate of displacement was varied from 1.69 cm/sec to 0.093 cm/sec and back to 1.69 cm/sec, suddenly and gradually, but no effect on the measured strength was observed.

Test V.14

σ_n= 200 kPa

$\phi_{residual}$= 45.0°

$\phi_{threshold}$= 51.3°

(Figures V.14.1-2)

As it has been the case in the previous test the sample appeared to dilate upon shearing at 0.09 mm/min. After 1.9 mm of shearing, the measured strength had reached its slow peak value of 48.0° while the sample height had increased by 0.045 mm. The additional strength which was measured after the sample had reached its slow peak value should be attributed to internal ring friction due to contamination. The measured strength reached its slow residual value of 45.0° after 96.9 mm of shearing and this slow shearing part of the test was terminated after 117.8 mm.

The rate of displacement was increased suddenly to 1.69 cm/sec and the measured strength reached immediately its threshold value of 51.3°. As shearing continued at the same rate of displacement, the sample height increased by 0.075 mm and the strength of the sample decreased. After 15.8 cm of shearing at 1.69 cm/sec the measured strength had dropped to 45.0° and the sample height appeared to be decreasing. Further shearing led to an additional decrease in strength which stabilised within a range between 40.2° and 43.9° and remained at this level until the test was terminated after 435 cm. Any further change of the rate of displacement, sudden or gradual, from 0.11 cm/sec to 1.69 cm/sec had no effect on the measured strength.

Test V.15

σ_n= 250 kPa

$\phi_{residual}$= 46.3°

$\phi_{threshold}$= 50.2°

(Figures V.15.1-2)

displacement which varied from 0.14 cm/sec to 1.74 cm/sec in both directions gave no additional information on the behaviour of the sample.

Test VI.20

σ_n= 200 kPa

OCR= 4

$\phi_{residual}$= 10.2°

$\phi_{threshold}$= 23.7°

(Figures VI.20.1-2)

The same sample was consolidated at 200 kPa, and sheared at 0.09 mm/min. The measured strength reached its slow peak value of 11.8° after 0.6 mm of shearing and its slow residual value of 10.2° after 20.9 mm of shearing. The apparent increase of the measured strength towards the end of this part of the test should be attributed to internal ring friction due to contamination of the rings.

The rate of displacement was increased suddenly to 1.72 cm/sec in the positive direction, and the measured strength reached its threshold value of 23.7° at zero associated displacement. After 0.3 min of shearing at 1.72 cm/sec in the positive direction, the measured strength had decreased to 15.6° and the direction of shearing was reversed. The measured strength was not affected by this reversal of the shearing direction and fluctuated between 15.5° and 14.8° until the direction was reversed again from negative to positive after 1 min of cumulative shearing. At that point the measured strength increased instantly from 14.8° to 15.6° and then fluctuated between 13.2° and 16.4°. The direction of shearing was reversed two more times at the same rate of displacement while the measured strength was not significantly affected and varied between 16.1° and 17.2° in the negative direction and between 17.9° and 19.4° in the positive. After 4.04 min of shearing at 1.72 cm/sec in both directions, the rate of displacement was decreased gradually to 0.98 cm/sec and then to 0.094 cm/sec -in the positive direction- while the measured sample strength dropped from 17.9° to 9.07°. While the sample was being sheared at 0.094 cm/sec in the positive direction, the direction was reversed and the measured strength decreased from 9.7° to 7.9°. Continued shearing at the same rate and direction led to an increase in measured strength from 7.9° to 10.2°. After the sample had been sheared for 9.8 min, the direction of shearing was reversed from negative to positive and the measured sample strength once again decreased from 10.2° to 7.9° and stabilised at 10.2°. Before the

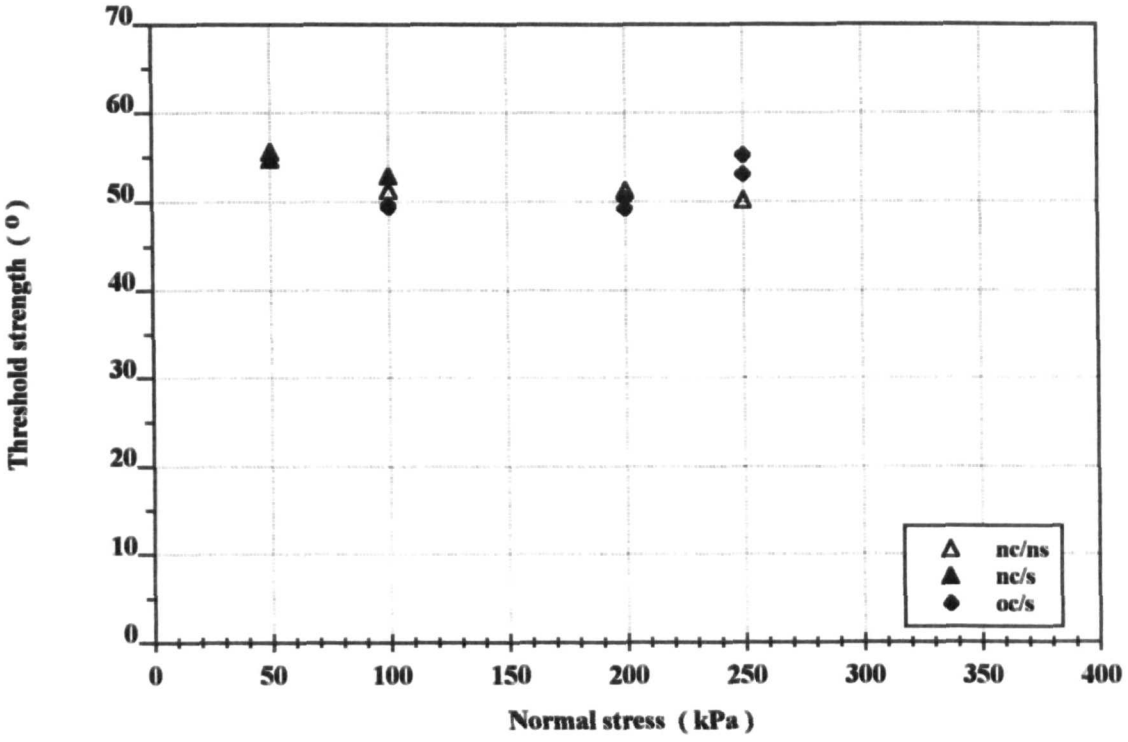


Fig. 6.4.3 Normal stress effect on threshold strength of sandy-silt samples

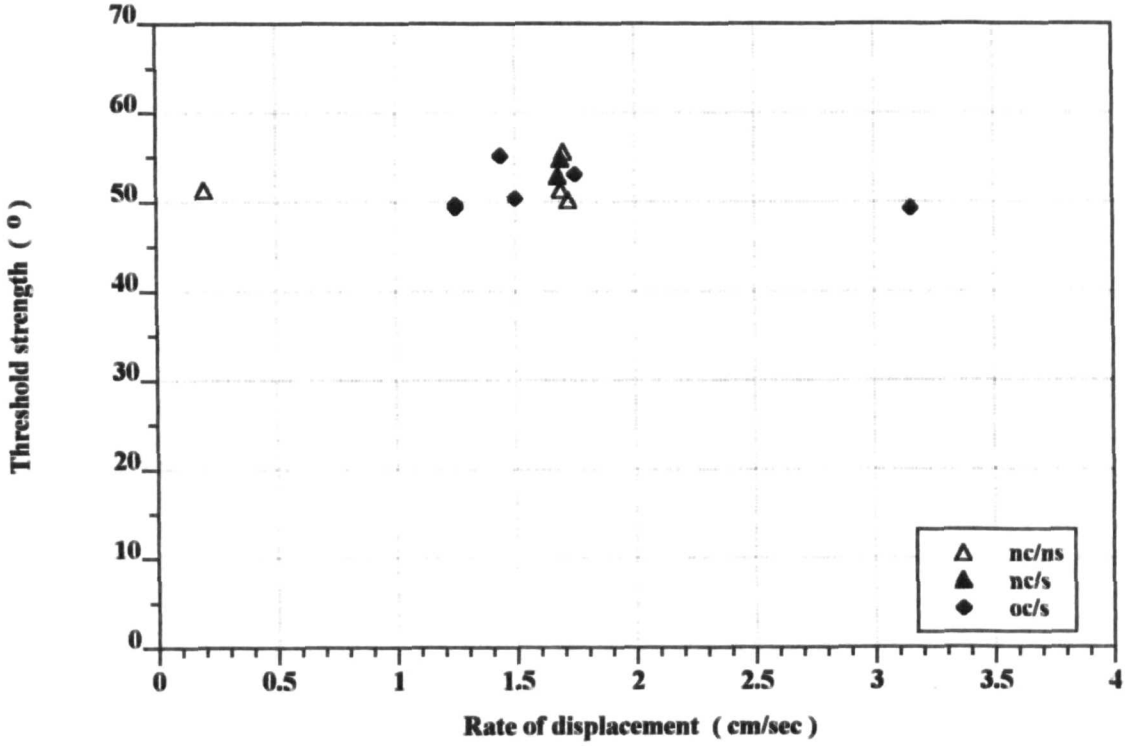


Fig. 6.4.4 Rate of displacement effect on threshold strength of sandy-silt samples

0.225 mm. After 28.6 mm of shearing at the same rate of displacement the sample reached its slow residual strength of 35.4° and its height decreased further by 0.039 mm. This part of the test was terminated after the sample had been sheared at the same rate of displacement for 135 mm.

The rate of shearing was suddenly increased to 1.70 cm/sec and the sample reached its threshold strength of 47.3° at zero associated shear displacement. As shearing progressed the sample started to dilate and its strength to decrease. After 31 cm of shearing at 1.70 cm/sec the height of the sample had increased by 0.475 mm, and its strength had decreased to 34.1°. Further shearing at the same rate of displacement led to an increase in measured strength as the sample height appeared to be decreasing. After 77.4 cm of shearing, the shear strength of the sample had stabilised around 42° while an undulating shear surface with 0.092 mm initial amplitude appeared to be forming. The rate of displacement was decreased suddenly to 0.12 cm/sec and the strength dropped to 39.9°. While the sample was being sheared at 0.12 cm/sec its strength appeared to be increasing, and, after 92.9 cm of cumulative shearing its strength had increased to 43.7°. The rate of displacement was suddenly increased once again to 1.70 cm/sec and the measured strength increased immediately to 47.3°. In the following 50.3 cm of shearing at the same rate of displacement the strength oscillated between 43.3° and 47.3° while the amplitude of the undulating shear zone increased to 0.146 mm. As it has often been the case, the wave-length of the undulating shear zone was around one sample perimeter. The rate of displacement was decreased to 0.12 cm/sec after 143.2 cm of shearing and the strength decreased to 43.1°. Subsequently, the rate of displacement remained constant for the next 16.8 cm of shearing and the measured sample strength varied around 41.2°. The third series of increasing/decreasing the rate of displacement in this particular test was aimed to observing any rate dependent phenomena of the fast residual strength of the sample. Therefore, the rate of displacement was varied gradually and increased from 0.125 cm/sec to 1.70 cm/sec within 49.7 cm of shear displacement. At 0.308 cm/sec the measured strength was 43.7°, at 0.729 cm/sec it was 44.4°, at 1.37 cm/sec it was 46.5° and at 1.70 cm/sec it was 47.8°. The rate of displacement was kept constant at that latter level for the following 98 cm of shearing in order to investigate any displacement dependency of the fast residual strength of the sample. It was concluded that the measured residual strength of the sample was not influenced by displacement. After 307.7 cm of shearing, the rate of displacement

0.105 mm, and with a decrease in measured strength which stabilised within a range between 43.9° and 47.7°. Again, the periodic internal ring friction which affected the results of the test could not be accounted for but it was concluded that the strength of the sample actually varied between 43.9° and 45.0°. After 238 cm of shearing the rate of displacement was suddenly increased to 1.75 cm/sec and the measured strength reached the value of 53.1° at zero associated displacement before it stabilised within the aforementioned range. No effect of any further change of the rate of displacement on the measured strength was observed until the test was terminated at 660 cm of shearing.

Tests On Uniform Coarse Clean Sand

General:

The samples which were investigated in this series of tests were obtained from the Soil Mechanics laboratory of Imperial College and consisted of commercially available dry, clean sand which passed the # 14 and was retained at the #10 test sieves. Two sets of tests were conducted, one on completely dry sand, non-submerged in water and one on saturated sand, submerged in water. It should be noted that only normally consolidated samples were tested in this series because excessive vibrations of the load cells were observed in an effort to complete a fast shearing test on an overconsolidated sample. Thus, in fear of damaging the equipment, tests on overconsolidated samples were abandoned. The multistage testing technique was used in this series of tests and the old rings were employed, mainly because the sand particles were large enough to ensure no sample loss but also in order to avoid any premature damage of the new rings.

Normally Consolidated, Dry

Test V.21

σ_n= 50 kPa

$\phi_{threshold}$= 40.4°

$\phi_{residual}$= 36.2°

(Figure V.21.1)

The sample was sheared at 0.079 cm/sec for the first 30.6 cm of this test and its measured strength reached a threshold value of 40.4° at zero associated

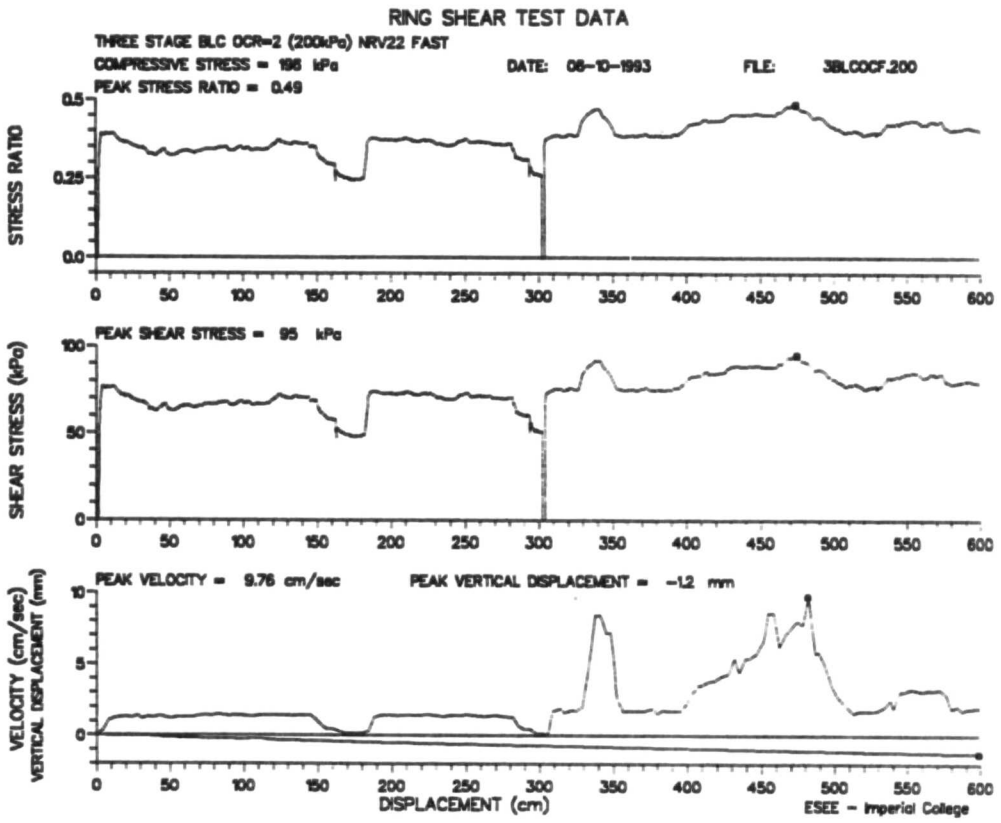


Fig. VI.2.2 Fast shearing test on over-consolidated, submerged, brown London clay (200 kPa - 3 stage test)

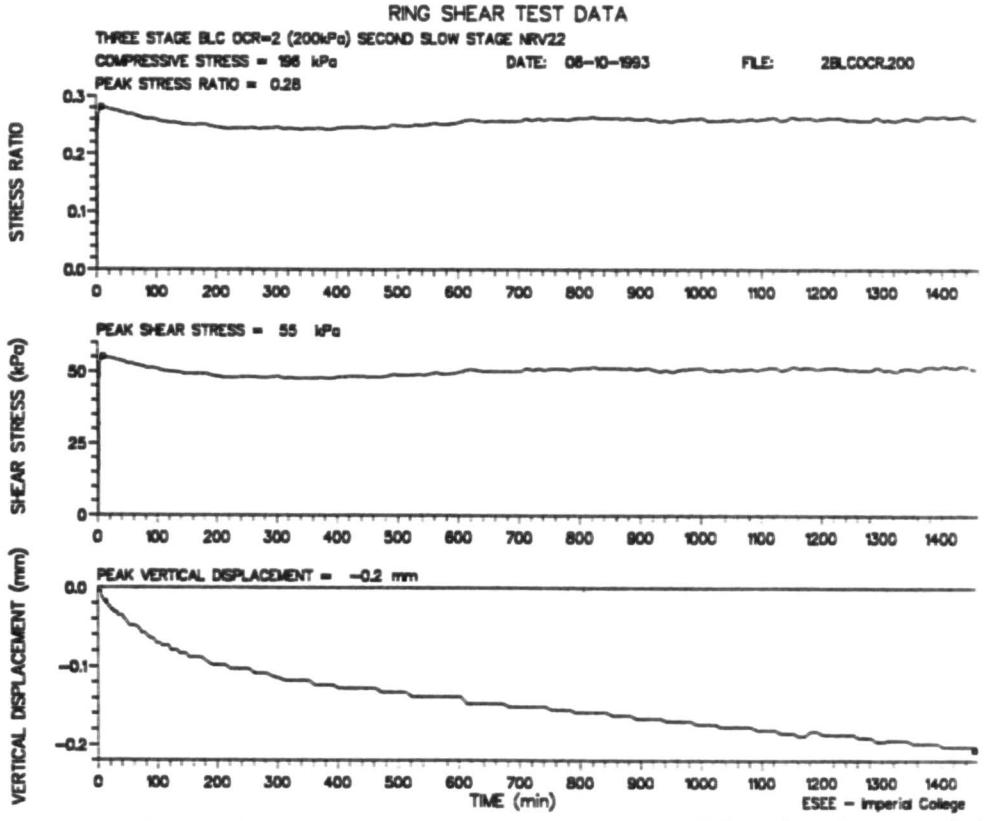


Fig. VI.2.3 Second slow shearing test on over-consolidated, submerged, brown London clay (200 kPa - 3 stage test)

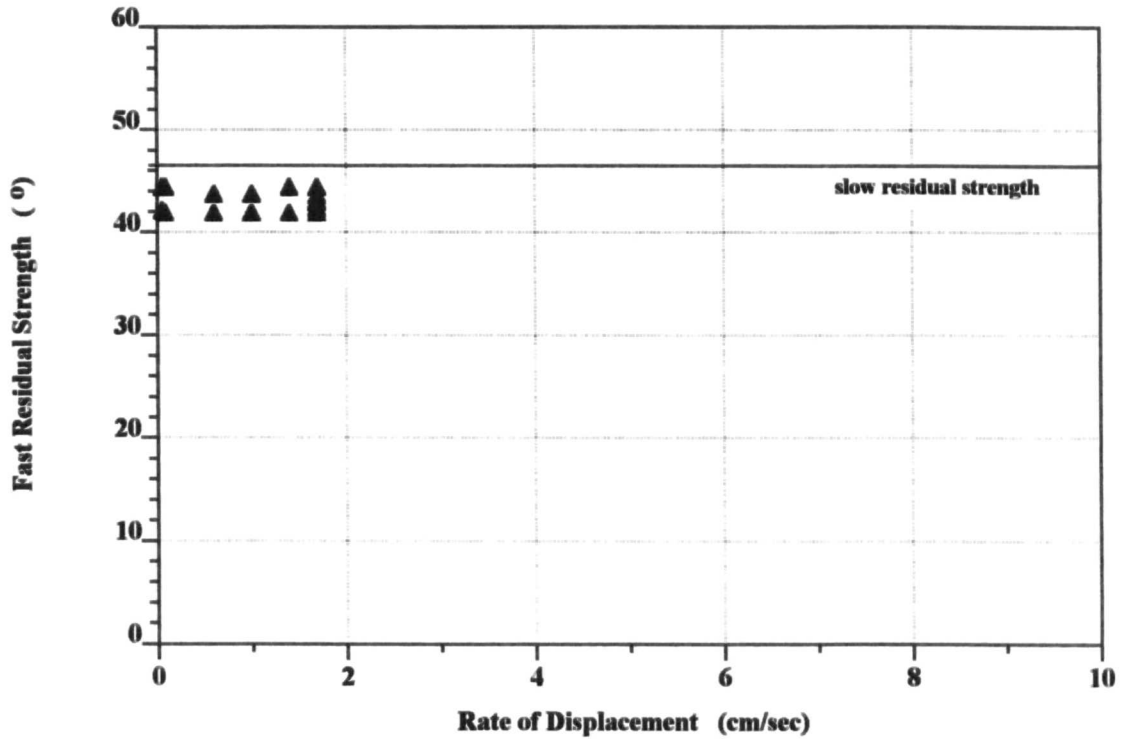


Fig. 6.4.13 Rate dependent characteristics of normally consolidated, non-submerged, pure silica sample(200 kPa)

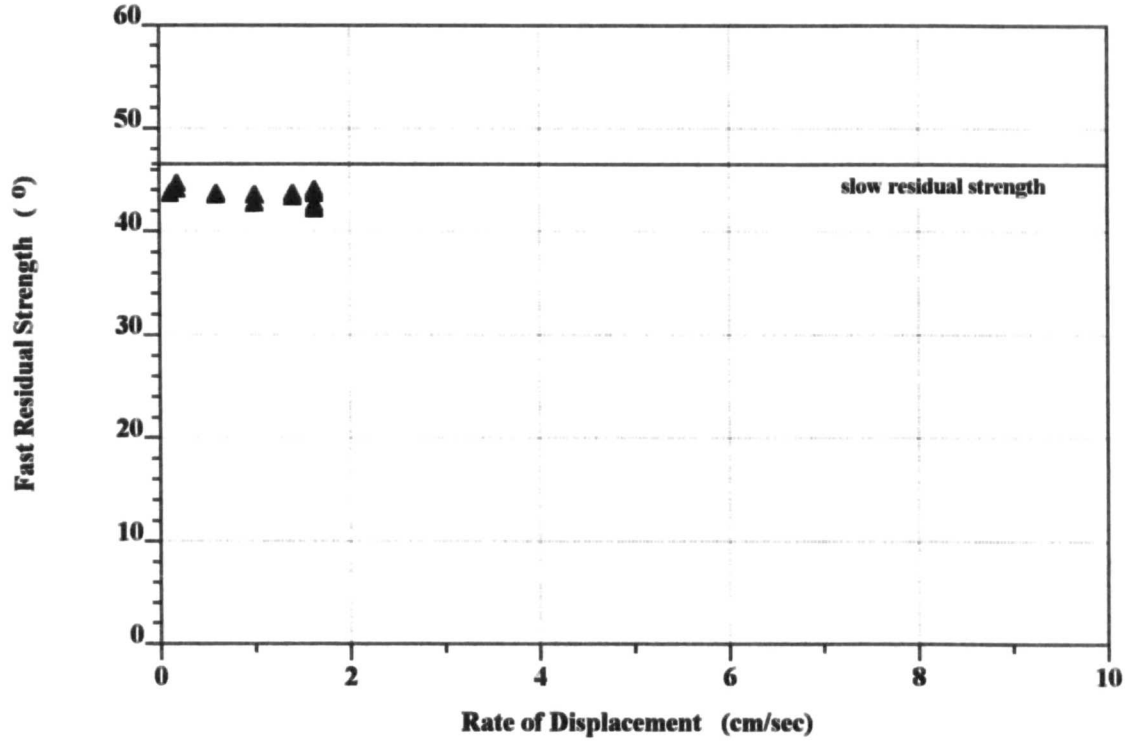


Fig. 6.4.14 Rate dependent characteristics of normally consolidated, non-submerged, pure silica sample(300 kPa)

increase in measured sample strength. After 56.3 cm of shearing at 1.29 cm/sec the sample height had dropped back to its level at the end of the slow shearing part of this test while the measured sample strength had increased to 36.2°. The rate of displacement remained at 1.29 cm/sec for a cumulative 152.8 cm of shearing and the strength of the sample fluctuated between 32.0° and 39.1° while the sample height curve gave evidence of an undulating shear surface with 0.54 mm amplitude. Subsequently, the rate of displacement was decreased to 0.45 cm/sec and to 0.27 cm/sec while the strength of the sample fluctuated between 32.0° and 25.7°. After 247 cm of shearing the rate of displacement was increased to 1.8 cm/sec and the measured strength of the sample reached immediately the value of 32.0°. In the following 200 cm of shearing, before this test was terminated, the rate of displacement was gradually increased from 1.8 cm/sec to 7.5 cm/sec, and suddenly from 1.8 cm/sec to 9.9 cm/sec. It was observed that neither of the changes of the rate of displacement influenced the measured strength of the sample which varied below the sample's slow residual value, between 16.6° and 25.7°. From the sample height curve it appeared that the shear surface had an undulating shape with 0.6 mm amplitude and 40 cm wave-length.

Test IV.18

σ_n= 100 kPa

$\phi_{residual}$= 31.7°

$\phi_{threshold}$= 39.7°

(Figures IV.18.1-2)

The sample was sheared at 0.09 mm/min and reached its residual state after 42.8 mm of shearing. Following the termination of the slow shearing part of the test after 150.6 mm of shearing, the rate of displacement was suddenly increased to 0.68 cm/sec and the strength reached its threshold value of 39.7°. Further shearing led to a drop in measured strength while the sample height increased. After 9.5 cm of shearing the strength had decreased to its slow residual value of 31.7° and continued to decrease even further. After 19.8 cm of shearing, the rate of displacement was increased to 1.3 cm/sec, the sample height had increased by 1.26 mm, and the measured strength had dropped to 24.3°. Further shearing at the same rate of displacement was associated with further decrease in measured strength to 23.5° as the sample height increased to a level of 2.09 mm above that at the end of the slow

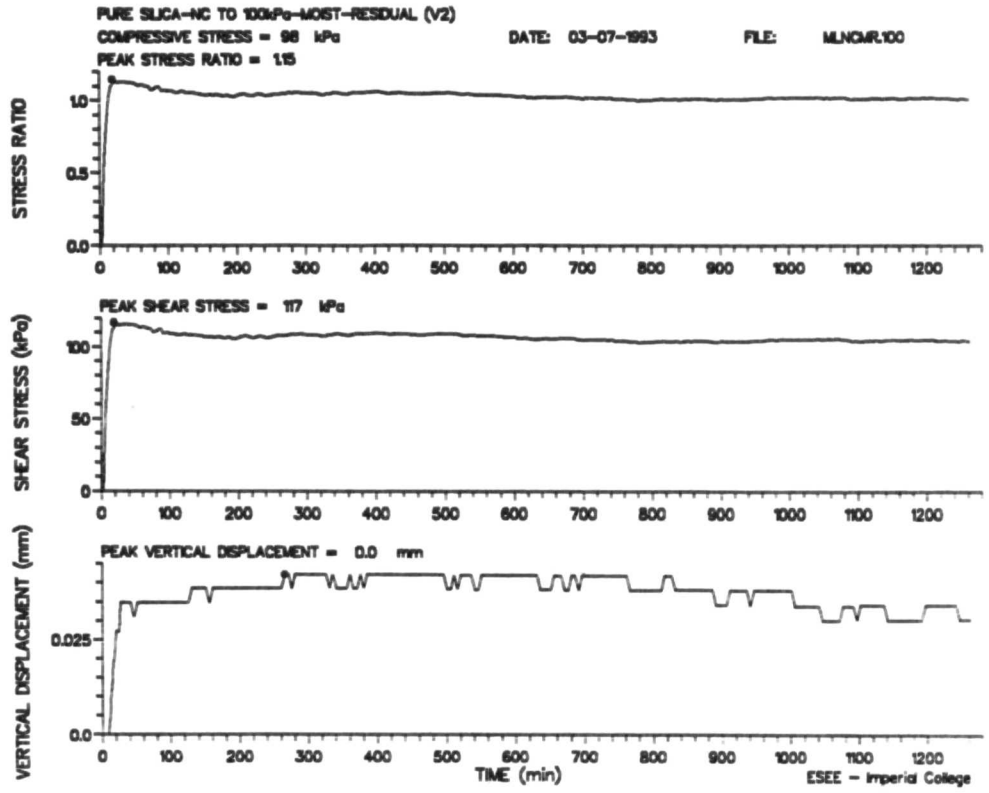


Fig. V.2.1 Slow shearing test on normally consolidated, non-submerged pure silica (100 kPa)

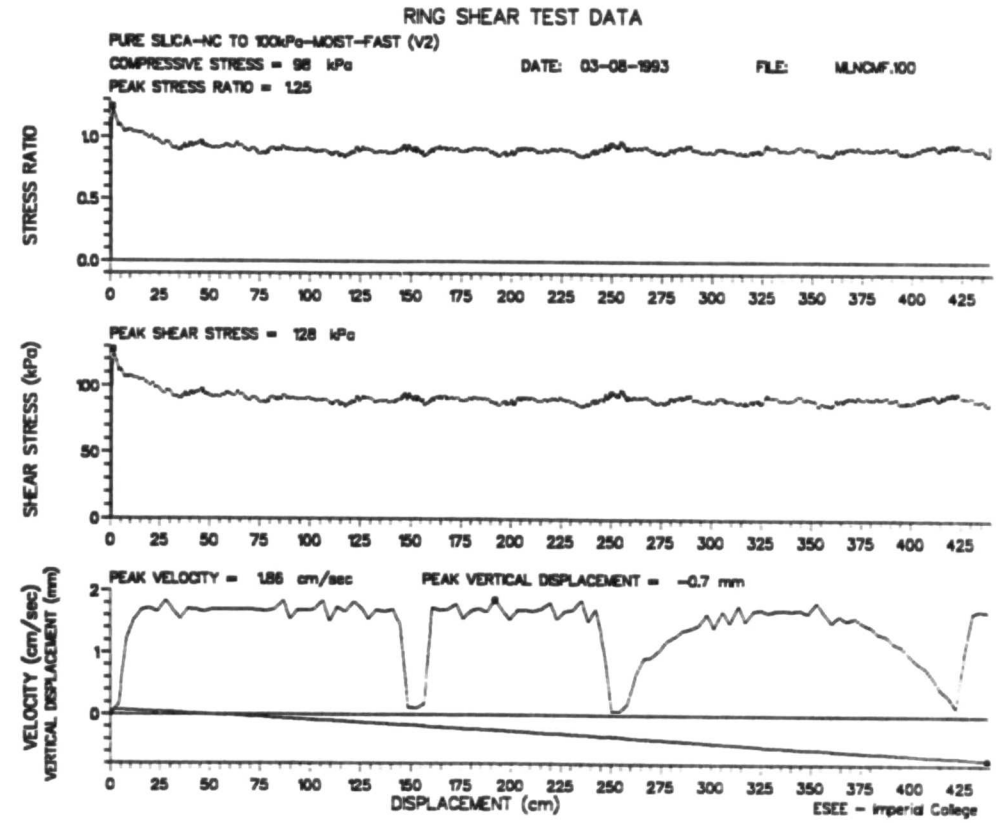


Fig. V.2.2 Fast shearing test on normally consolidated, non-submerged pure silica (100 kPa)

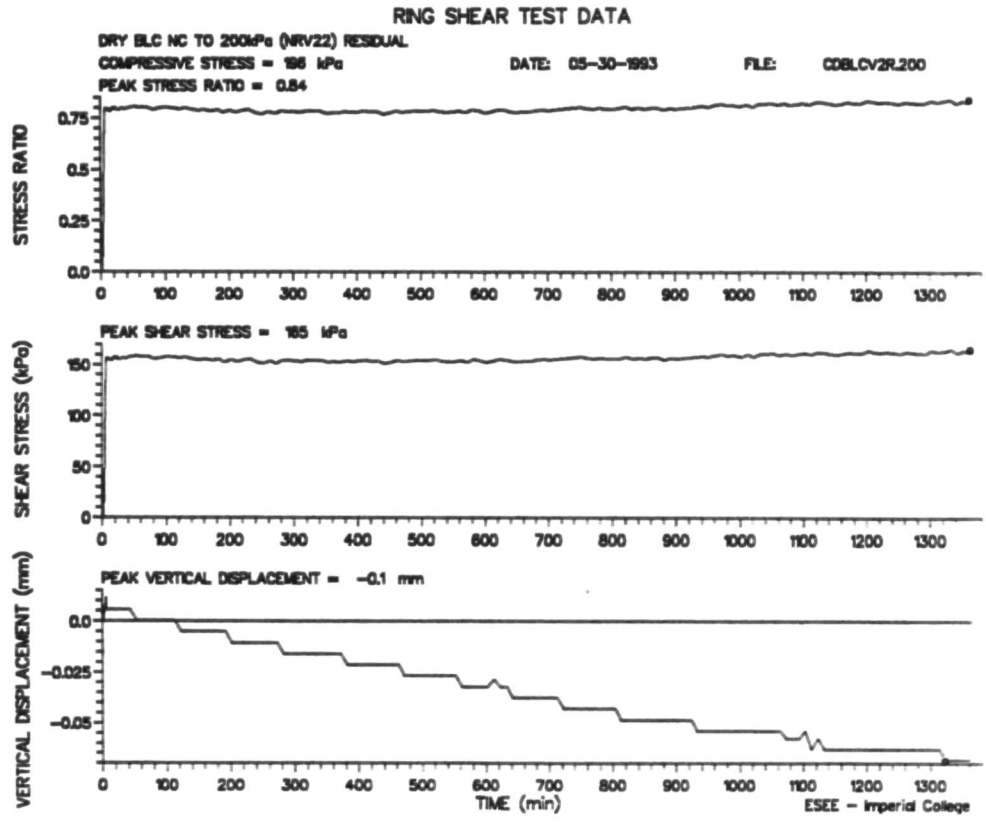


Fig. VI.6.1 Slow shearing test on normally consolidated, completely dry, brown London clay (200 kPa)

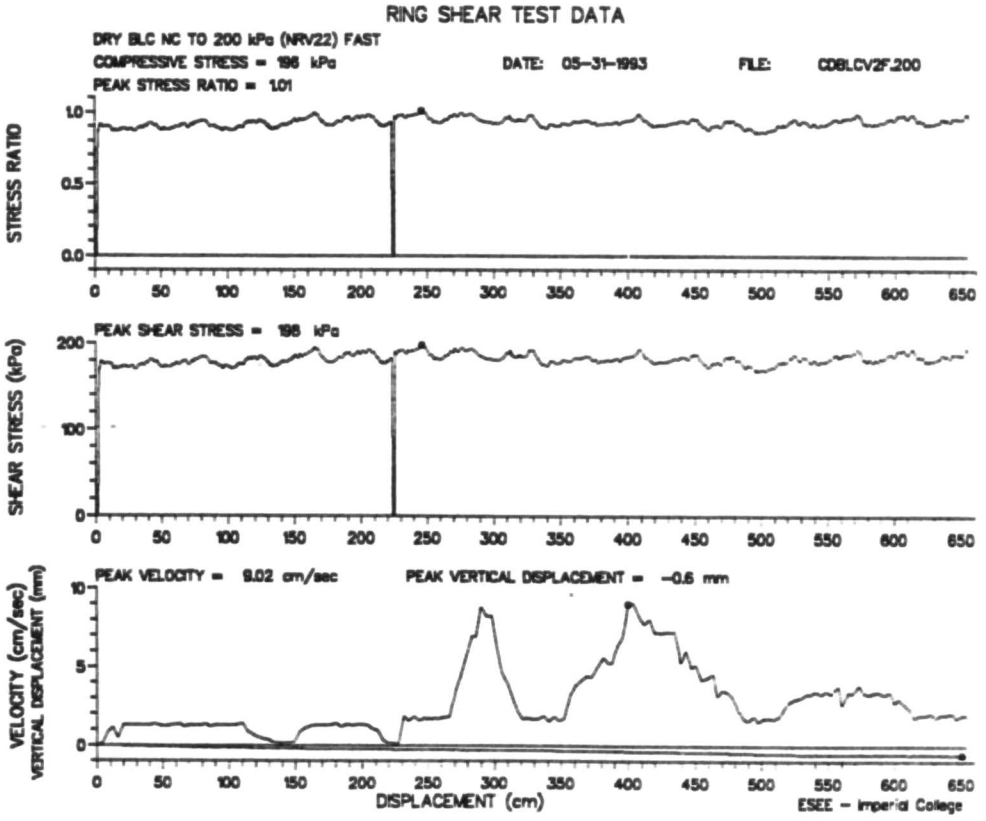


Fig. VI.6.2 Fast shearing test on normally consolidated, completely dry, brown London clay (200 kPa)

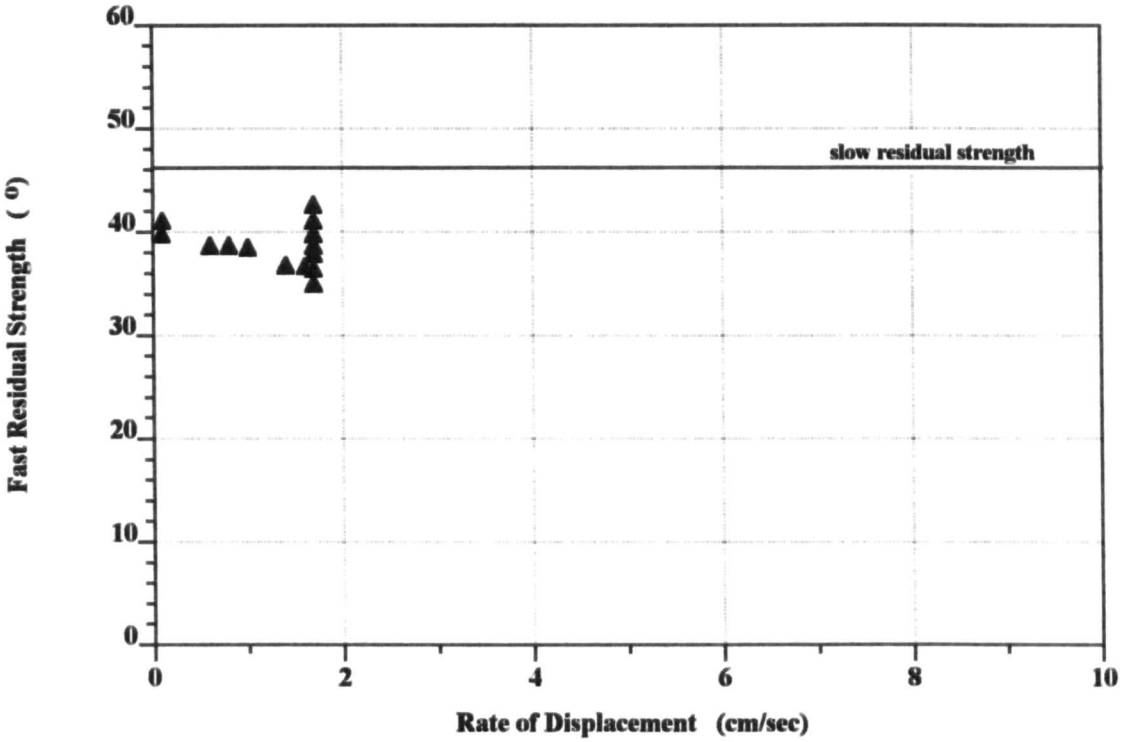


Fig. 6.4.23 Rate dependent characteristics of normally consolidated, non-submerged, sandy-silt sample (100 kPa)

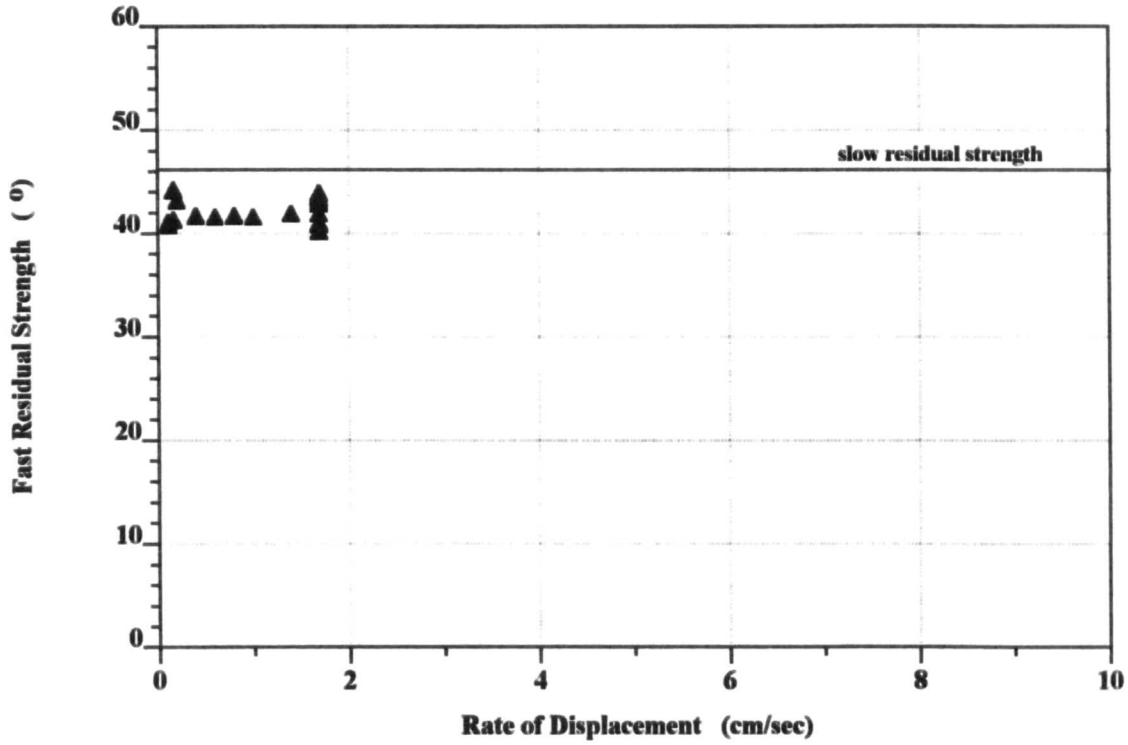


Fig. 6.4.24 Rate dependent characteristics of normally consolidated, non-submerged, sandy-silt sample (200 kPa)

0.05 mm amplitude appeared to have developed. The rate of displacement was suddenly decreased to 0.1 cm/sec and the strength decreased to 31.1°. During the second stage of this fast shearing part of the test the rate of displacement was increased gradually to 1.7 cm/sec and then decreased accordingly to 0.1 cm/sec. Positive rate characteristics were observed not only in this stage but also in those which followed until the test was terminated after 440 cm of displacement. Also, the fast residual strength of the sample was not affected by any change in the rate of displacement above 0.96 cm/sec. At 0.558 cm/sec the measured strength was 33.5°, at 0.23 cm/sec it was 32.2° and it remained constant at 35.1° for any rate of displacement above 0.96 cm/sec. The undulating shape of the sample height curve with an amplitude of 0.025 mm should be attributed mainly to the apparatus and not to the existence of an undulating shear surface, since, if the latter was the case, it would have been reflected on the stress ratio/displacement curve.

Test IV.23

σ_n= 300 kPa

OCR= 1.5

$\phi_{residual}$= 31.2°

$\phi_{threshold}$= 33.8°

(Figures IV.23.1-2)

The sample reached its slow residual strength of 31.2° after 47.5 mm of shearing, and this slow shearing part of the test was terminated after 129.2 mm. It was expected that, at this point of this multistage testing procedure, there would have been a substantial amount of soil contaminating the rings but from the shape of the stress-strain curve it may be concluded that this does not affect significantly the results of the test.

The rate of displacement was increased suddenly to 1.71 cm/sec and the measured sample strength reached its threshold value of 33.8° at zero associated displacement. Further shearing at the same rate of displacement led to an increase in sample height and a decrease in measured strength. After 2.75 cm of shearing the measured strength was 31.2° and after 18.3 cm it had decreased to 19.8° while the sample height had increased by 1.09 mm. Continued shearing at the same rate of displacement was associated with the development of an undulating shear surface with an amplitude of 0.54 mm and a wave-length of 40 cm. The fast residual strength

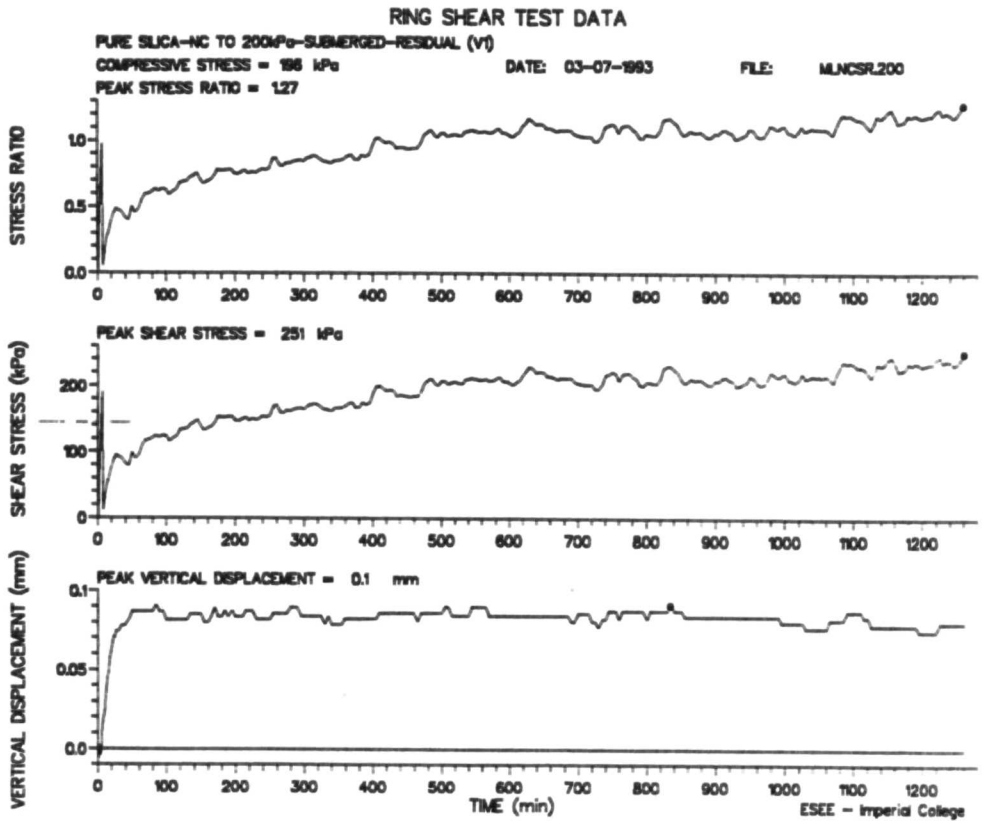


Fig. V.7.1 Slow shearing test on normally consolidated, submerged pure silica (200 kPa)

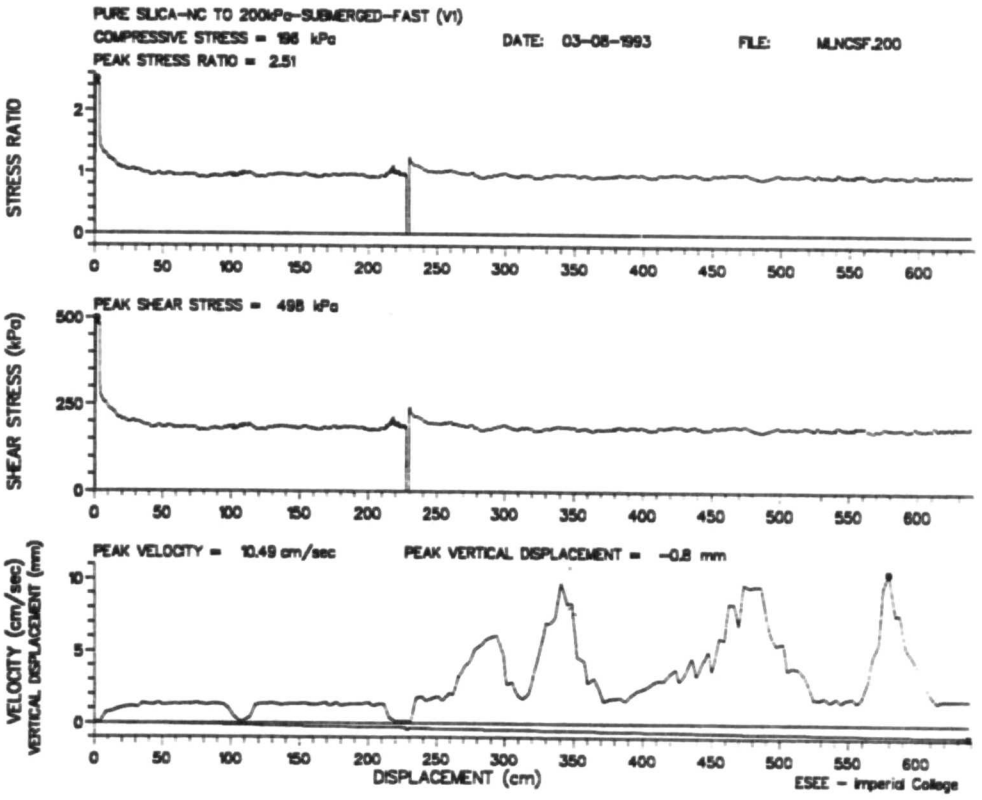


Fig. V.7.2 Fast shearing test on normally consolidated, submerged pure silica (200 kPa)

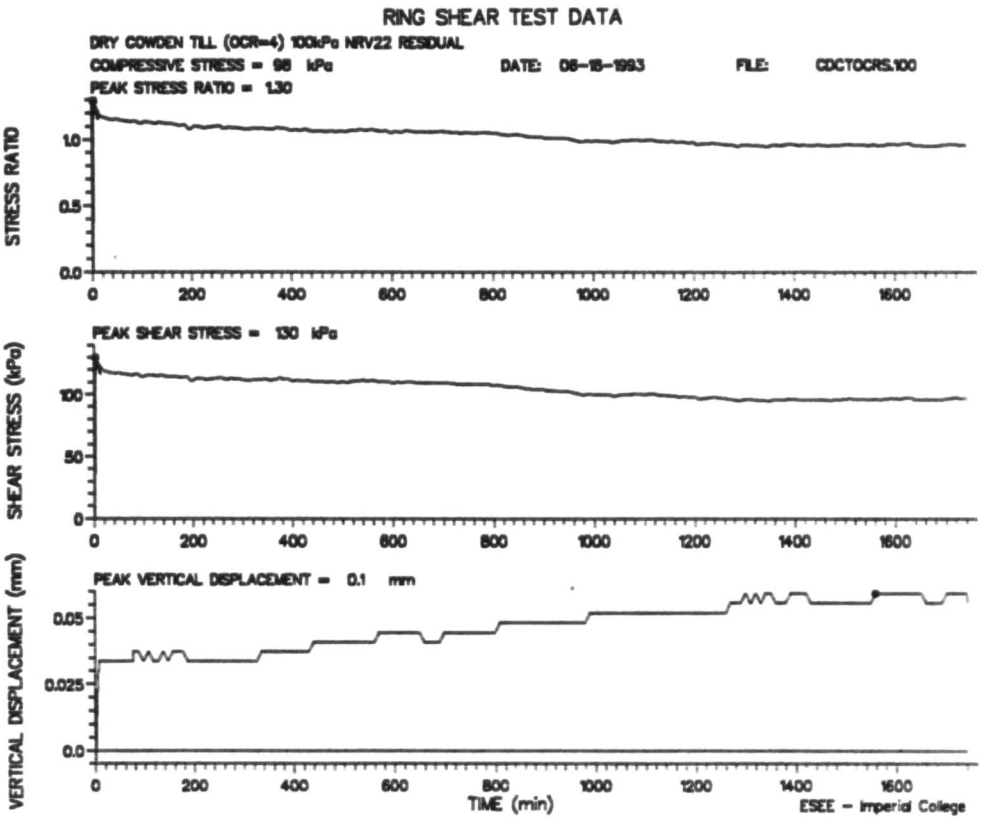


Fig. VI.11.1 Slow shearing test on over-consolidated, completely dry, Cowden till (100 kPa - OCR= 4)

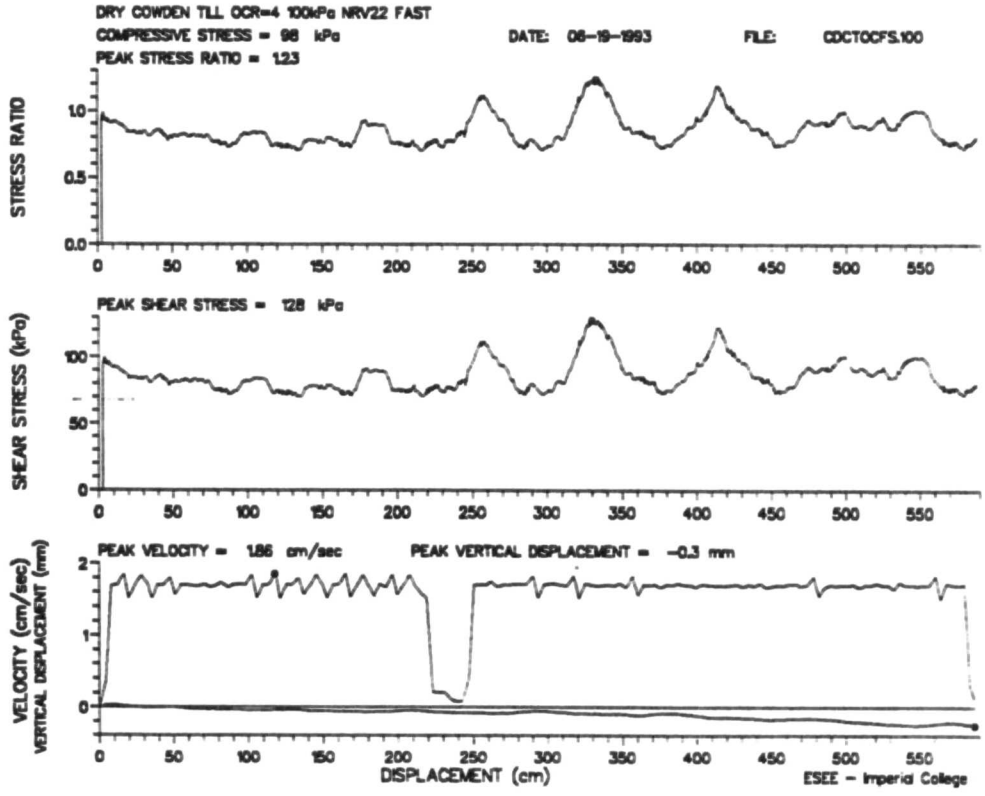


Fig. VI.11.2 Fast shearing test on over-consolidated, completely dry, Cowden till (100 kPa - OCR= 4)

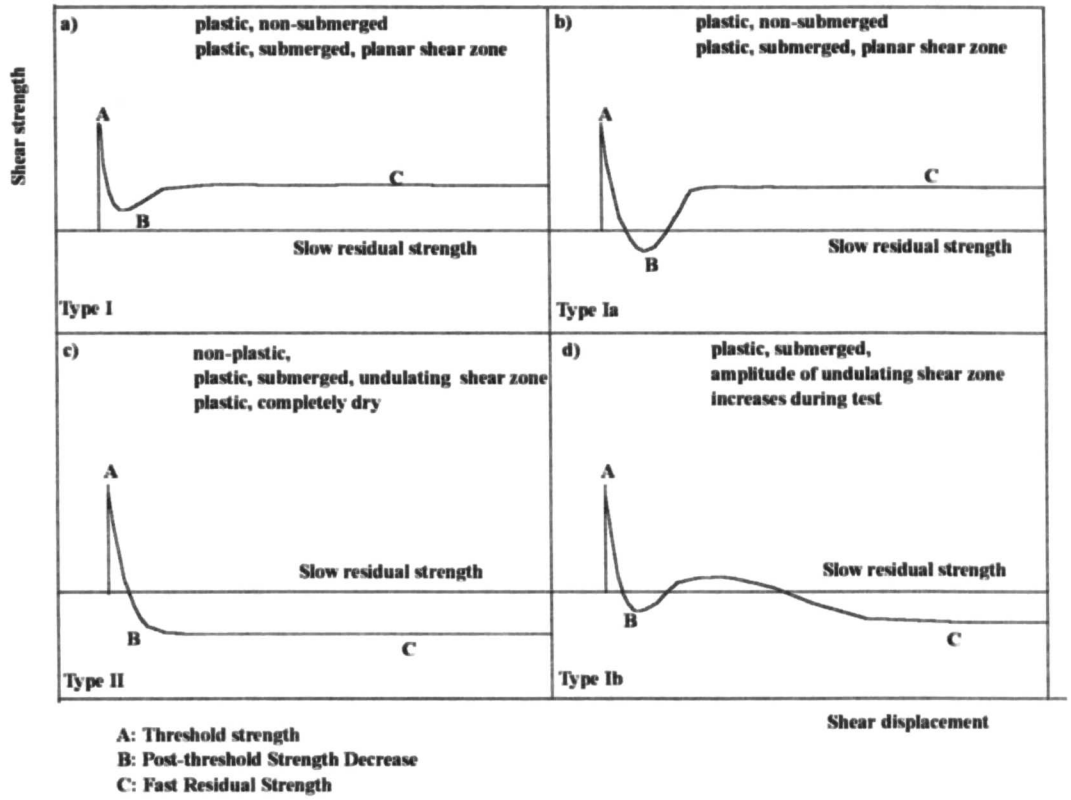


Fig. 6.5.1 Types of soil behaviour observed during fast shearing

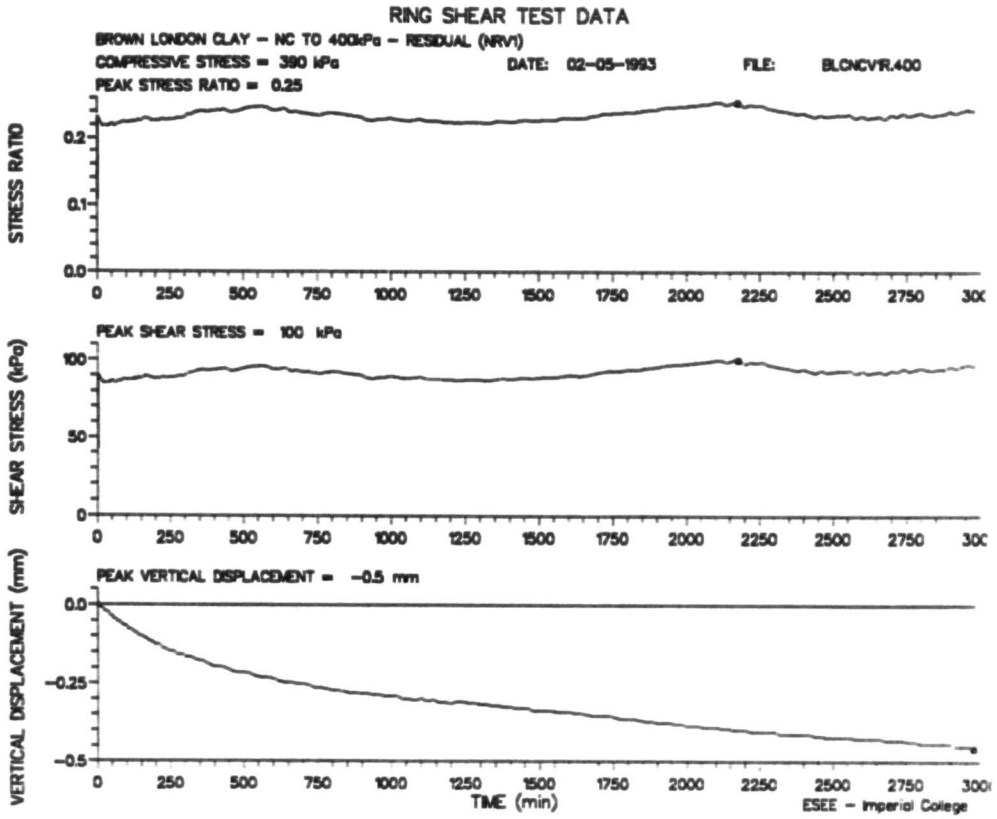


Fig. IV.4.1 Slow shearing test on normally consolidated, non-submerged brown London clay (400 kPa)

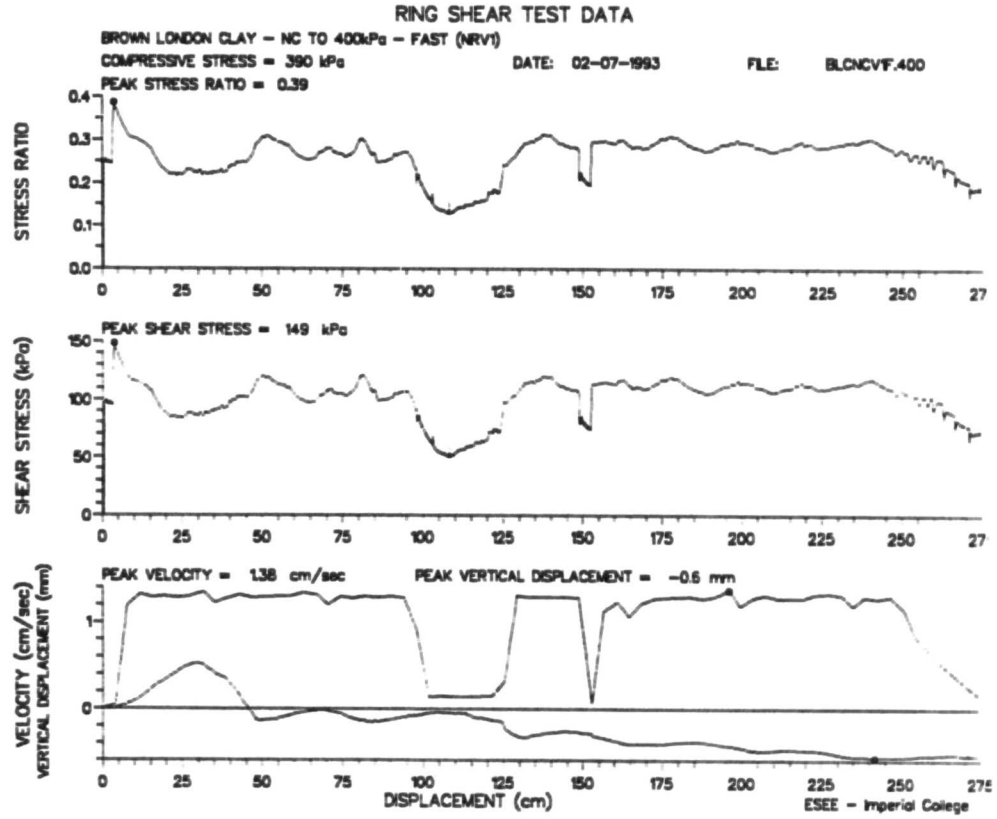


Fig. IV.4.2 Fast shearing test on normally consolidated, non-submerged brown London clay (200 kPa)

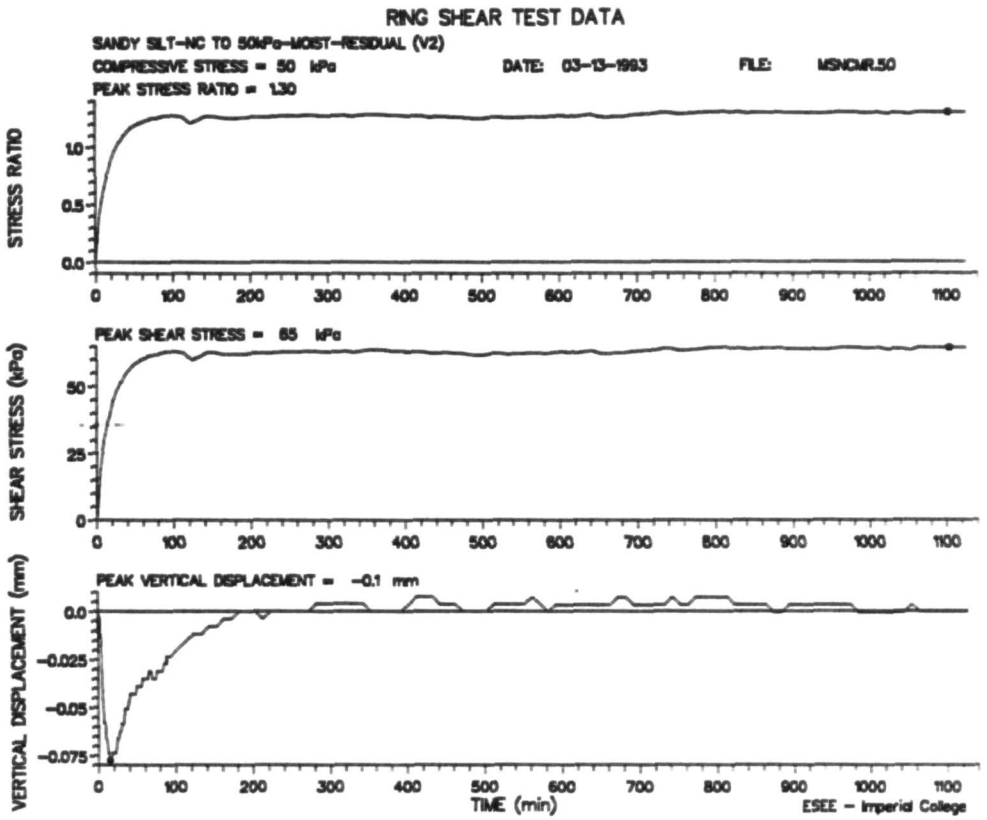


Fig. V.12.1 Slow shearing test on normally consolidated, non-submerged artificial sandy-silt (50 kPa)

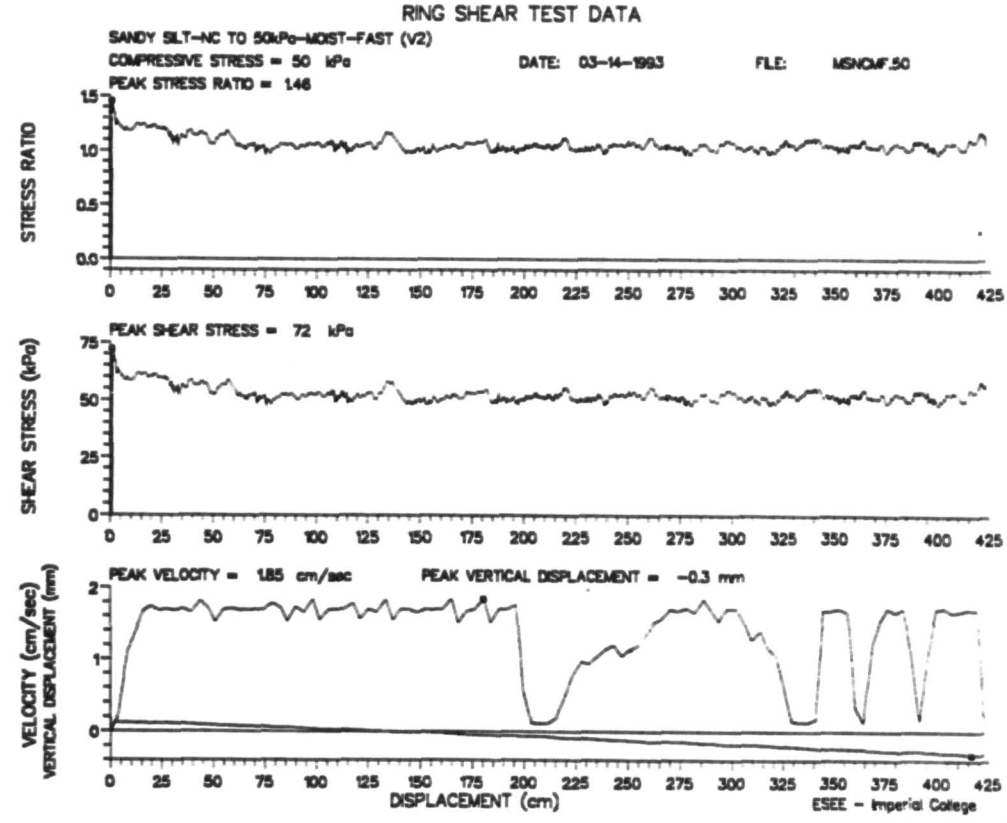


Fig. V.12.2 Fast shearing test on normally consolidated, non-submerged artificial sandy-silt (50 kPa)

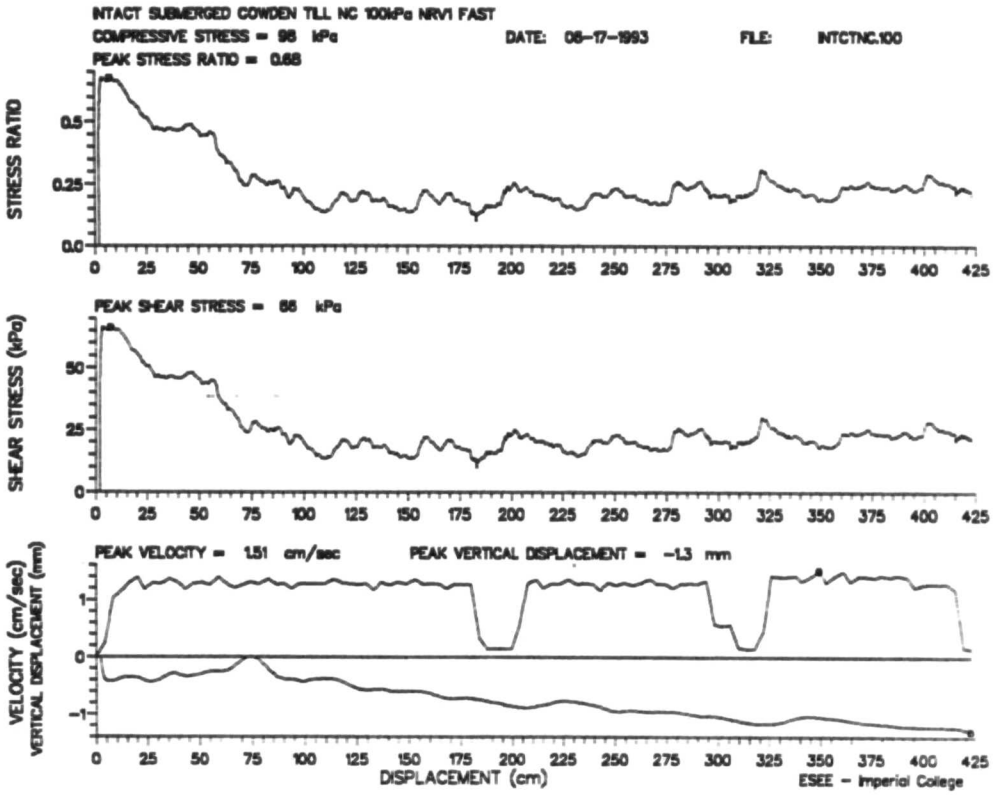


Fig. VI.17.1 Fast shearing test on "intact", normally consolidated, submerged, Cowden till (100 kPa)

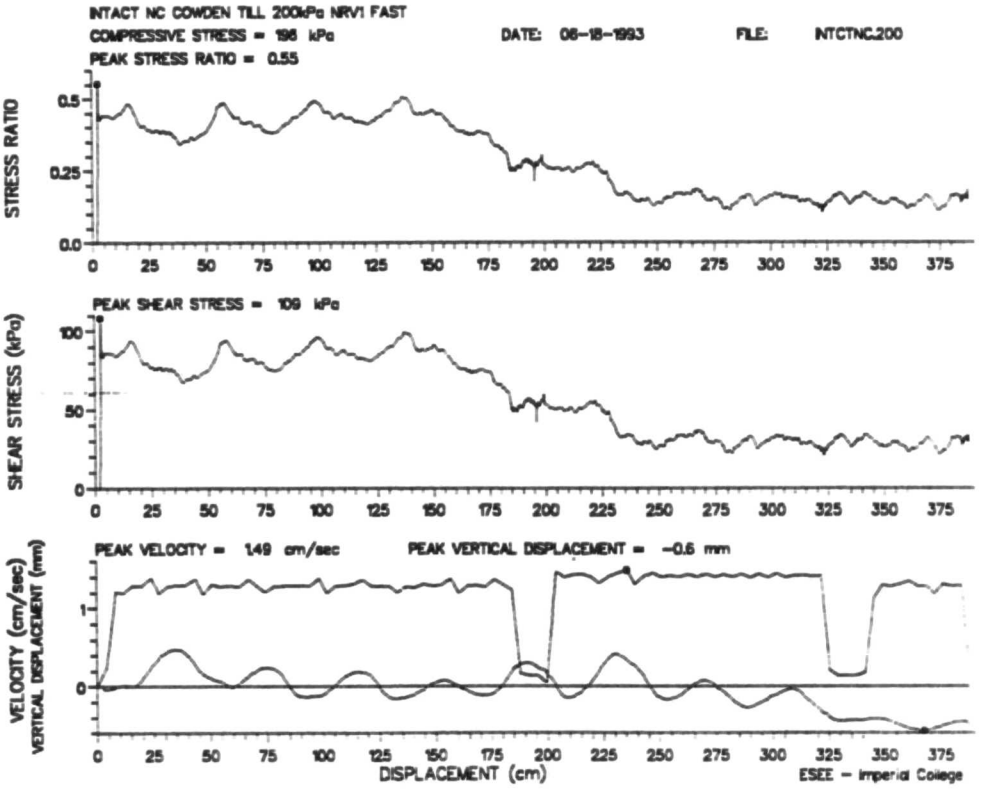


Fig. VI.18.1 Fast shearing test on "intact", normally consolidated, submerged, Cowden till (200 kPa)

1.41 cm/sec for the next 80 cm of shearing while the strength of the sample fluctuated between 31.1° and 33.1°. The test was terminated after the sample had been sheared for 340 cm.

Tests on Intact Samples:

Brown London clay

Overconsolidated, Submerged

Test VI.15

σ_n= 100 kPa

OCR= 4

$\phi_{threshold}$= 49.5°

(Figure VI.15.1)

The sample was sheared at 1.29 cm/sec, and the measured strength reached its threshold value of 49.5° but it immediately appeared to be decreasing while the sample height increased. After 16.9 cm of shearing at the same rate of displacement the measured strength had decreased to 26.0° and the sample height had increased by 0.52 mm. Further shearing at 1.29 cm/sec led to an increase in measured strength which fluctuated between 27.8° and 33.1° while the shape of the sample height curve revealed the development of an irregular shear surface. This shear surface appeared to be undulating at the beginning of the test with an amplitude which varied from 0.52 mm, to 0.25 mm, to 0.4 mm and a wave-length around 50 cm. However, after the sample had been sheared for 143.3 cm at 1.29 cm/sec the undulating shape of the shear surface had ceased to exist. After the sample had been sheared for 222.9 cm the rate of displacement was decreased suddenly to 0.15 cm/sec and the measured strength decreased and stabilised at 17.0°. After 246.1 cm of cumulative shearing the rate of displacement was increased suddenly to 1.40 cm/sec were it remained for another 108.1 cm while the measured strength fluctuated between 28.3° and 31.2°. The rate of displacement was decreased to 0.15 cm/sec just before the test was terminated -after 440 cm of shearing- while the measured strength also decreased.

Test VI.16

σ_n= 200 kPa

test was terminated, the rate of displacement was varied from 0.08 cm/sec to 1.66 cm/sec in both directions but no further evidence on the behaviour of the sample were observed.

Cowden till

Normally Consolidated, Submerged

Test VI.21

σ_n= 100 kPa

$\phi_{residual}$= 38.6°

$\phi_{threshold}$= 45.3°

(Figures VI.21.1-2)

The sample was sheared at 0.09 mm/min, and the measured strength reached its slow residual value of 38.6° after 131.1 mm of shearing while its height had decreased by 0.45 mm. This part of the test was terminated 138.7 mm of shearing.

The rate of displacement was increased suddenly to 1.38 cm/sec and the measured strength immediately reached its threshold value of 45.3°. While shearing at 1.38 cm/sec continued in the positive direction for the first 0.4 min of this test, the measured strength decreased to 43.2° and when the direction of shearing was reversed the strength slightly increased to 43.9°. During the first 4.08 min of this test the sample had been sheared at rates of displacement which varied between 1.38 cm/sec and 1.64 cm/sec and the direction of shearing had been reversed six times. However, the measured strength had stabilised within a range between 40.2° and 40.9°. Therefore, no significant effect of the reversal of shearing direction on the measured strength was observed. After the sample had been sheared for 4.08 min the rate of displacement was gradually decreased from 1.64 cm/sec, to 0.53 cm/sec, and to 0,13 cm/sec, in the positive direction, but the measured strength remained at 40.2°. The direction of shearing was reversed from positive to negative at 0.13 cm/sec and the strength increased gradually from 39.2° to 40.2°. Before the test was terminated, the rate of displacement was varied from 0.078 cm/sec to 1.41 cm/sec and the direction of shearing was reversed four more times. However, no additional information on the behaviour of the sample was accumulated.

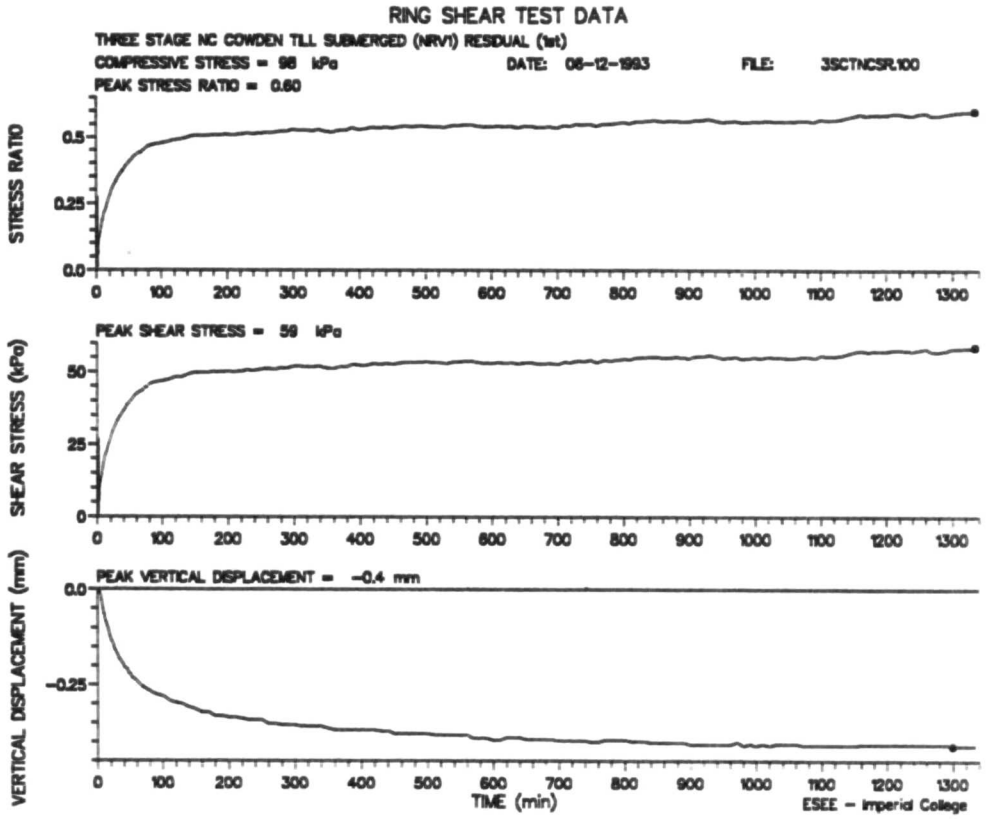


Fig. VI.3.1 First slow shearing test on normally consolidated, submerged, Cowden till (100 kPa - 3 stage test)

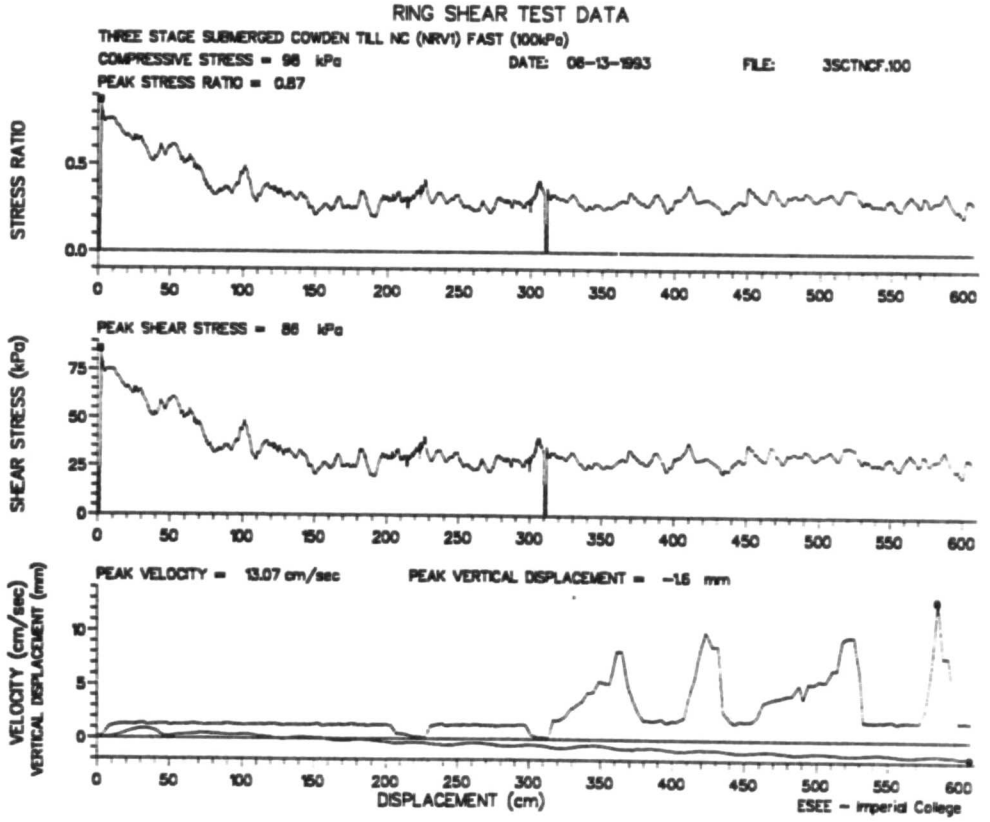


Fig. VI.3.2 Fast shearing test on normally consolidated, submerged, Cowden till (100 kPa - 3 stage test)

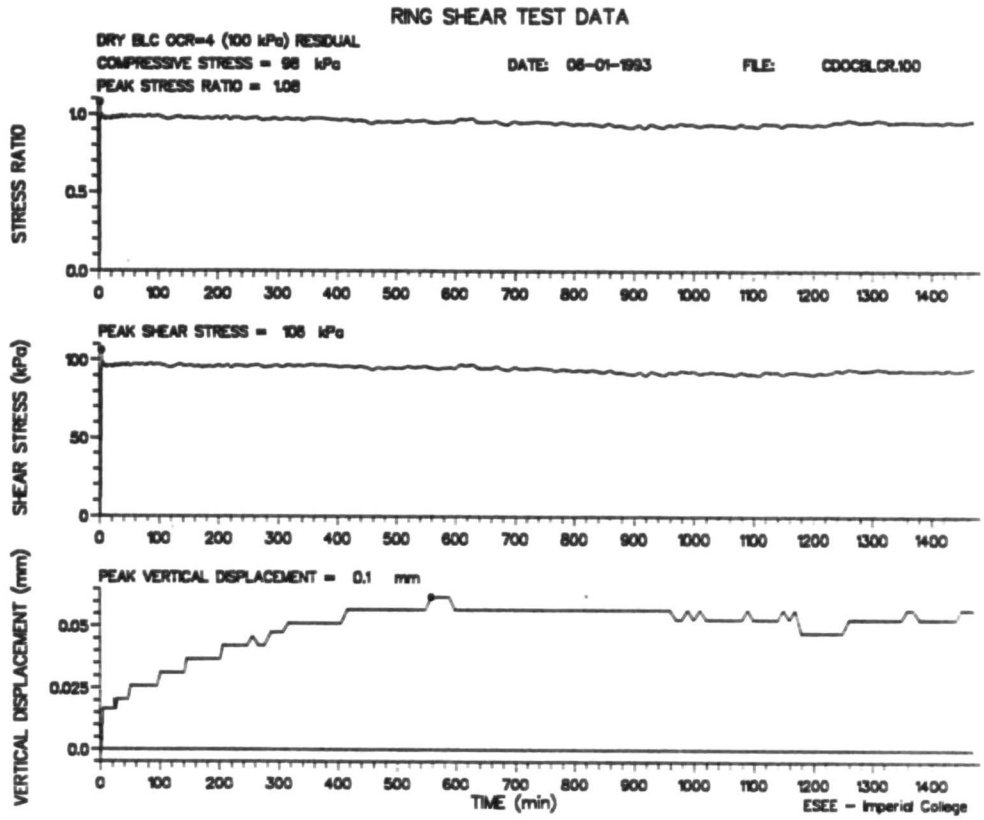


Fig. VI.7.1 Slow shearing test on over-consolidated, completely dry, brown London clay (100 kPa - OCR= 4)

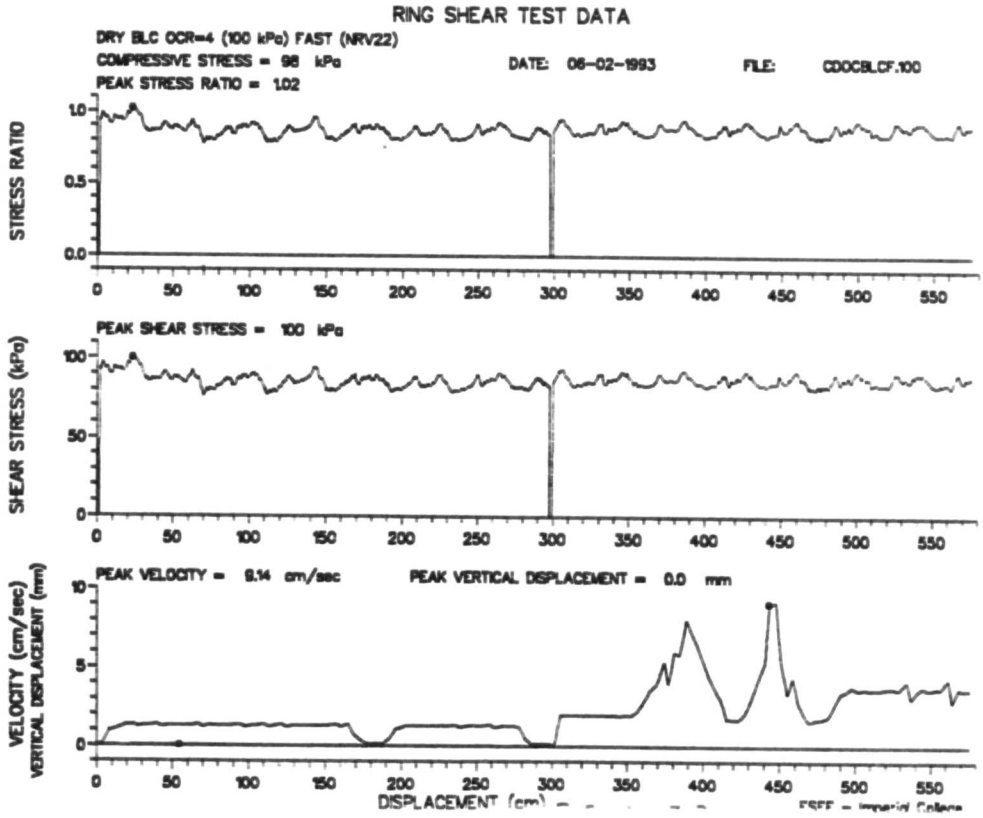


Fig. VI.7.2 Fast shearing test on over-consolidated, completely dry, brown London clay (100 kPa - OCR= 4)

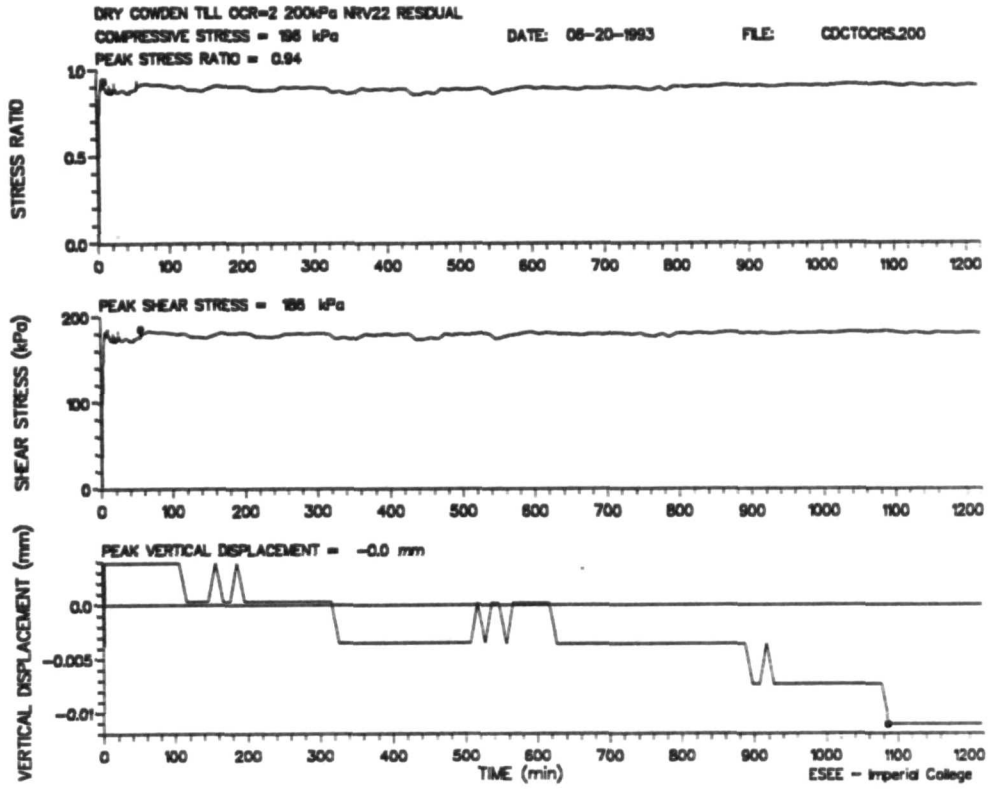


Fig. VI.12.1 Slow shearing test on over-consolidated, completely dry, Cowden till (200 kPa - OCR= 2)

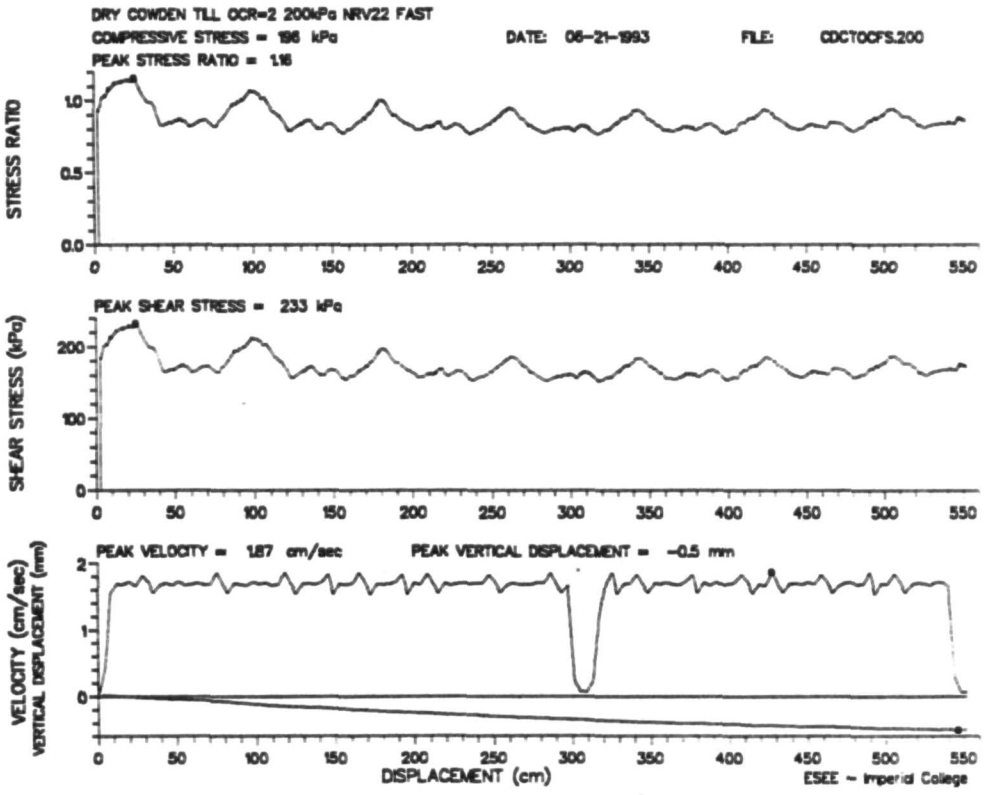


Fig. VI.12.2 Fast shearing test on over-consolidated, completely dry, Cowden till (200 kPa - OCR= 2)

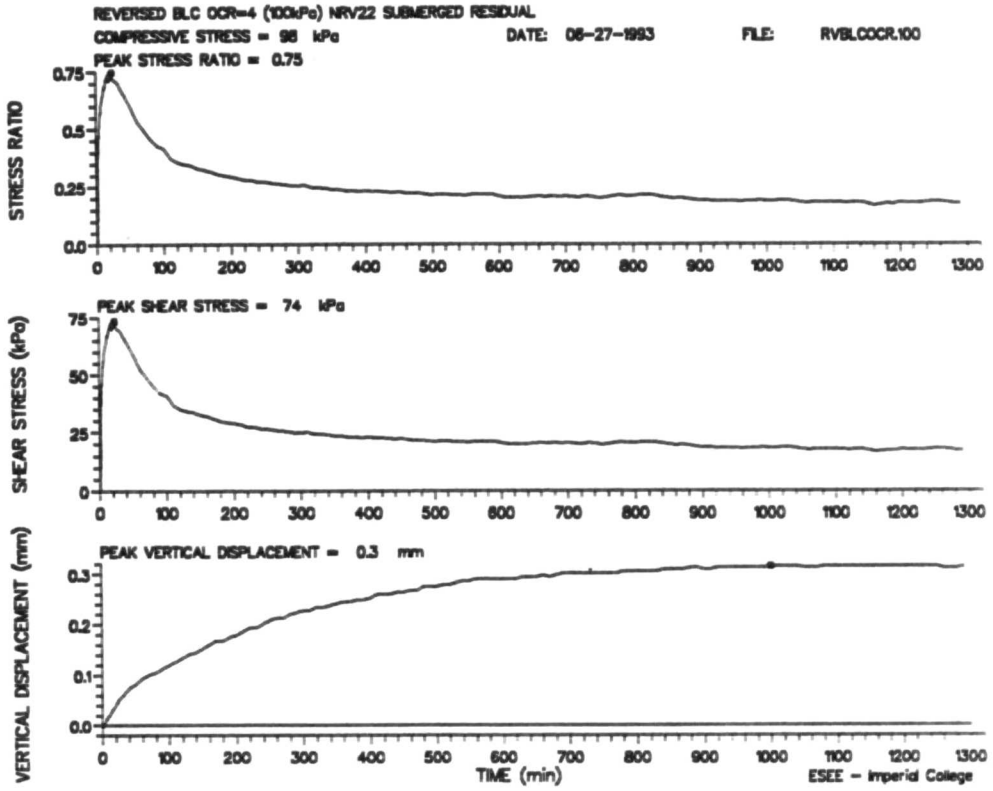


Fig. VI.19.1 Slow shearing test on over-consolidated, submerged, brown London clay (100 kPa - reversed direction)

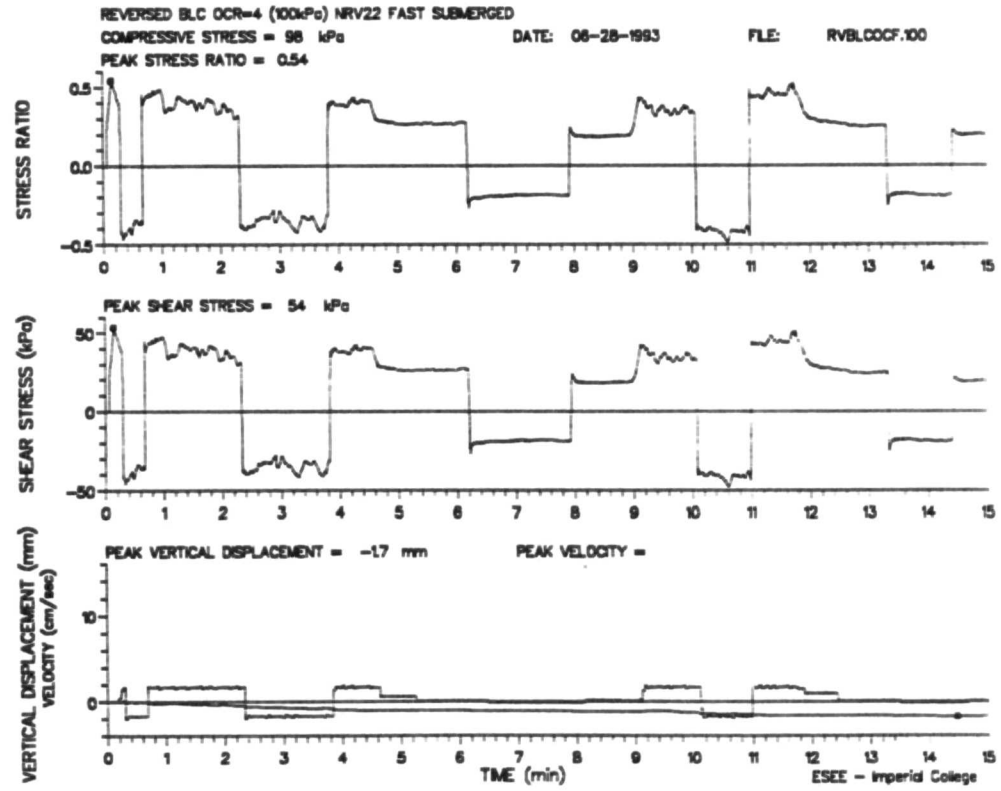


Fig. VI.19.2 Fast shearing test on over-consolidated, submerged, brown London clay (100 kPa - reversed direction)

OCR= 2

$\phi_{threshold}$= 20.3°

(Figure VI.16.1)

The sample was sheared at 1.40 cm/sec, and the measured strength reached immediately its threshold value of 20.3°. Shearing at the same rate of displacement continued for the first 173.3 cm of this test. The measured strength fluctuated between 18.9° and 21.1° and the sample height curve indicated the existence of an irregularly shaped shear surface which was fluctuating, at times, within a range of 0.25 mm. After the sample had been sheared for 173.3 cm, the rate of displacement was decreased gradually from 1.40 cm/sec, to 0.86 cm/sec, to 0.16 cm/sec and to 0.046 cm/sec while the measured strength decreased from 18.9°, to 14.0°, to 13.5°, and to 9.06°, respectively. Further shearing was associated with a sudden increase in the rate of displacement to 1.40 cm/sec, and an increase in measured strength which fluctuated between 16.6° and 19.6°. During the last part of this test, the rate was increased to 1.40 cm/sec and the measured sample strength stabilised between 18.07° and 19.6°. The test was terminated after 420 cm of shearing.

Cowden till

Normally Consolidated, Submerged

Test VI.17

σ_n= 100 kPa

$\phi_{threshold}$= 34.2°

(Figures VI.17.1)

The rate of displacement was increased gradually from 0.158 cm/sec, to 1.0 cm/sec, and to 1.26 cm/sec were it remained for the first 178.9 cm of this test. The measured strength reached the value of 25.2° but, while the rate of displacement continued to be increased, the former also continued to increase until it reached the value of 34.2°. Meanwhile, the sample height had decreased by 0.42 mm. As shearing continued at 1.26 cm/sec the sample appeared to dilate and its measured strength to decrease. After 73.0 cm of cumulative shearing, the measured strength had decreased to 14.0° and the sample height had increased back to its initial level. Further shearing at the same rate of displacement was associated with further decrease of the measured strength which stabilised within a range between 8.35° and 14.0°. The rate

Test VI.22

σ_n= 200 kPa

$\phi_{residual}$= 38.0°

$\phi_{threshold}$= 43.5°

(Figures VI.22.1-2)

The sample which was sheared at 0.09 mm/min, and the measured strength reached its slow residual value of 38.0° after 55.1 mm of shearing. This part of the test was terminated after 131.1 mm of shearing.

The rate of displacement was increased suddenly to 1.7 cm/sec and the measured strength immediately reached its threshold value of 43.5°. Further shearing at the same rate of displacement in the positive direction led to a decrease in the measured strength to 35.3°. The direction of shearing was reversed after 0.23 min of testing, and the strength regained its previous value of 35.3° and fluctuated between that and 38.0°. During the following 4.1 min of this test the direction of shearing was reversed three times while the measured strength fluctuated between 39.8° and 41.1°. The reversal of the shearing direction appeared again to have no significant effect on the measured sample strength. The rate of displacement was decreased gradually from 1.7 cm/sec, to 0.15 cm/sec, and to 0.1 cm/sec after the sample had been sheared for 3 min but this also seemed to have no effect on the measured strength. While the sample was being sheared at 0.1 cm/sec, the shearing direction was reversed once more from positive to negative and the measured strength gradually returned to its previous level. Before the test was terminated, the rate of displacement was changed from 0.1 cm/sec to 1.7 cm/sec and the shearing direction was reversed three more times. However, no additional evidence about the behaviour of the sample were observed.

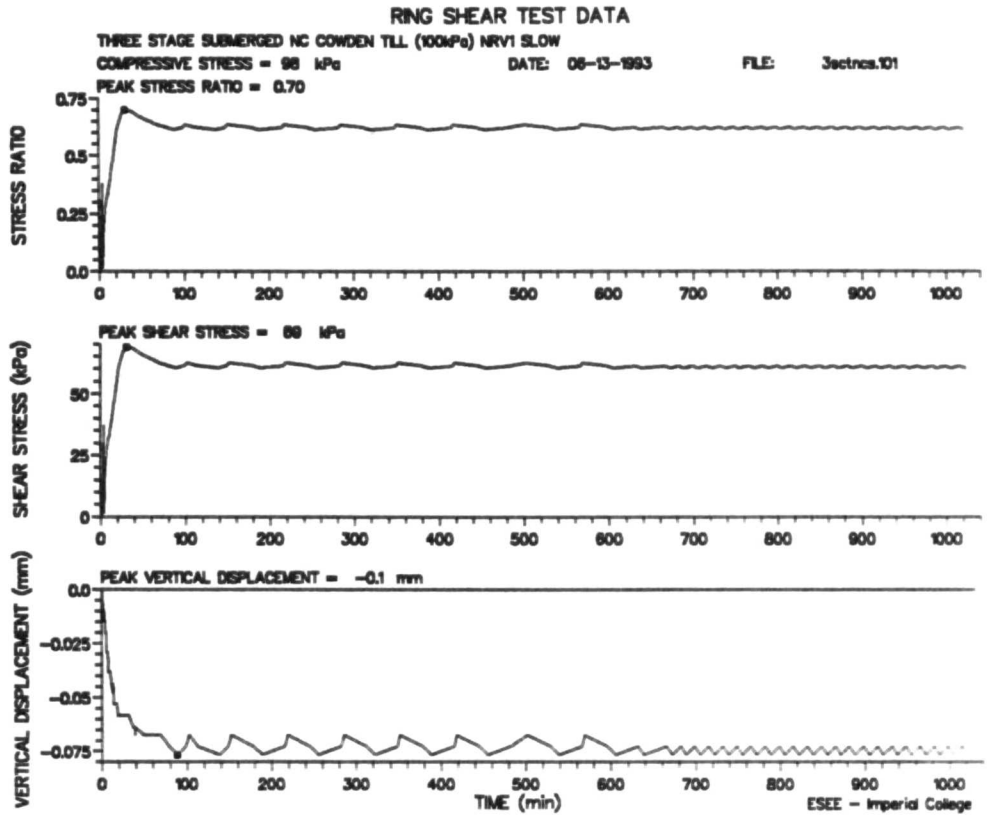


Fig. VI.3.3 Second slow shearing test on normally consolidated, submerged, Cowden till (100 kPa - 3 stage test)

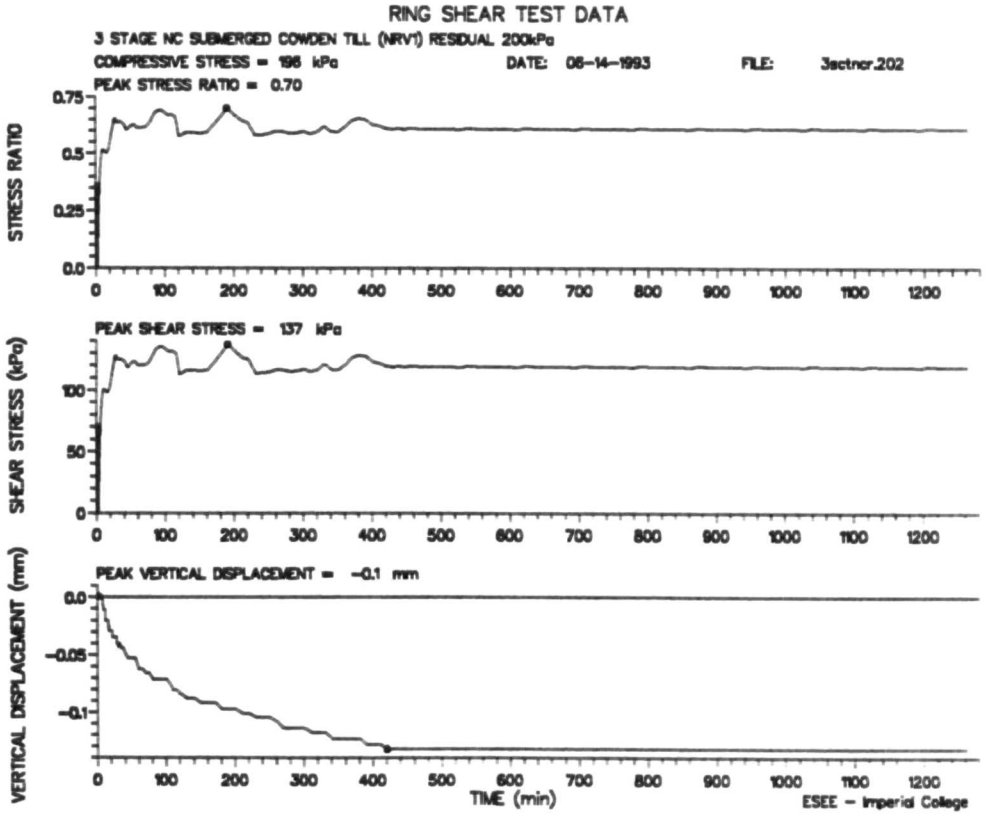


Fig. VI.4.1 First slow shearing test on normally consolidated, submerged, Cowden till (200 kPa - 3 stage test)

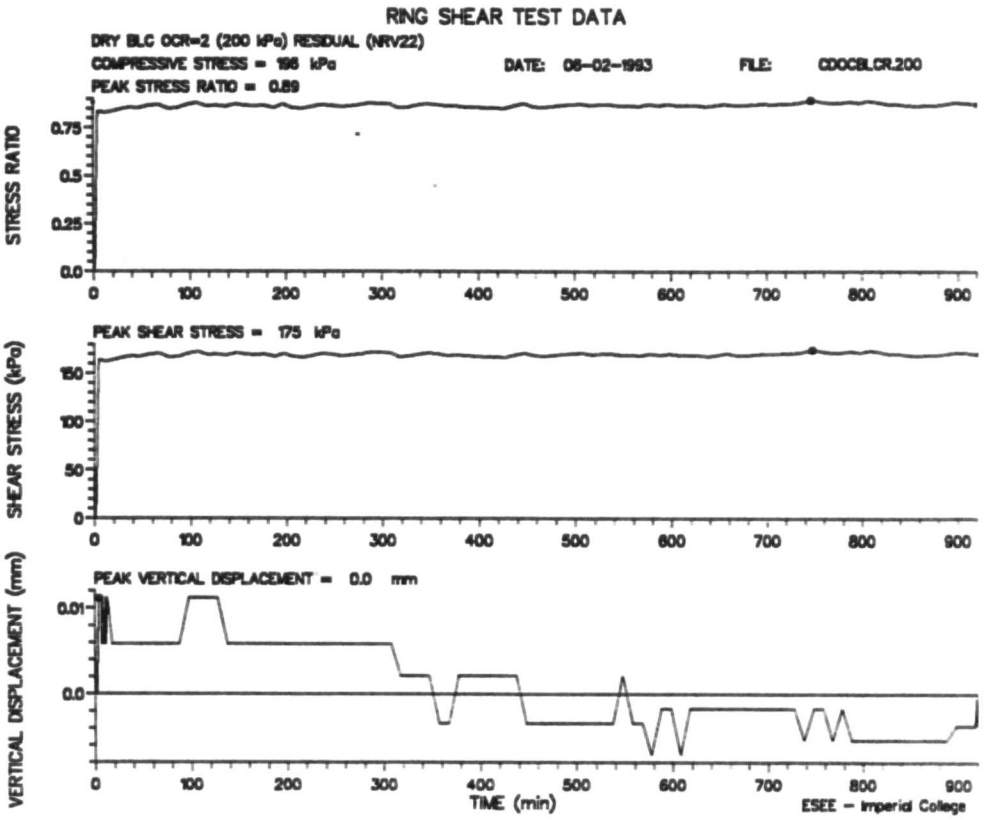


Fig. VI.8.1 Slow shearing test on over-consolidated, completely dry, brown London clay (200 kPa - OCR= 2)

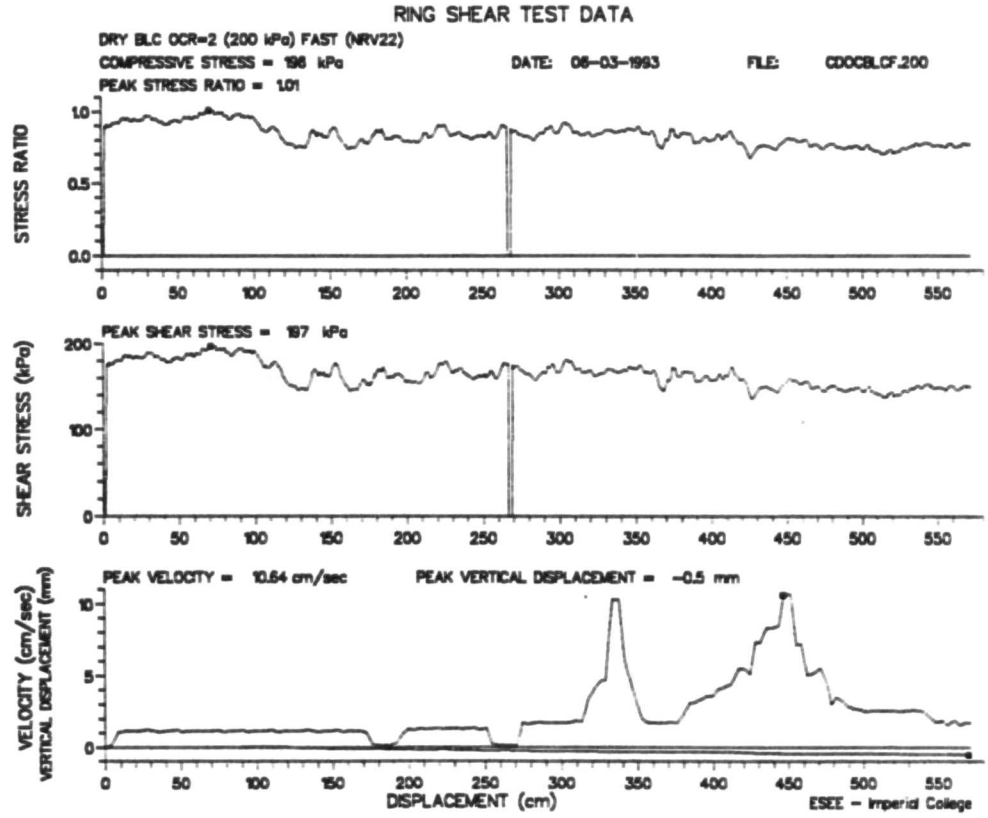


Fig. VI.8.2 Fast shearing test on over-consolidated, completely dry, brown London clay (200 kPa - OCR= 2)

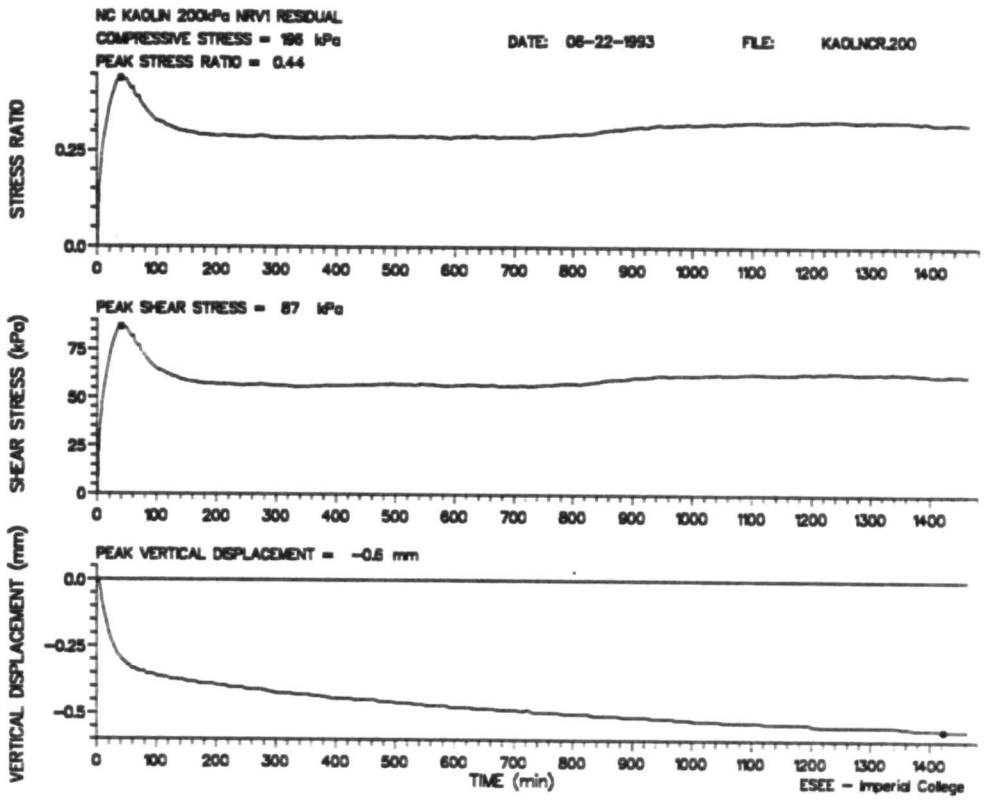


Fig. VI.13.1 Slow shearing test on normally consolidated, submerged, kaolin (200 kPa)

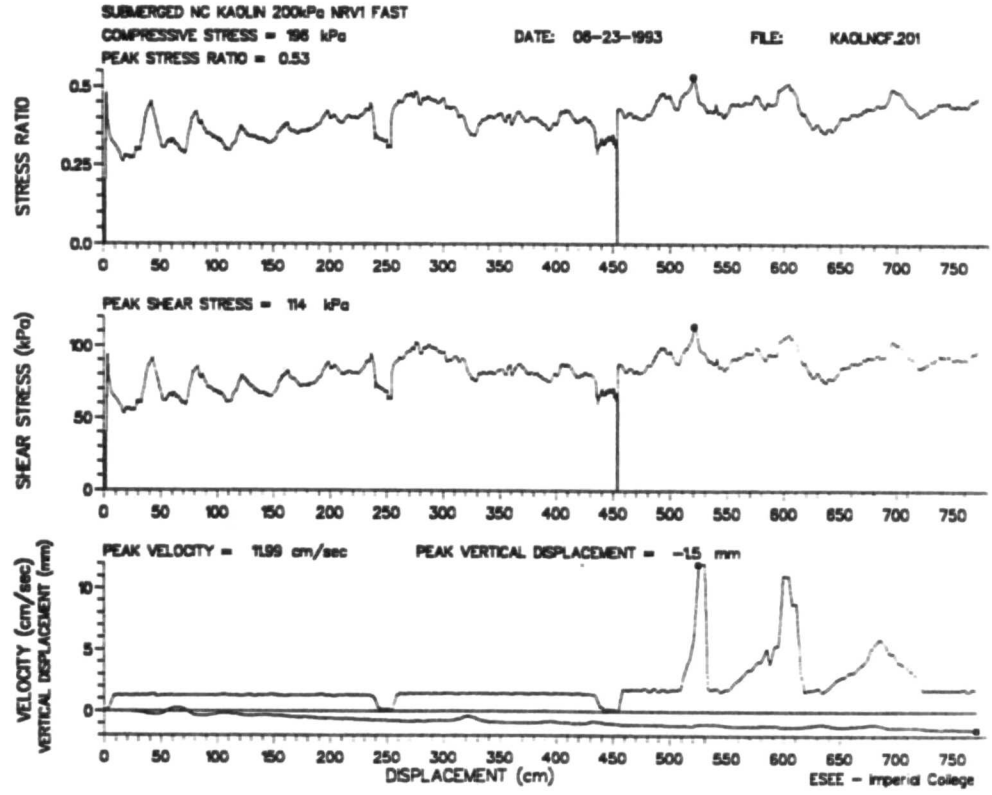


Fig. VI.13.2 Fast shearing test on normally consolidated, submerged, kaolin (200 kPa)

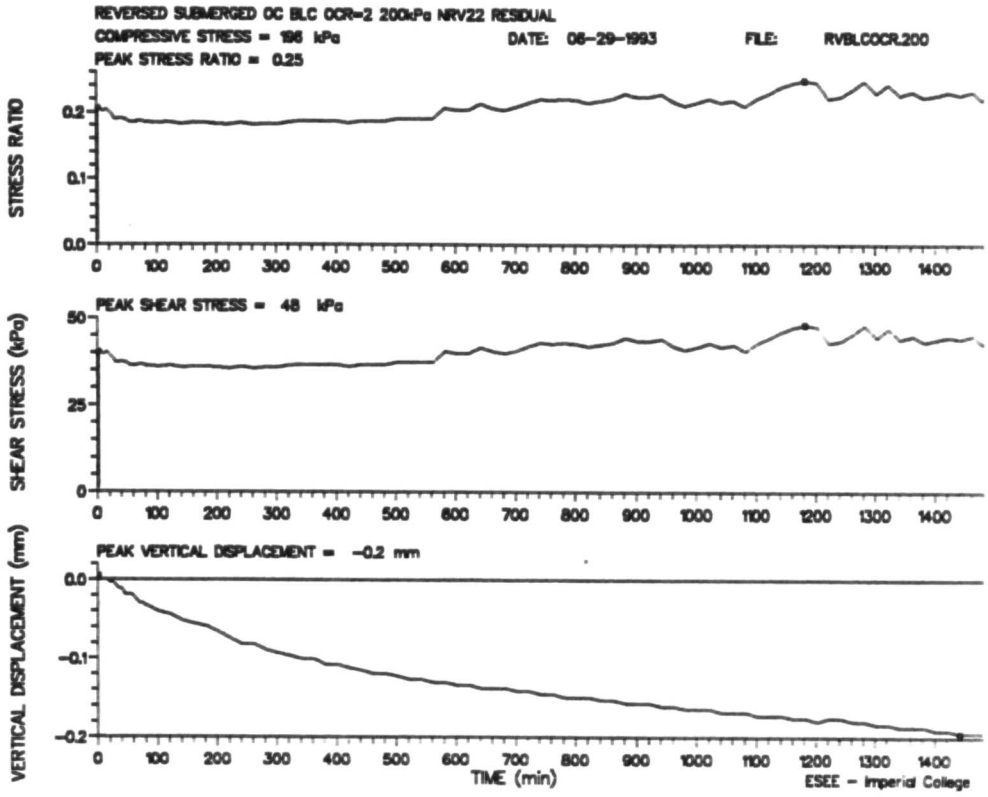


Fig. VI.20.1 Slow shearing test on over-consolidated, submerged, brown London clay (200 kPa - reversed direction)

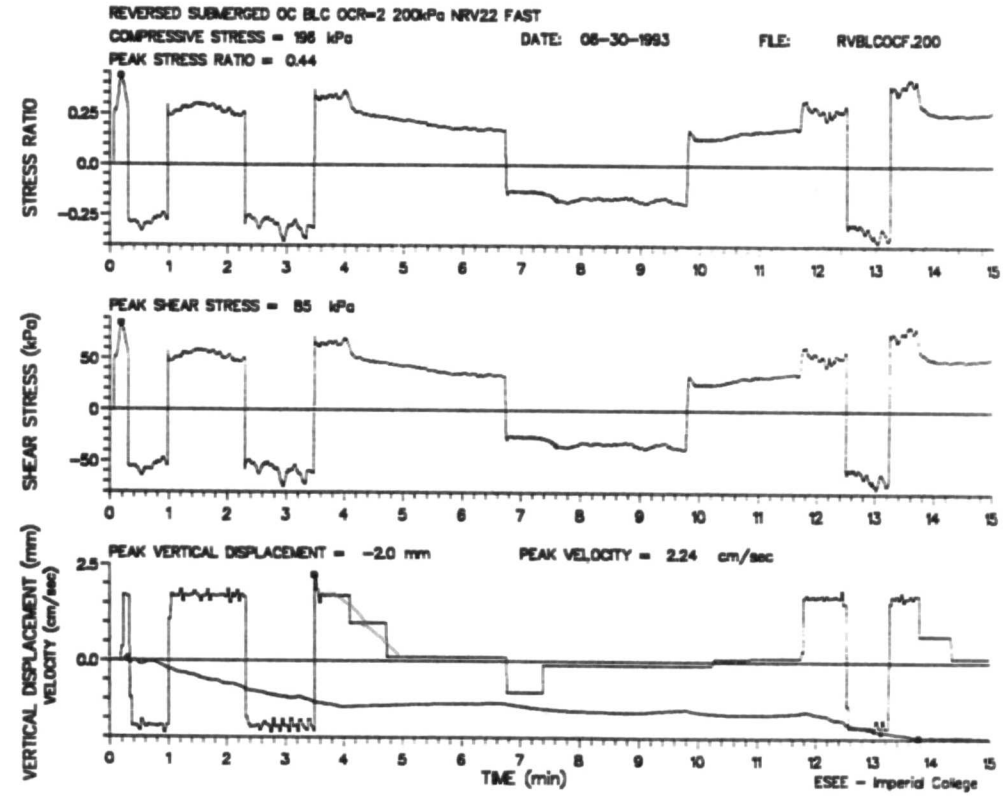


Fig. VI.20.2 Fast shearing test on over-consolidated, submerged, brown London clay (200 kPa - reversed direction)

of displacement was decreased suddenly to 0.158 cm/sec for the next 20.1 cm of shearing while the measured strength appeared to be steadily increasing from 8.35° to 14.0°. After the sample had been sheared for 199 cm, the rate of displacement was increased suddenly to 1.26 cm/sec, and the measured strength fluctuated again between 8.35° and 14.0°. The test was terminated after the sample had been sheared for 425 cm. It should be mentioned that, throughout the duration of this test, the shape of the sample height curve revealed the existence of an irregular shear surface, and that was reflected on the measured strength.

Test VI.18

σ_n= 200 kPa

$\phi_{threshold}$= 28.8°

(Figure VI.18.1)

The same sample was sheared at 1.28 cm/sec, and the measured strength reached immediately its threshold value of 28.8°. Continued shearing was associated with an decrease in strength while the sample height curve indicated the development of an undulating shear surface with a wave-length of 40 cm and an amplitude which varied between 0.48 mm and 0.36 mm. After the sample had been sheared for 35.3 cm the measured strength had decreased to 19.4°. As shearing at 1.28 cm/sec continued, the measured strength fluctuated between 20.9° and 25.8° before it began to decrease. After the sample had been sheared for 229.4 cm the amplitude of the undulating shear surface increased to 0.57 mm and the measured strength decreased even further to stabilise within a range between 7.24° and 10.04°. The rate of displacement was decreased from 1.4 cm/sec to 0.138 cm/sec and then increased to 1.28 cm/sec before the test was terminated at 390 cm of cumulative shear displacement. However, no effect on the measured sample strength was observed.

Reversible Direction Tests:

Brown London clay

Overconsolidated, Submerged

Test VI.19

σ_n= 100 kPa

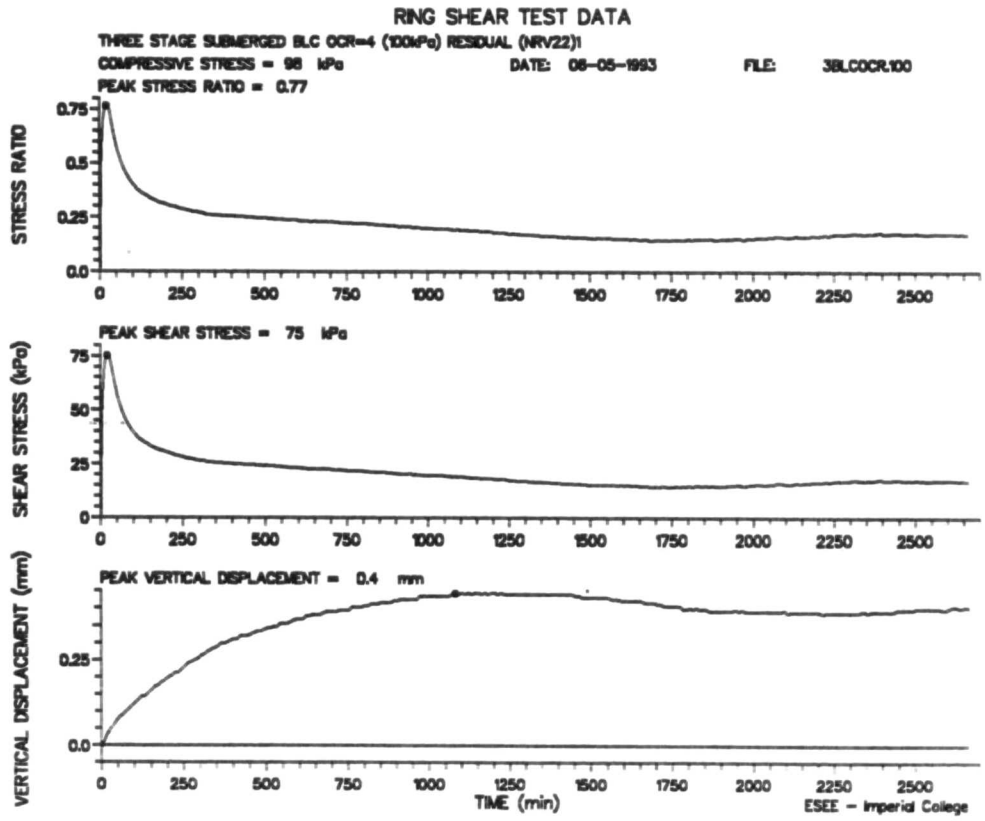


Fig. VI.1.1 First slow shearing test on over-consolidated, submerged, brown London clay (100 kPa - 3 stage test)

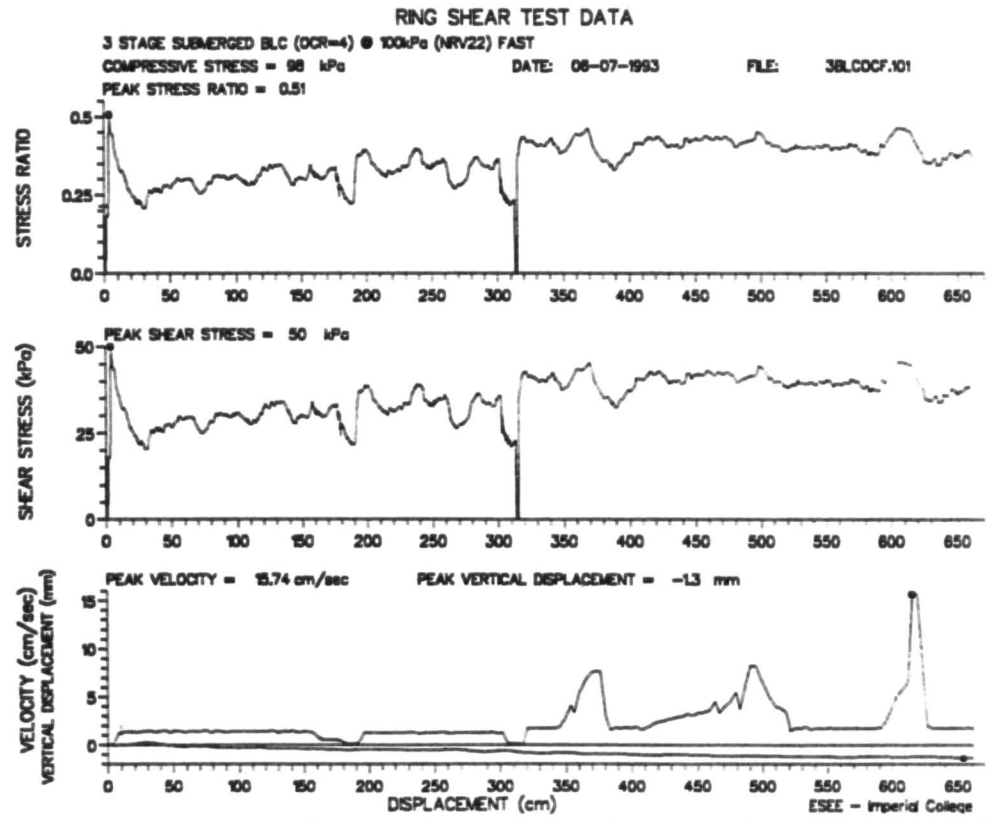


Fig. VI.1.2 Fast shearing test on over-consolidated, submerged, brown London clay (100 kPa - 3 stage test)

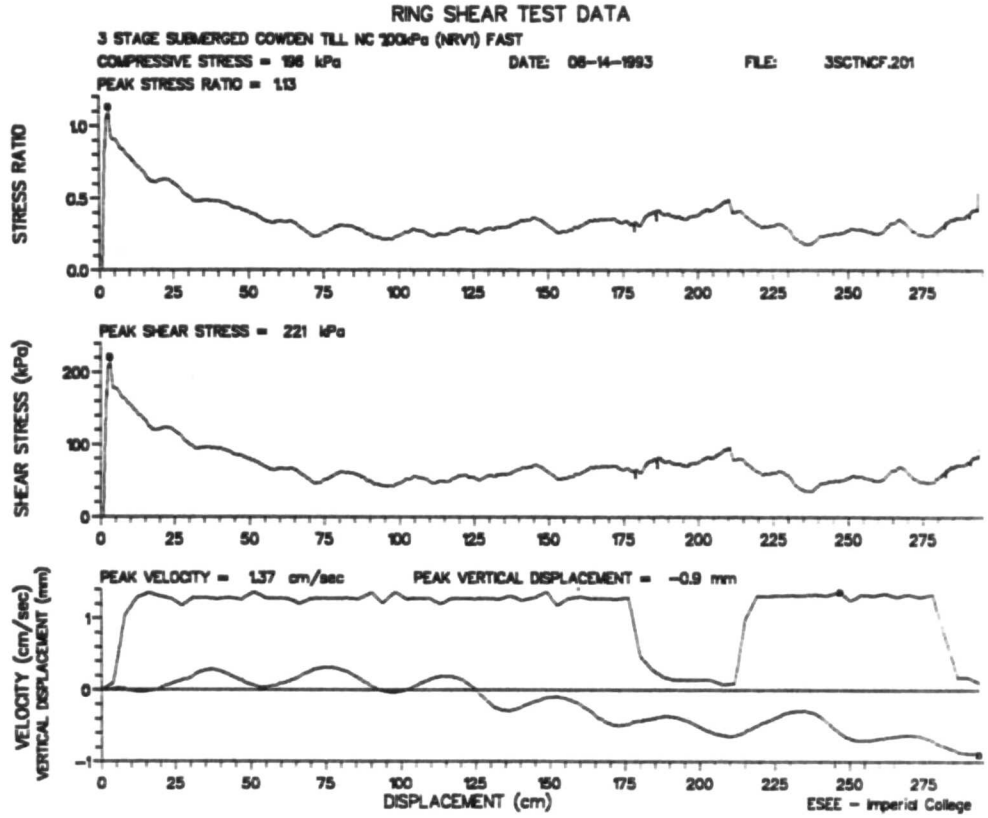


Fig. VI.4.2 Fast shearing test on normally consolidated, submerged, Cowden till (200 kPa - 3 stage test)

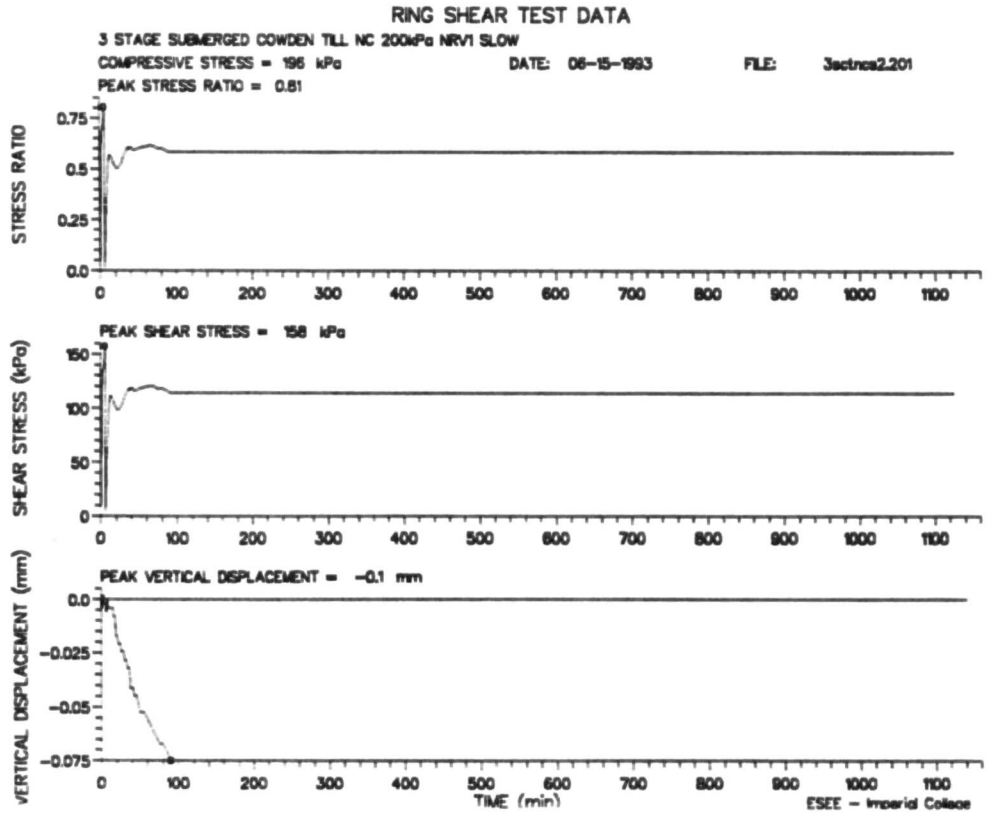


Fig. VI.4.3 Second slow shearing test on normally consolidated, submerged, Cowden till (200 kPa - 3 stage test)

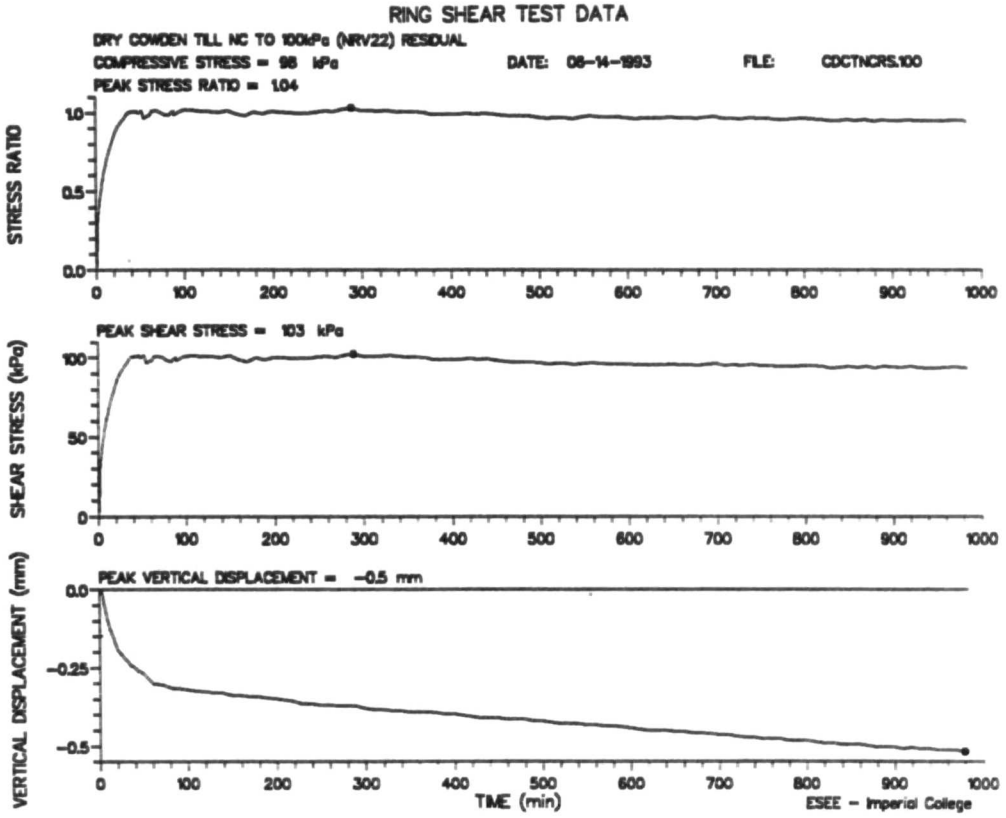


Fig. VI.9.1 Slow shearing test on normally consolidated, completely dry, Cowden till (100 kPa)

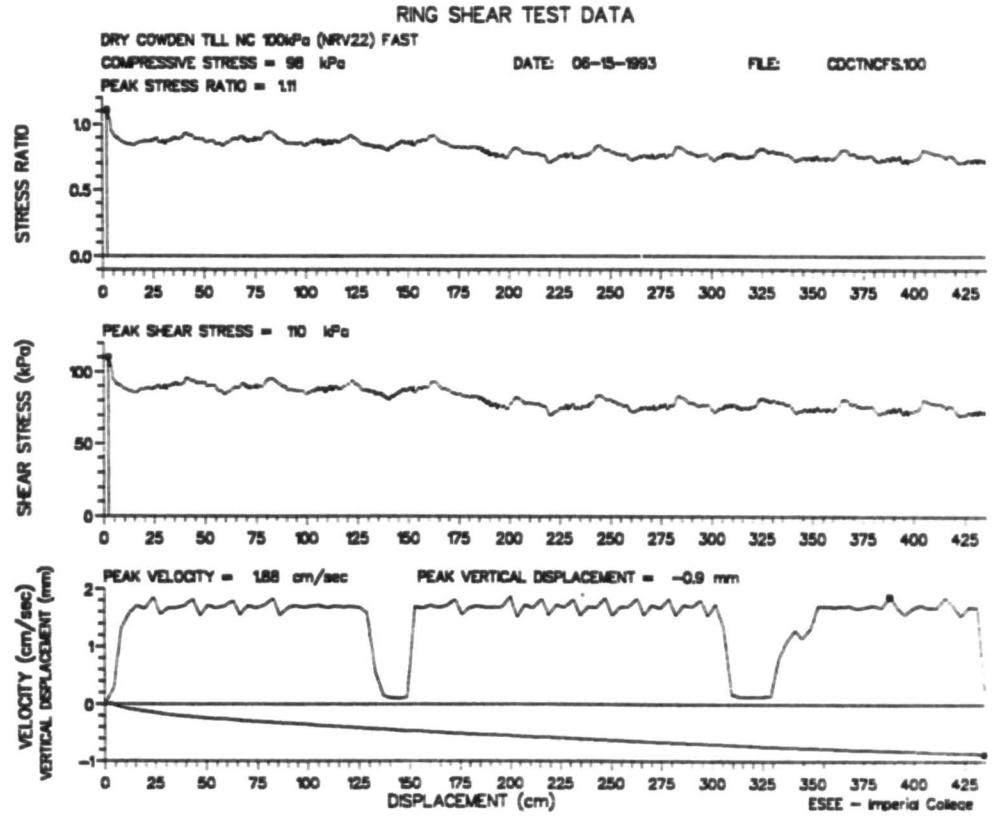


Fig. VI.9.2 Fast shearing test on normally consolidated, completely dry, Cowden till (100 kPa)

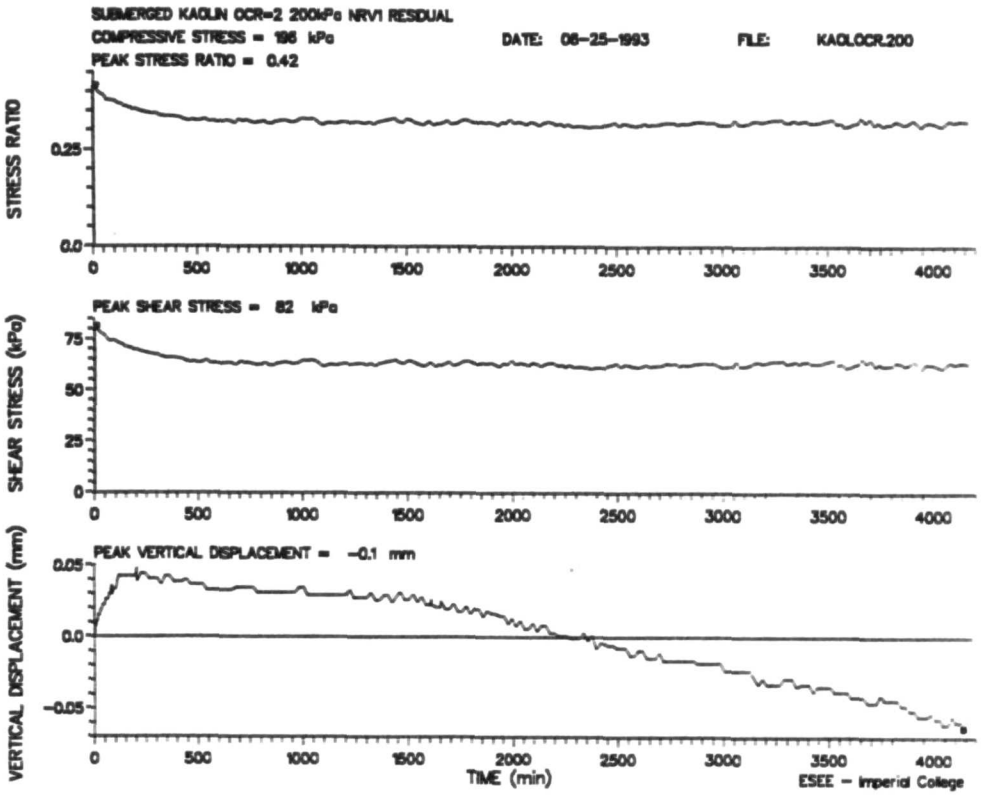


Fig. VI.14.1 Slow shearing test on over-consolidated, submerged, kaolin (200 kPa - OCR= 2)

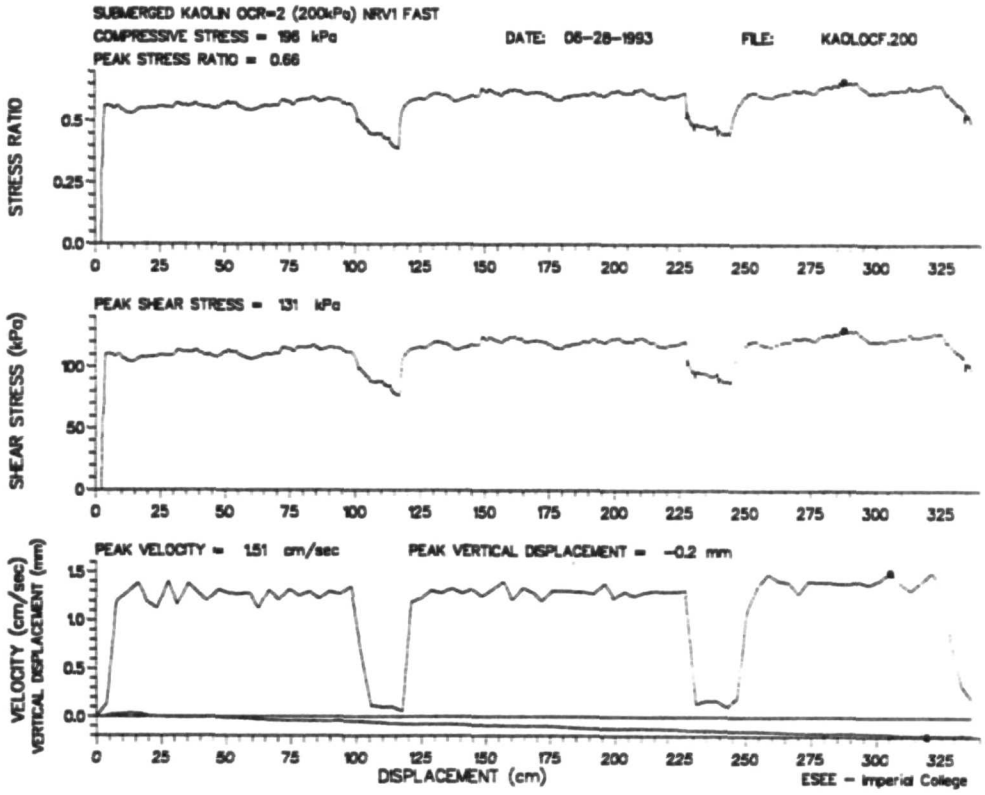


Fig. VI.14.2 Fast shearing test on over-consolidated, submerged, kaolin (200 kPa - OCR= 2)

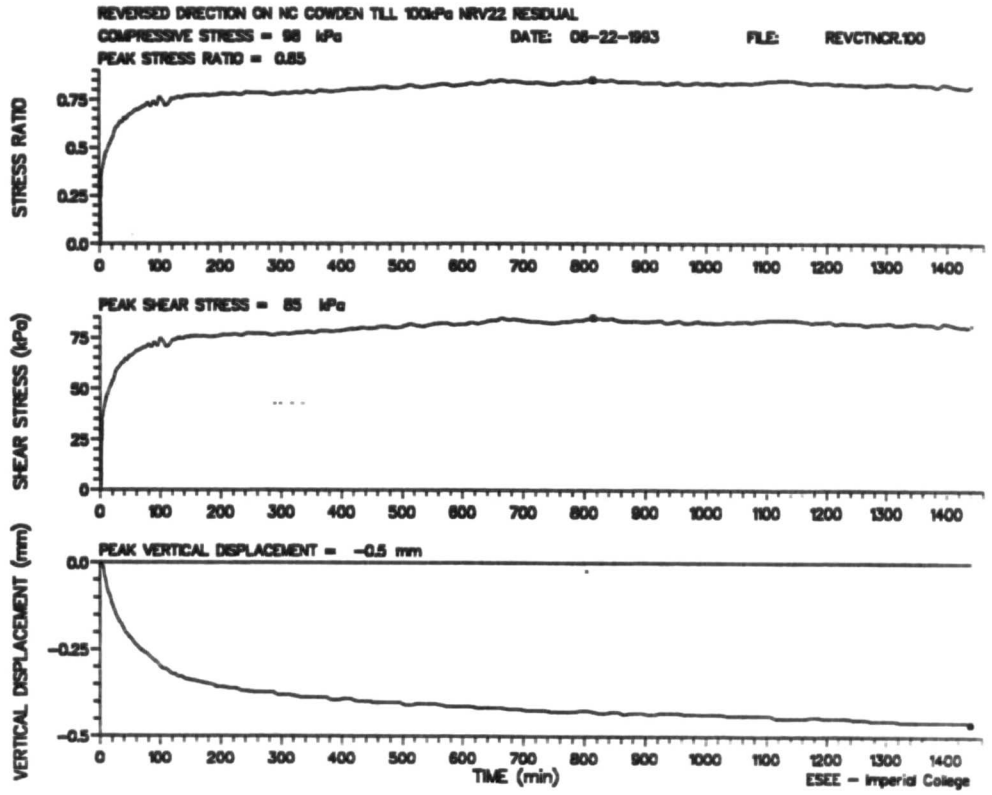


Fig. VI.21.1 Slow shearing test on normally consolidated, submerged, Cowden till (100 kPa - reversed direction)

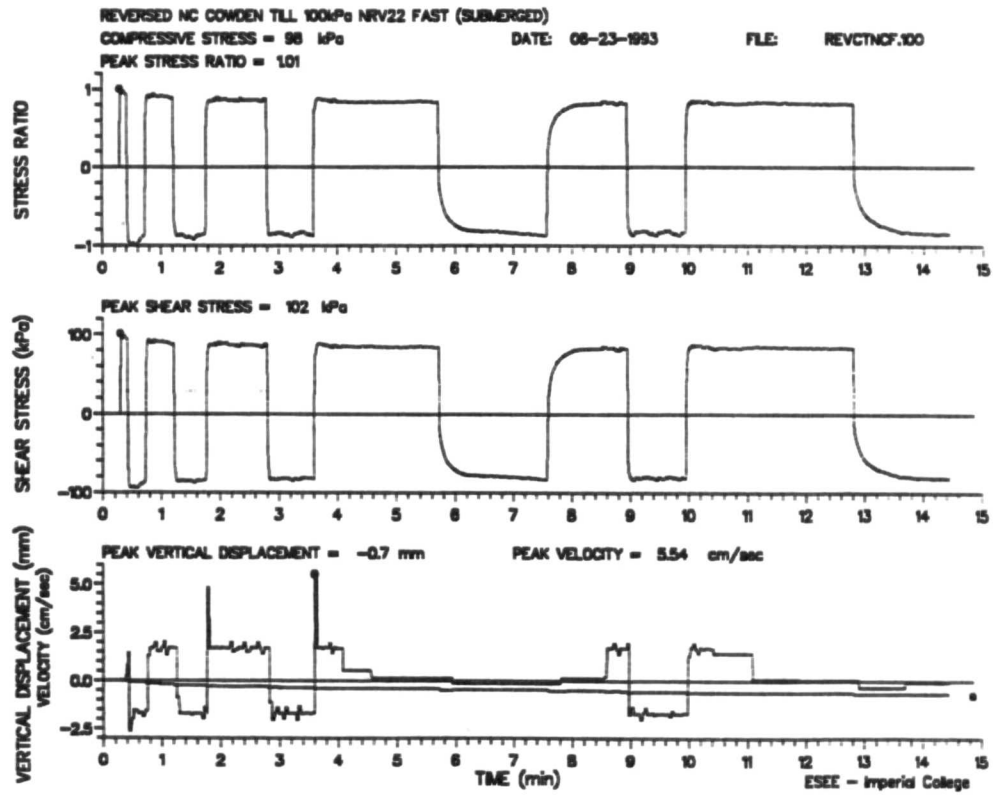


Fig. VI.21.2 Fast shearing test on normally consolidated, submerged, Cowden till (100 kPa - reversed direction)

OCR= 4

$\phi_{residual}$= 10.4°

$\phi_{threshold}$= 28.9°

(Figures VI.19.1-2)

The sample was sheared at 0.09 mm/min, and the measured strength reached its slow peak value of 36.9° after 2.25 mm of shearing while its height increased 0.045 mm. After the sample had been sheared for 100.7 mm, its height had increased by 0.31 mm, and its measured strength had reached its slow residual value of 10.4°. This part of the test was terminated after 123.5 mm of shearing.

The rate of displacement was increased suddenly to 1.74 cm/sec and the measured strength reached immediately its threshold value of 28.4°. After 0.28 min of shearing at the same rate of displacement, in the **positive** (clockwise) direction, the measured sample strength had decreased to 20.3°. At that point the direction of shearing was reversed for the first time while the rate of displacement was kept constant. The measured strength, upon reversing the direction of shearing, increased slightly to 22.6° but dropped back to 20.3° just before the direction of shearing was reversed again. The direction of shearing was reversed from **negative** (anti-clockwise) to positive after 0.67 min of shearing, and the measured strength increased again from 20.3° to 22.6°. The direction and rate of shearing were kept constant for a further 1.66 min while the measured sample strength fluctuated between 24.4° and 17.2°. After 2.33 min of cumulative shearing at 1.74 cm/sec the direction of shearing was once more reversed and the measured sample strength increased from 17.2° in the positive direction to 20.9° in the negative. The sample was sheared in the negative direction, at 1.74 cm/sec, for 1.5 min while its strength varied between 22.0° and 16.9°. The direction of shearing was reversed again from negative to positive after 3.83 min of shearing and the measured strength increased from 16.9° to 20.8°. Further shearing in the positive direction at 1.74 cm/sec had no significant effect on the measured strength which stabilised within a range between 20.8° and 22.6°. The rate of displacement was decreased gradually, in the positive direction, from 1.74 cm/sec, to 0.63 cm/sec and to 0.14 cm/sec while the measured sample strength also decreased and stabilised at 15.5°. Reversing the direction of shearing from positive to negative at 0.20 cm/sec was associated with a gradual decrease of the measured strength from 15.2°, immediately after the reversal of direction, to 10.4° when the direction was reversed again to positive. Further shearing at rates of

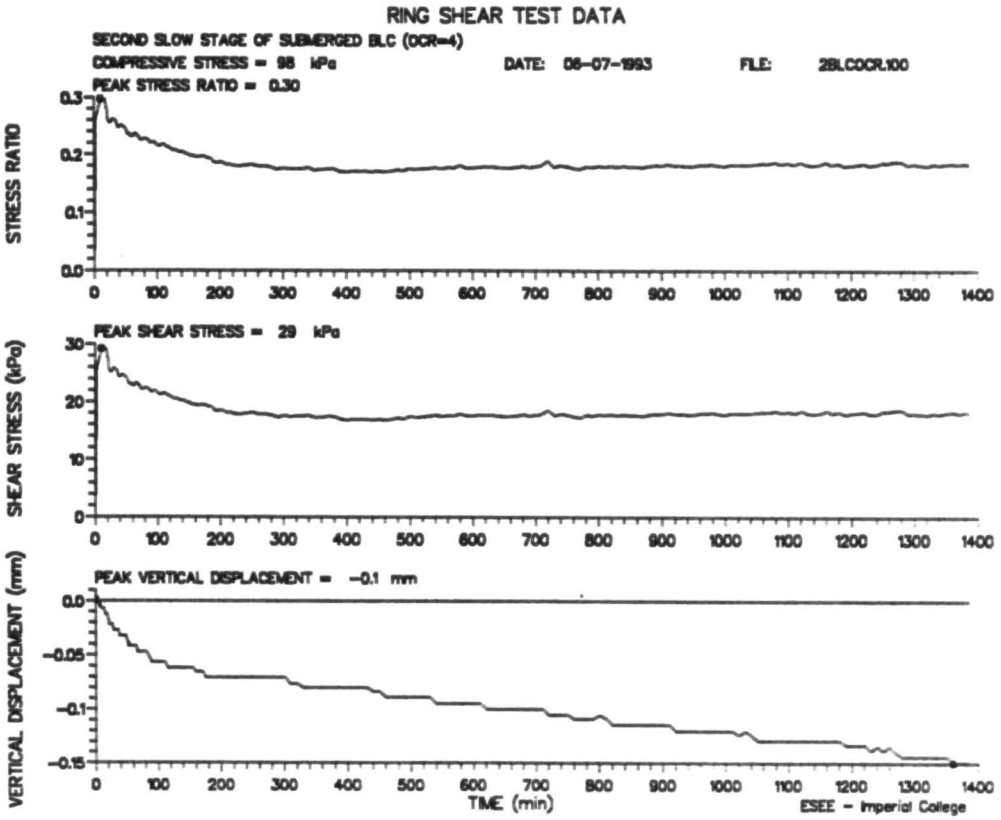


Fig. VI.1.3 Second slow shearing test on over-consolidated, submerged, brown London clay (100 kPa - 3 stage test)

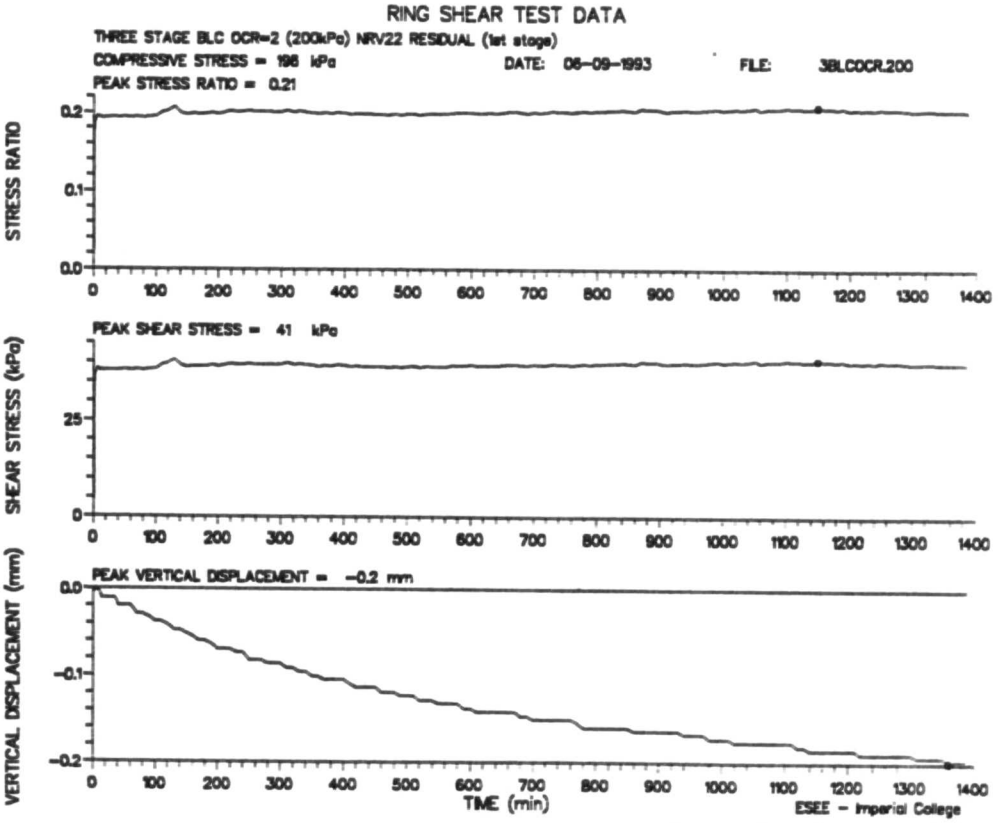


Fig. VI.2.1 First slow shearing test on over-consolidated, submerged, brown London clay (200 kPa - 3 stage test)

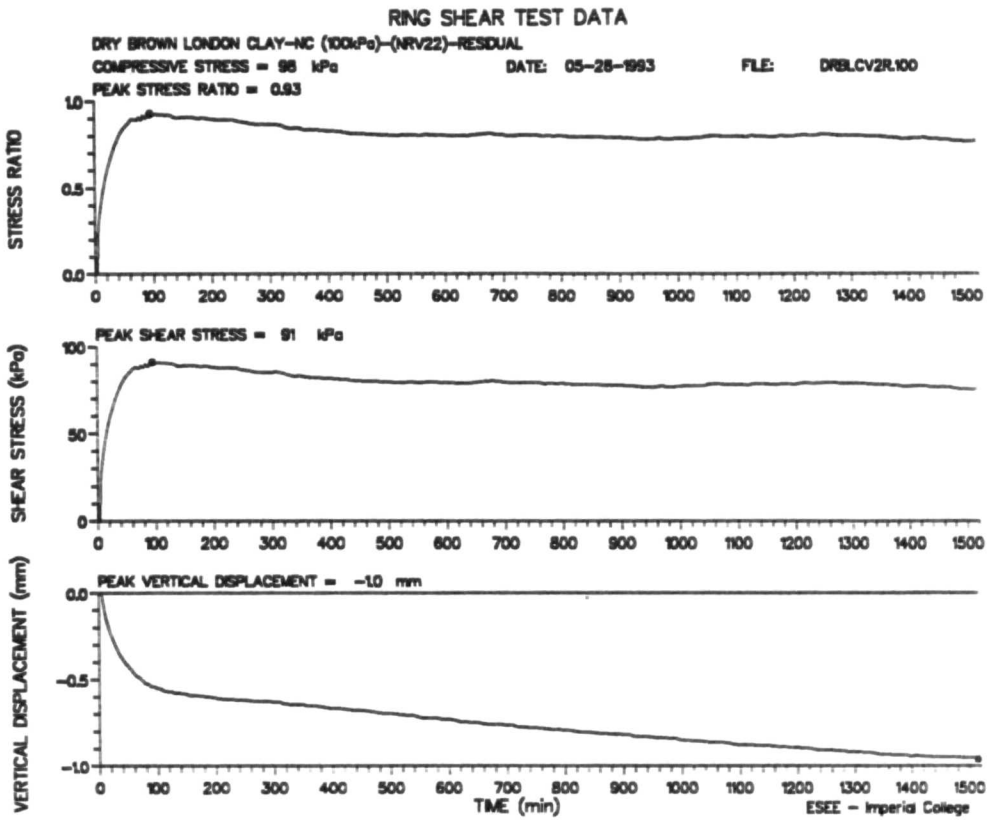


Fig. VI.5.1 Slow shearing test on normally consolidated, completely dry, brown London clay (100 kPa)

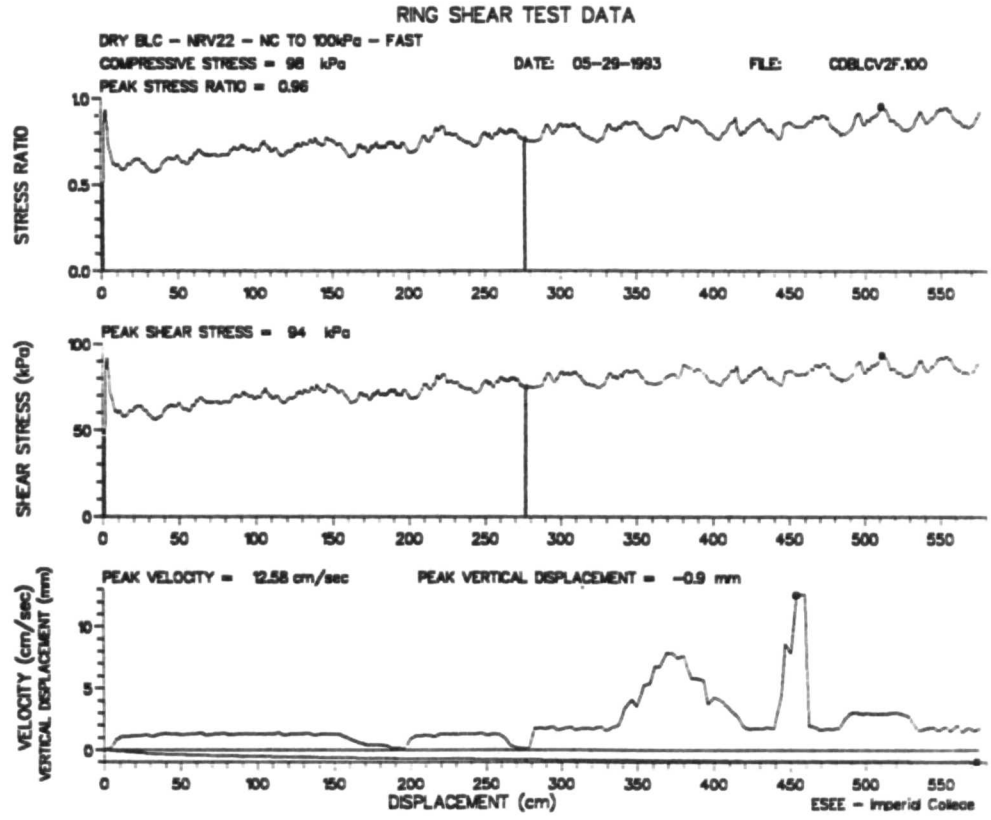


Fig. VI.5.2 Fast shearing test on normally consolidated, completely dry, brown London clay (100 kPa)

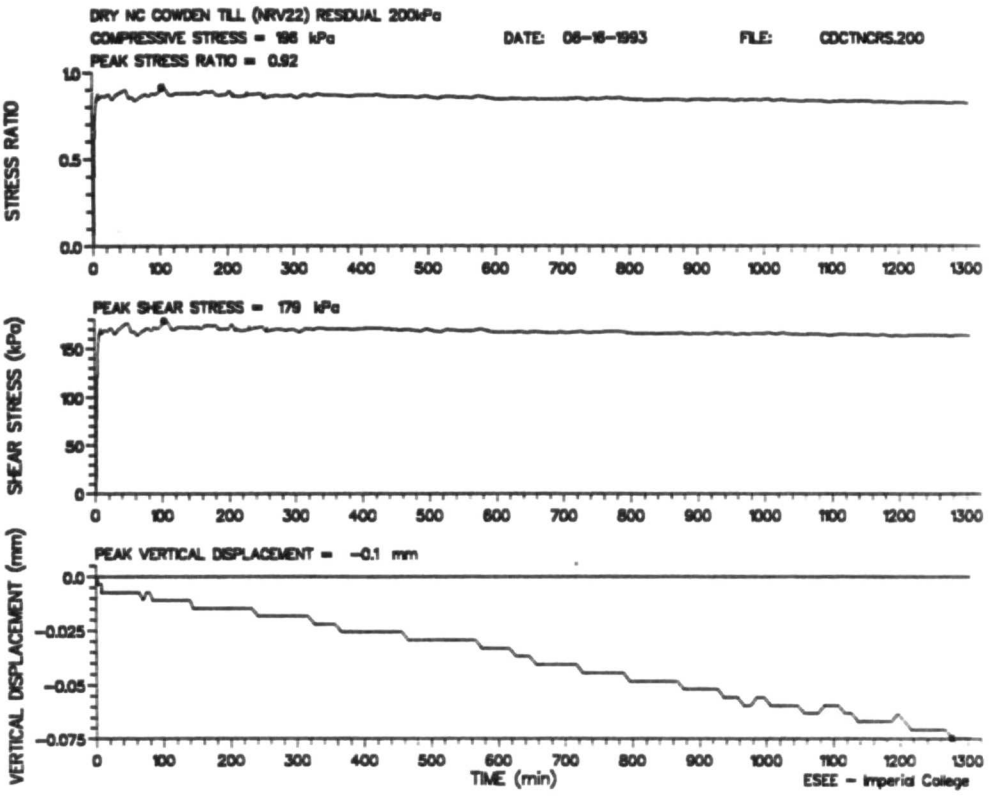


Fig. VI.10.1 Slow shearing test on normally consolidated, completely dry, Cowden till (200 kPa)

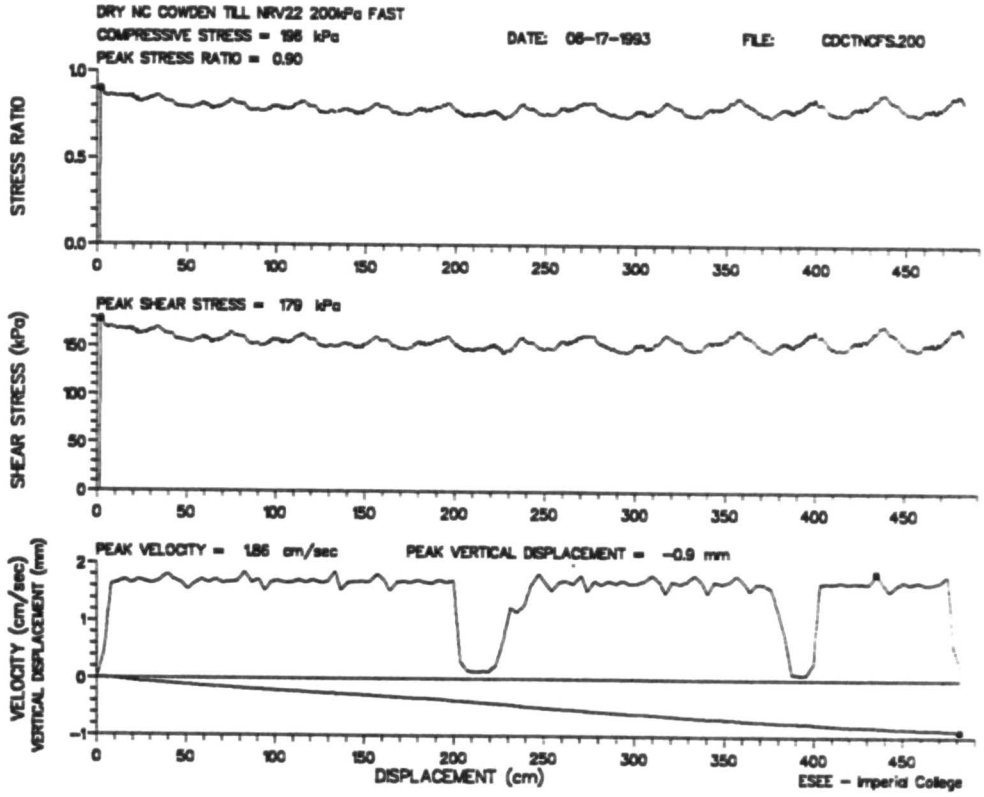


Fig. VI.10.2 Fast shearing test on normally consolidated, completely dry, Cowden till (200 kPa)

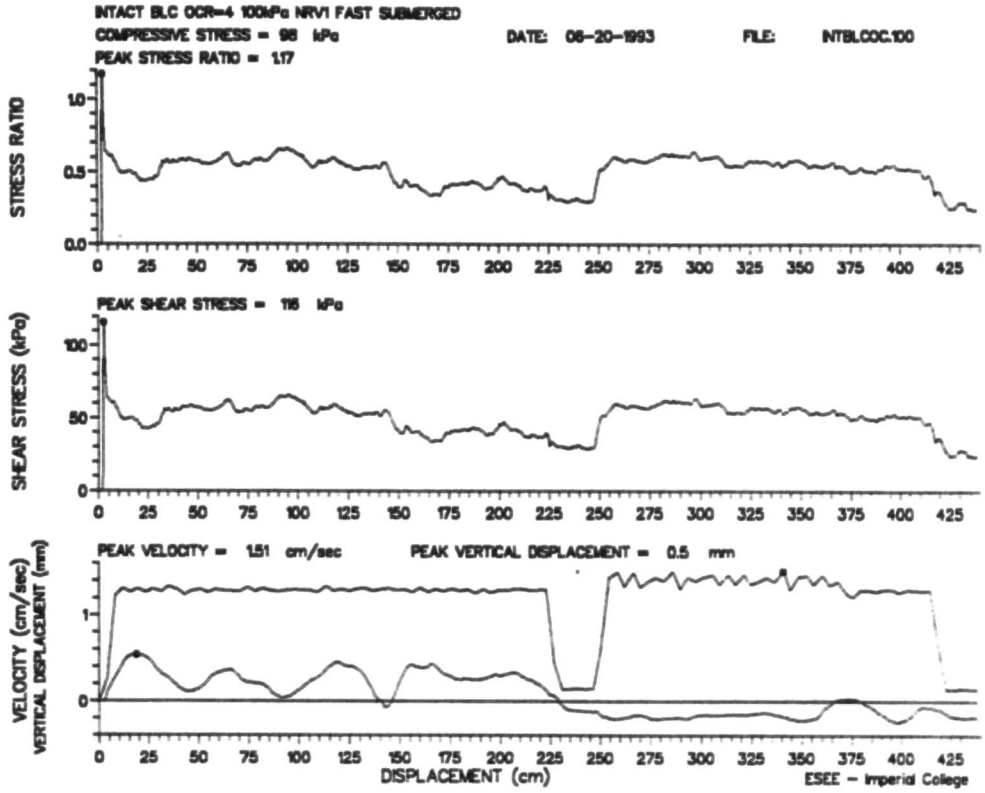


Fig. VI.15.1 Fast shearing test on "intact", over-consolidated, submerged, brown London clay (100 kPa - OCR= 4)

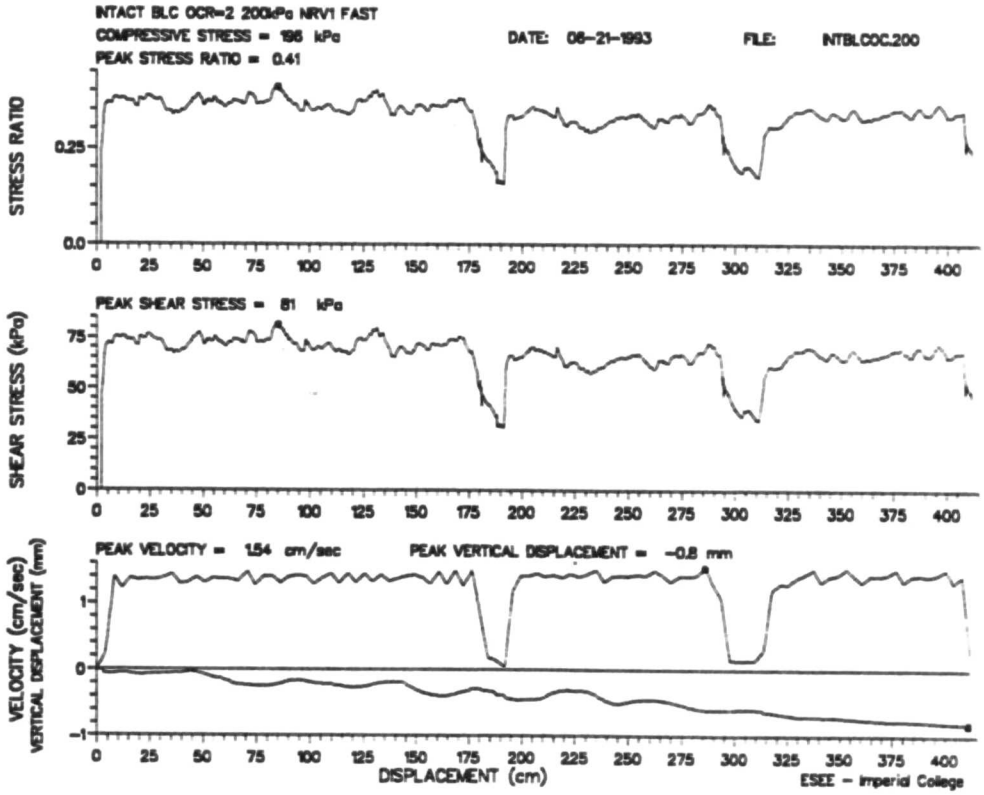


Fig. VI.16.1 Fast shearing test on "intact", over-consolidated, submerged, brown London clay (200 kPa - OCR= 2)

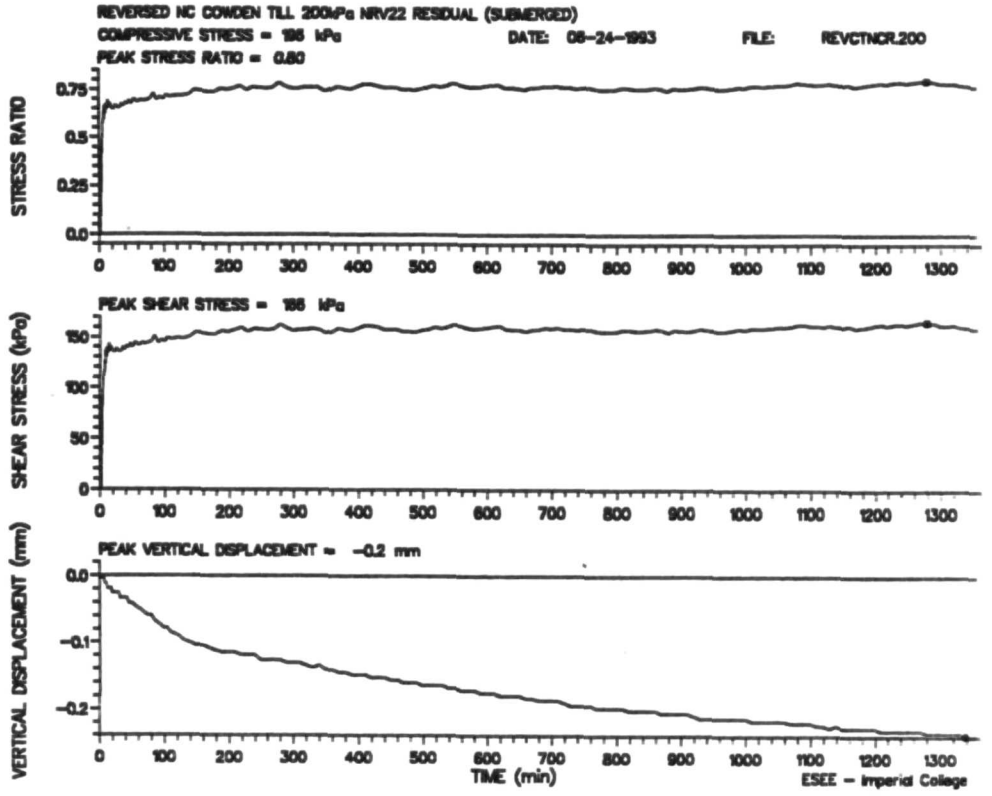


Fig. VI.22.1 Slow shearing test on normally consolidated, submerged, Cowden till (200 kPa - reversed direction)

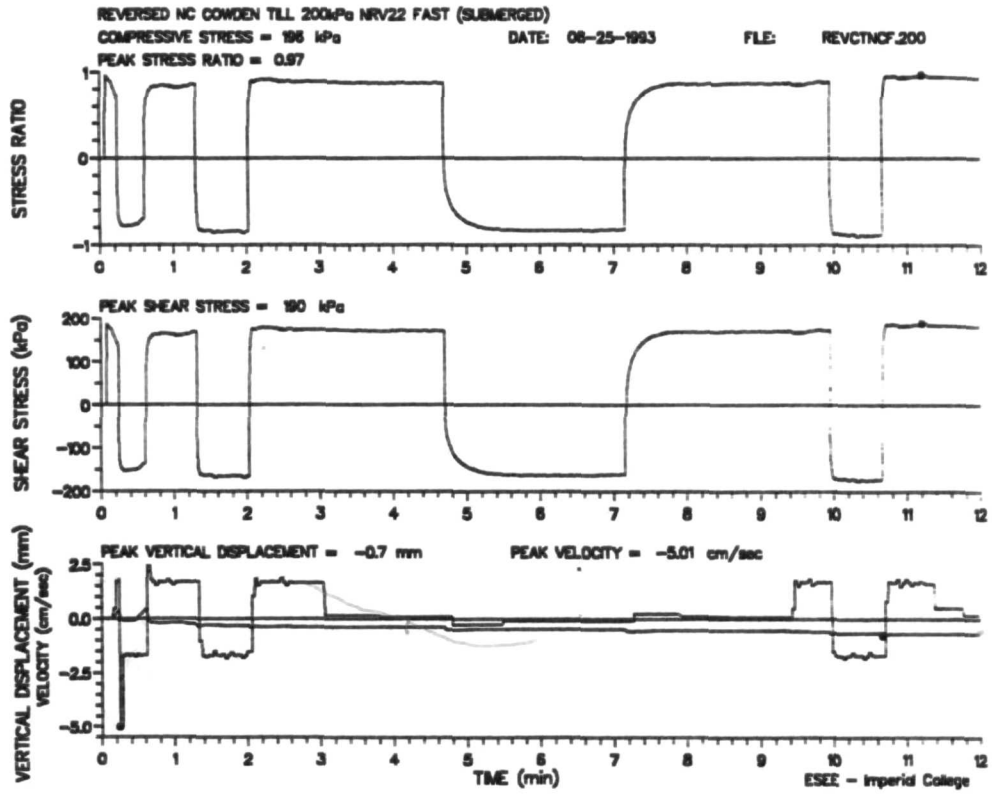


Fig. VI.22.2 Fast shearing test on normally consolidated, submerged, Cowden till (200 kPa - reversed direction)

7.2 Recommendations for Future Research

The future research on this subject should concentrate on the immediate response of soils upon the application of fast shearing. The threshold strength and the post-threshold strength decrease must be investigated further, and their importance in engineering practice must be assessed. With this in mind, the confining rings could be further modified in order to eliminate soil loss and to allow volume change measurements. In addition, the testing procedure should involve a gradual, uninterrupted increase in the rate of displacement from the drained to the fast values. This investigation has shown that the shear displacement associated with each fast shearing stage does not need to exceed 120 cm, and that particular attention should be concentrated on rates of displacement from 0 to 2.0 cm/sec, since any further increase does not appear to have any significant effect on soil strength. Pore pressure and side friction measurements are essential to understand fully the behaviour of soils during fast shearing, and the instrumentation necessary to achieve these measurement must be considered. In relation to further modifications of the IC/NGI ring shear apparatus, it is believed that an automatically controlled motor which would enable a broad, uninterrupted variation of the rate of displacement is required.

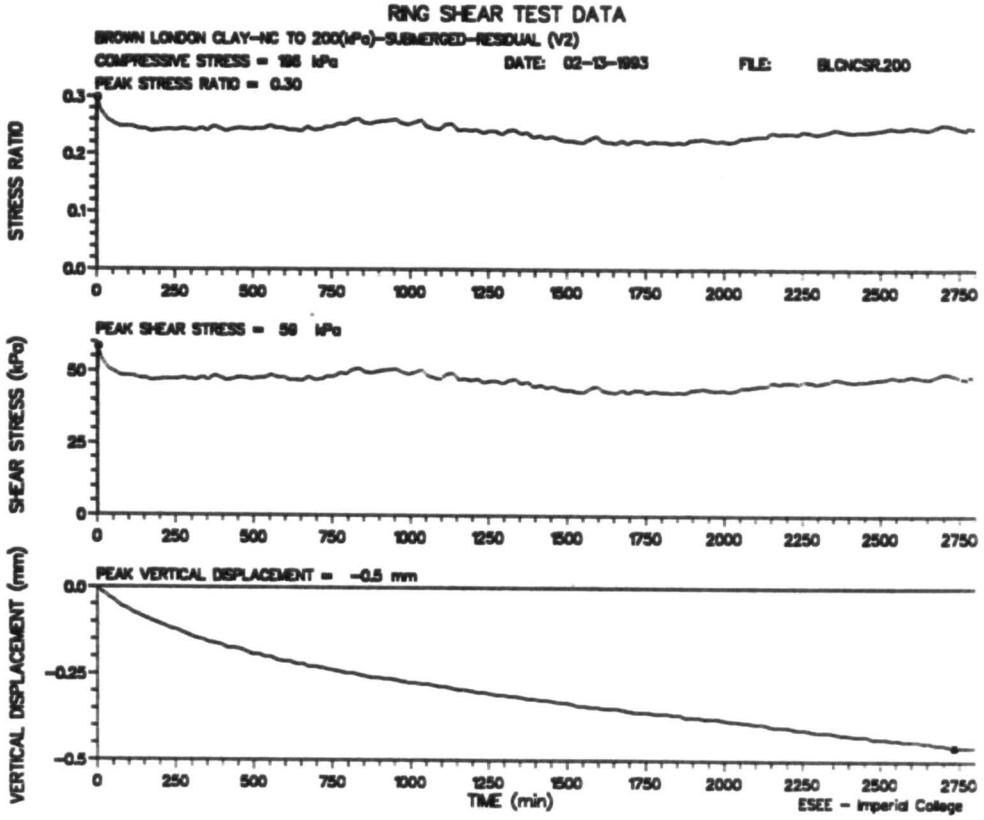


Fig. IV.9.1 Slow shearing test on normally consolidated, submerged brown London clay (200 kPa)

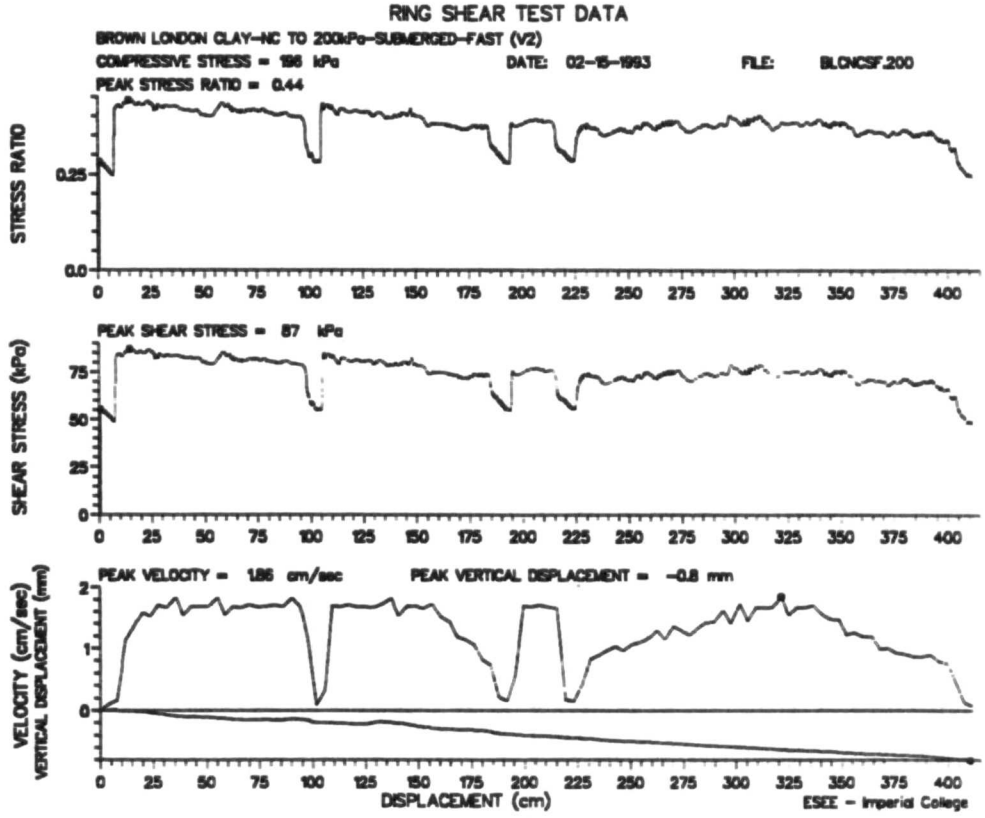


Fig. IV.9.2 Fast shearing test on normally consolidated, submerged brown London clay (200 kPa)

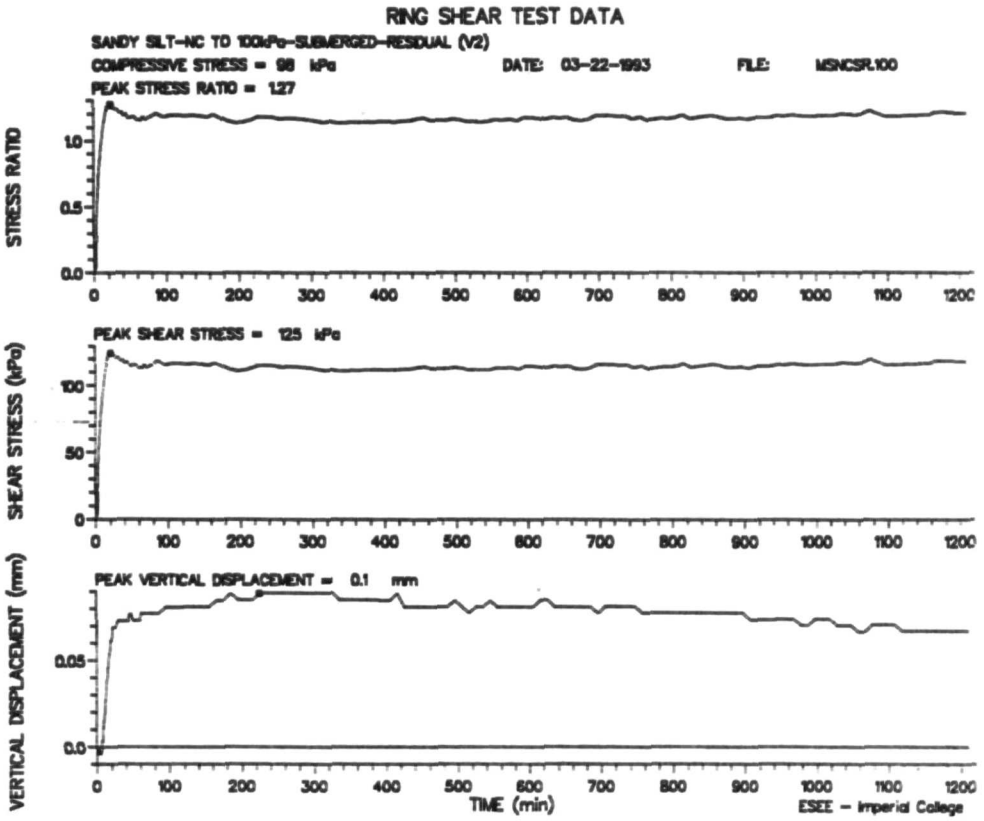


Fig. V.17.1 Slow shearing test on normally consolidated, submerged artificial sandy-silt (100 kPa)

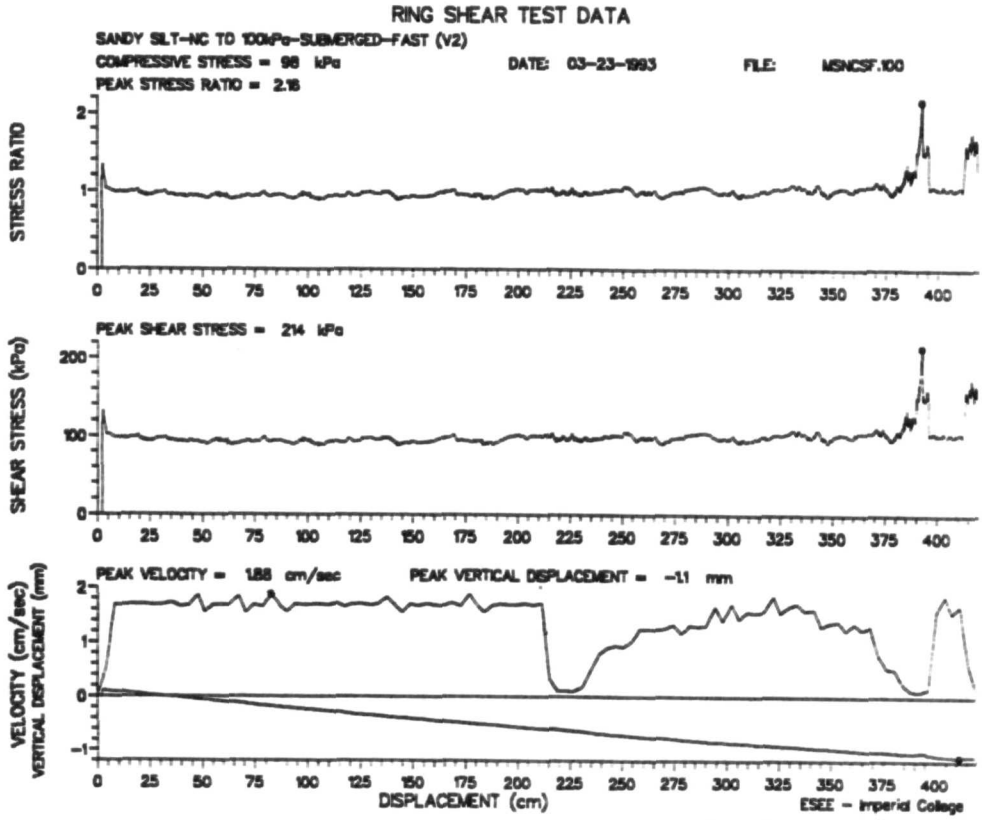


Fig. V.17.2 Fast shearing test on normally consolidated, submerged artificial sandy-silt (100 kPa)

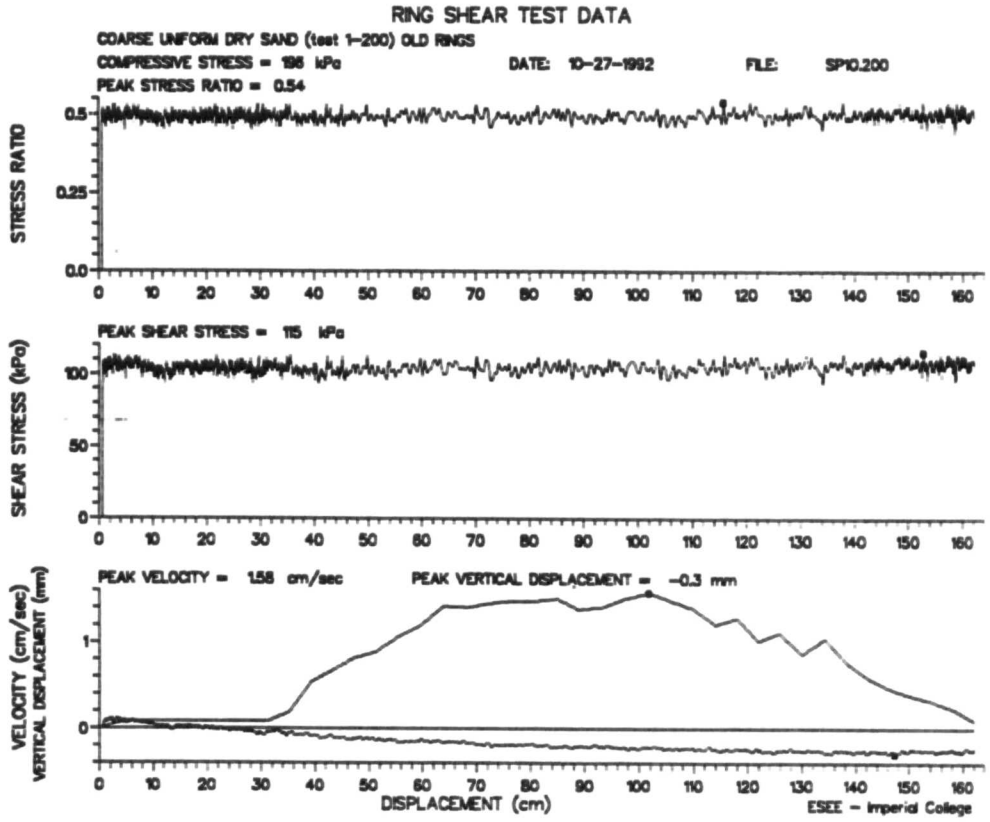


Fig. V.23.1 Fast shearing test on normally consolidated, dry medium-coarse sand (200 kPa)

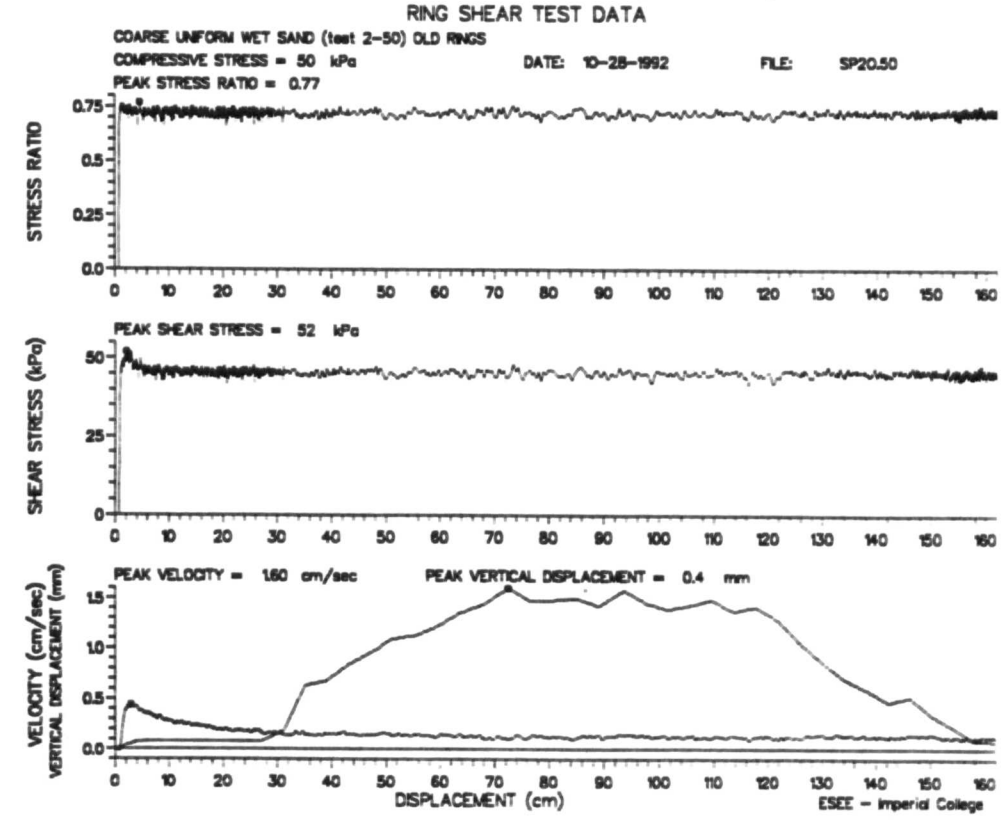


Fig. V.24.1 Fast shearing test on normally consolidated, wet, non-submerged medium-coarse sand (50 kPa)

and stabilised at 1.25 cm/sec after 38.9 cm- the strength of the sample dropped to 17.9° after 38.9 cm and stabilised at 19.0° until the rate of displacement was decreased after 149.5 cm. Between 149.5 cm and 180.6 cm of cumulative shear displacement the rate of displacement was decreased gradually from 1.25 cm/sec to 0.125 cm/sec. The measured strength began to decrease only after the rate of displacement had dropped below 1.04 cm/sec and exhibited positive rate characteristics. At 0.58 cm/sec the measured strength was 17.2°, at 0.46 cm/sec it was 16.4° and at 0.125 cm/sec it was 14.1°. The rate of displacement was suddenly increased back to 1.25 cm/sec were it remained for the following 102.3 cm of shearing and the strength of the sample stabilised within a range between 19.6° and 20.8°. After 291.8 cm of cumulative shearing the rate of displacement was decreased gradually. At 0.46 cm/sec the measured strength was 17.2° and at 0.125 cm/sec it was 14.7°. Therefore, it appeared that the shear displacement had no effect on the measured strength. After 305.3 cm of shearing the rate of displacement was increased suddenly to 1.75 cm/sec and the measured strength increased to 20.9°. Further shearing was associated with a sudden increase of the rate of displacement to 8.44 cm/sec and an increase in strength to 25.2°. When the rate of displacement was decreased back to 1.75 cm/sec the strength dropped to 21.2°. The following gradual increase in the rate of displacement from 1.74 cm/sec to 7.92 cm/sec within 85.2 cm of shearing indicated that strength of the sample continued to increase with the rate of displacement. At 3.54 cm/sec the measured strength was 23.1°, at 5.42 cm/sec it was 24.5° and at 7,92 cm/sec it was 26.1°. The rate of displacement was decreased back to 1.75 cm/sec and the strength of the sample dropped to 21.2°. This fast shearing part of the test was terminated after the sample had been sheared for 600 cm and throughout its duration no evidence of an undulating shear surface were apparent.

Subsequently, the rate of displacement was decreased to 0.09 mm/sec for the third part of this test. After 20.9 cm of shearing at the same rate of displacement the measured strength had decreased to 11.0°, its slow residual value, where it remained until the test was terminated after a displacement of 138.7 cm.

$\phi_{residual}$= 42.1°

$\phi_{threshold}$= 44.7°

(Figures VI.7.1-2)

The same sample which was sheared in the previously described tests was consolidated to 400 kPa, unloaded to 100 kPa, and sheared at 0.09 mm/min. The measured strength reached its slow peak value of 45.0° while the sample height appeared to be increasing as the overconsolidated structure relaxed. The measured strength reached its slow residual value of 42.1° after 76 mm of shearing, while the sample height increased by 0.055 mm. This slow shearing part of the test was terminated after 140.6 mm of shearing.

The rate of displacement was suddenly increased to 1.36 cm/sec and the measured strength reached immediately its threshold value of 44.7°. Shearing at the same rate of displacement continued while the sample appeared to dilate and then to contract leading to an increase in strength to 45.6° after 23.2 cm. Further shearing led to a decrease in strength which, after the sample had been sheared for 29.4 cm, stabilised within a range between 38.8° and 42.1°. The range of the variation of the measured strength may have been influenced by internal ring friction which could not be accounted for. The rate of displacement was changed from 1.36 cm/sec, to 0.16 cm/sec, to 2.03 cm/sec, to 8.08 cm/sec, to 9.14 cm/sec and to 3.69 cm/sec before the test was terminated, but no effect on the measured strength was observed.

Test VI.8

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 41.5°

$\phi_{threshold}$= 41.9°

(Figures VI.8.1-2)

The sample reached its residual state after 47.5 mm of shearing while its measured strength stabilised at 41.5°. This part of the test was terminated after the sample had been sheared for 87.4 mm.

The rate of displacement was suddenly increased to 1.15 cm/sec and the measured strength reached its threshold value of 41.9° at zero associated displacement. Further shearing at the same rate of displacement led to an increase in measured strength which reached the value of 45.3° after 70 cm of displacement.

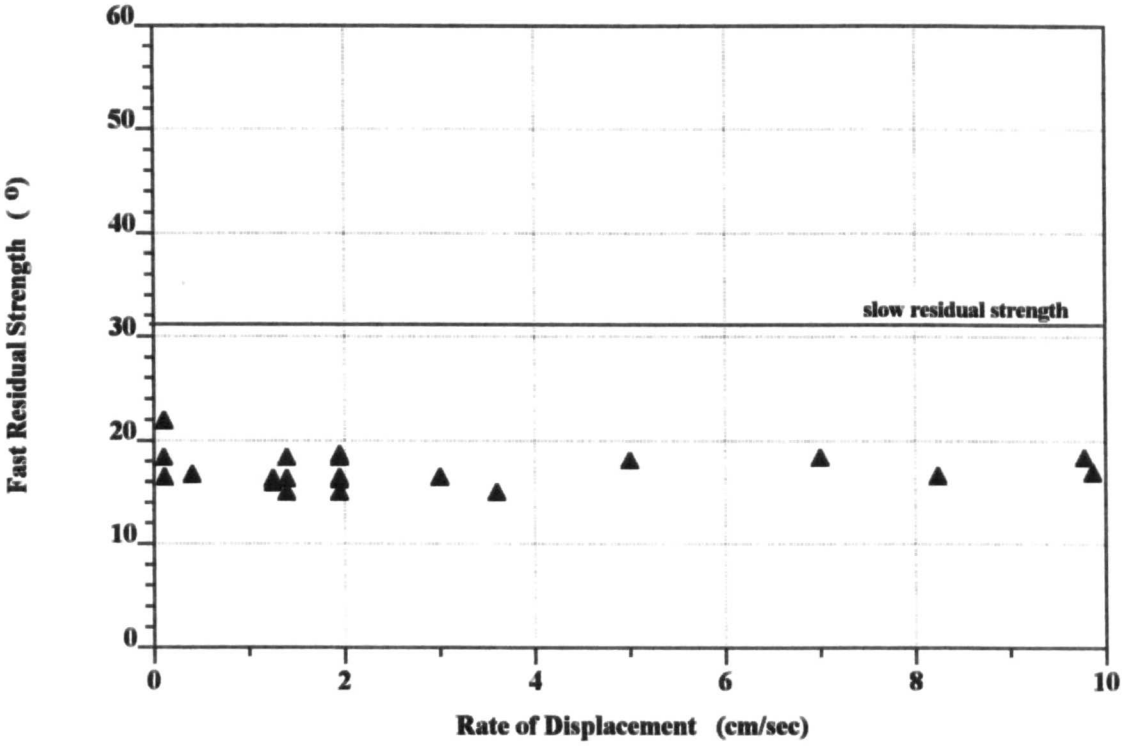


Fig. 6.3.43 Rate dependent characteristics of normally consolidated, submerged, Cowden till (3-stage test, 100kPa)

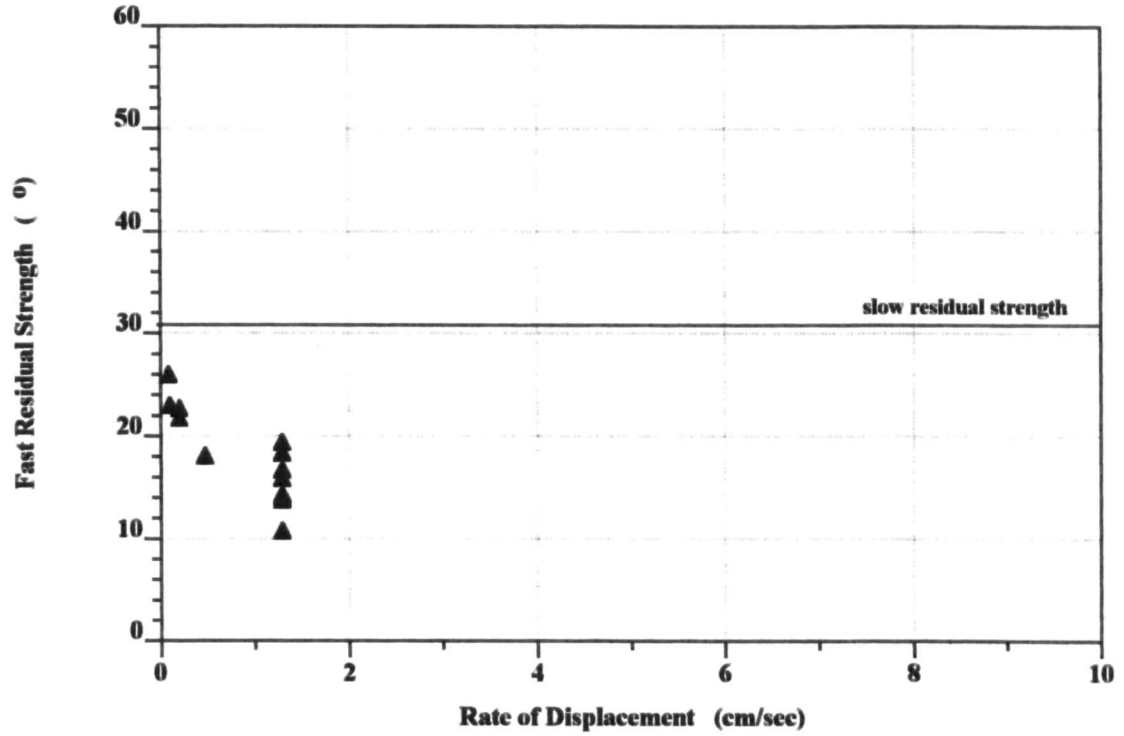


Fig. 6.3.44 Rate dependent characteristics of normally consolidated, submerged, Cowden till (3-stage test, 200 kPa)

displacement was changed gradually and its strength responded immediately to any sudden changes in the rate of displacement.

TEST IV.5

σ_n= 50 kPa

$\phi_{residual}$= 12.3°

ϕ_{peak}= 28.6°

(Figures IV.5.1-2)

Another test on a different identical sample of the same soil was conducted in order to investigate, using the thin section technique, the differences between the shear zones of the sample tested under 50 kPa normal stress and the sample tested under 400 kPa normal stress. The soil was sheared at 0.09 mm/min and after a displacement of 0.78 mm it reached its slow peak strength of 24.2°. Continued shearing led to a decrease of the shear strength of the sample while its height decreased due to particle orientation and some soil loss. After 5.18 mm of shearing the sample height had reached its lowest point 0.143 mm below its level at the end of the consolidation stage while the measured strength continued to decrease. After a shear displacement of 19.3 mm the strength reached its slow residual value of 12.3°, while the sample height curve exhibited an upward movement. Further shearing led to continuing increase of the measured strength accompanied with an upward movement of the sample height curve. After 90.3 mm of shear displacement the measured strength had increased to 27.5° and the sample height curve had increased 0.029 mm above its level at the point where the sample had reached its residual state. One could argue that the increase in measured strength might have been due to friction caused by the contamination of the rings, however, the sample height curve suggests the formation of a non-planar (possibly undulating) shear surface which might have caused that increase.

Subsequently, the rate of displacement was increased to 0.055 cm/min for the next 3.19 cm of displacement, and the strength increased and remained constant at 16.7°. The rate of displacement was increased further to 1.33 cm/sec in the next 15.7 cm of shearing and the strength of the sample also increased to its fast peak value of 28.6° without reaching a threshold value but rather exhibiting a positive rate response. Similar behaviour was also observed in the previous sample when it was tested under the same conditions. Continued shearing led to the formation of an

residual value of 49.5° after 155.8 mm of shearing. This part of the test was terminated after 161.5 cm of shearing.

The rate of displacement was increased suddenly to 1.29 cm/sec and the measured strength reached immediately its threshold value of 53.8°. Further shearing led to a decrease in strength which had dropped to 49.4° after a displacement of 45 cm. While shearing continued at the same rate of displacement, the strength of the sample stabilised at 45.0° except for the periodic spike-like increases measured by the load cells which should be attributed mainly to friction developed within the rings. The consistency of the magnitude, shape, and periodic re-appearance of these increases in the measured strength, while no evidence of an undulating shear surface were observed from the sample height curve, can only mean that they were due to internal ring friction. The rate of displacement was varied from 1.29 cm/sec, to 0.97 cm/sec, to 0.15 cm/sec and then back to 1.29 cm/sec but no effect on the measured strength was observed. The same was the case for the second stage of this fast shearing part of the test where the rate of displacement was varied between 1.76 cm/sec, 8.49 cm/sec and 10.0 cm/sec, gradually and suddenly, until the test was terminated after 660 cm of cumulative shearing.

Test V.9

σ_n= 100 kPa

OCR= 4

$\phi_{residual}$= 45.8°

$\phi_{threshold}$= 47.2°

(Figures V.9.1-2)

The sample was sheared at 0.09 mm/min and the measured strength reached its slow peak value of 50.9° after 1.9 mm while the sample height increased by 0.05. Further shearing led to an additional increase in sample height and the measured strength reached its slow residual value of 45.8° after 95 mm. This part of the test was terminated after 108.3 mm of shearing.

The rate of displacement was increased suddenly to 0.61 cm/sec and the measured strength reached immediately its threshold value of 47.2° before it dropped to 42.3°. While the rate of displacement continued to be increased until it reached 1.43 cm/sec, the measured strength continued to decrease until it stabilised at a range between 38.7° and 40.4°. The rate of displacement was varied from 1.43 cm/sec to

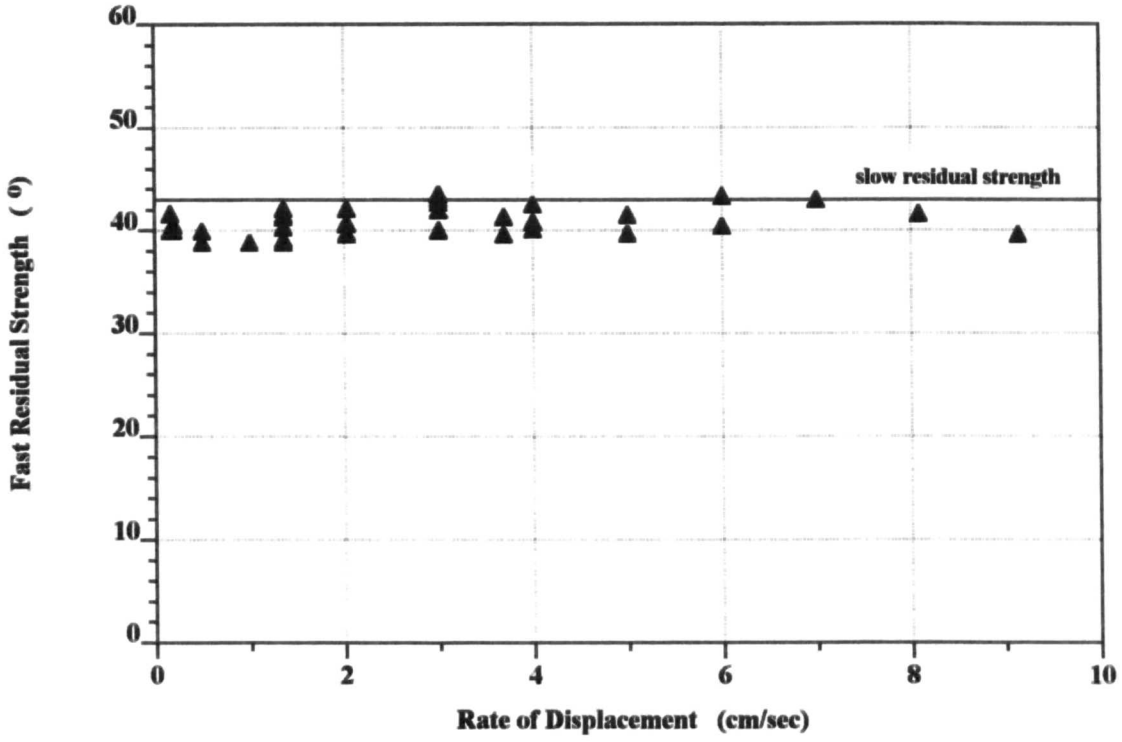


Fig. 6.3.53 Rate dependent characteristics of over-consolidated, completely dry, brown London clay (100 kPa)

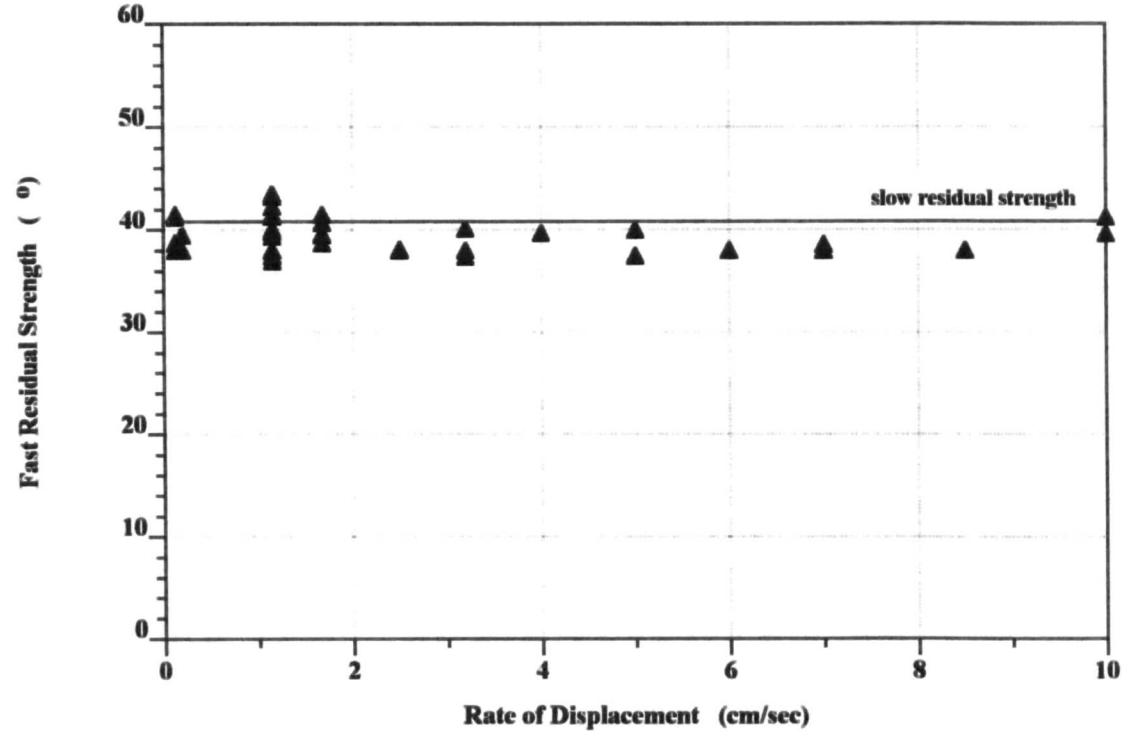


Fig. 6.3.54 Rate dependent characteristics of over-consolidated, completely dry, brown London clay (200 kPa)

before the test was terminated. On the second cycle one could observe that, while the rate of displacement was being decreased gradually from 1.70 cm/sec to 0.11 cm/sec, the measured strength of the sample remained constant at 20.6° for rates ranging from 1.70 cm/sec to 0.70 cm/sec. On the fourth cycle one could again observe that, while the rate of displacement was being increased gradually from 0.11 cm/sec to 1.70 cm/sec, the measured strength remained constant for rates higher than 0.70 cm/sec. Positive rate characteristics were observed in this test, specially upon decreasing the rate of displacement. The sample height and the stress ratio curves indicated the existence of a planar (or non-undulating) shear surface.

Overconsolidated, Submerged

This sample was put together and tested in order to compare the response of the overconsolidated structure during fast shearing. The soil was remoulded to a water content around its plastic limit, consolidated under 400 kPa normal stress and then unloaded to 100 kPa.

TEST IV.10

σ_n= 100 kPa

OCR= 4

$\phi_{residual}$= 15.5°

$\phi_{threshold}$= 36.9°

(Figures IV.10.1-2)

The sample was sheared at 0.09 mm/min and reached its slow peak strength of 24.2° after 1.3 mm of shearing. At that point the sample height had increased by 0.011 mm, apparently due to the overconsolidated structure's relaxation upon shearing. Further shearing at the same rate of displacement led to an additional increase in sample height while the strength of the sample decreased towards its slow residual value. After 49.7 mm of shearing and 0.218 mm of increase in sample height the sample had reached its slow residual strength of 15.5°.

The rate of displacement was increased suddenly to 1.76 cm/sec and the strength reached its threshold value of 16.9°. Further shearing at the same rate of displacement led to a dramatic decrease in measured strength below its slow residual value, after a shear displacement of 19.6 cm, while the sample height increased to 0.25 mm above its level at the end of the slow shearing part of this test. Continued

The sample appeared to be dilating as shearing at 0.09 mm/min commenced. After 1.9 mm of shearing the sample height had increased by 0.088 mm and the measured strength had reached its slow peak value of 52.2°. Further shearing led to a decrease in the measured strength while the sample height increased by an additional 0.019 mm. The sample reached its slow residual strength of 46.3°, and this part of the test was terminated after 97.9 mm of shearing.

The rate of displacement was increased suddenly to 1.72 cm/sec and the measured sample strength reached its threshold value of 50.2° at zero associated displacement. Further shearing at the same rate of displacement led to a decrease in strength while the sample height increased by 0.102 mm. After 9.4 cm of shearing the measured strength had dropped to 46.3° and the sample height had began to decrease due to soil being loss. While shearing continued at 1.72 cm/sec the strength of the sample decreased and after 69.0 cm of cumulative displacement it stabilised within a range between 42.0° and 45.0°. Any further change of the rate of displacement, sudden or gradual, from 0.107 cm/sec to 1.72 cm/sec had no effect on the strength of the sample. The test was terminated after 480 cm of shearing.

Normally Consolidated, Submerged

Test V.16

σ_n= 50 kPa

$\phi_{residual}$= 47.7°

$\phi_{threshold}$= 54.8°

(Figures V.16.1-2)

The behaviour of this sample upon shearing at the drained rate of displacement was similar to the behaviour of the non-submerged sample under the same normal stress. The sample contracted initially and, after 0.9 mm of shearing, its height had decreased by 0.047 mm. Further shearing led to an increase in sample height by 0.42 mm while measured strength reached its slow peak value of 52.0°. The strength reached its slow residual value of 47.7° after 84.5 mm of shearing while the sample height increased by a further 0.025 mm. This part of the test was terminated after 129.2 mm of shearing.

The rate of displacement was increased suddenly to 1.69 cm/sec and the measured strength reached immediately its threshold value of 54.8°. However, as

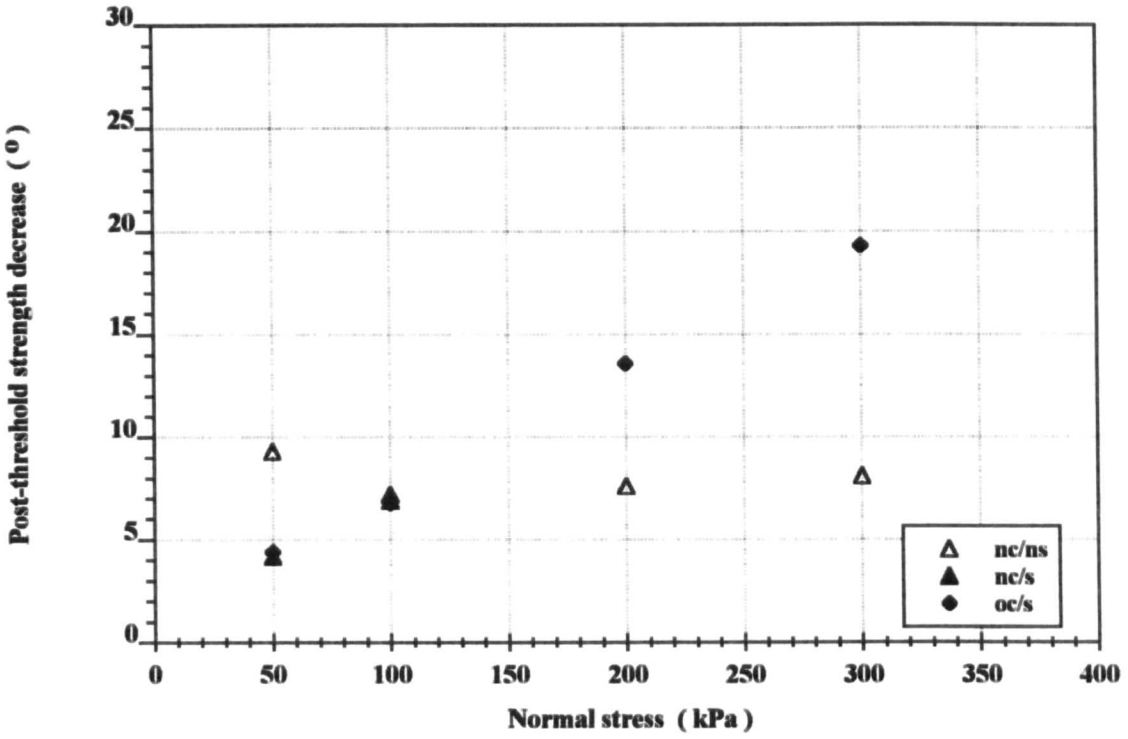


Fig. 6.4.5 Normal stress effect on post-threshold strength decrease of pure silica samples

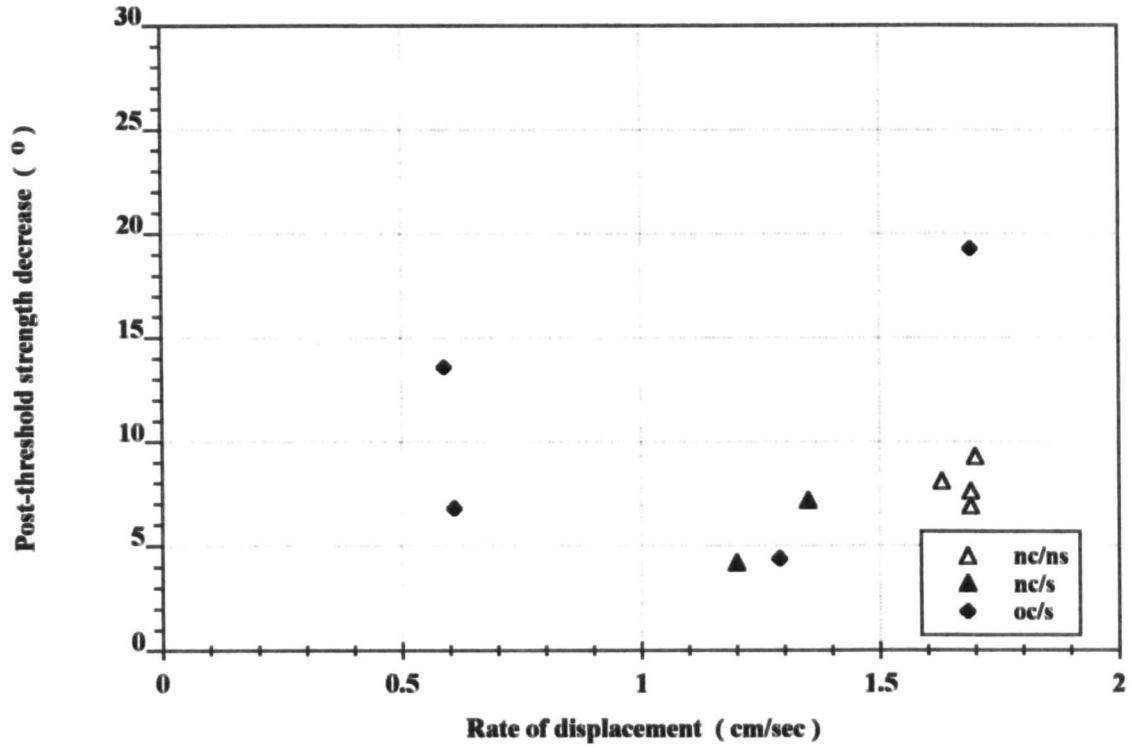


Fig. 6.4.6 Rate of displacement effect on post-threshold strength decrease of pure silica samples

was decreased gradually to 0.125 cm/sec during the final 52.3 cm of shearing of this test. The measured strength of the sample did not change when the rate of displacement was decreased from 1.70 cm/sec to 0.729 cm/sec.

Test IV.14

σ_n= 100 kPa

$\phi_{residual}$= 31.4°

$\phi_{threshold}$= 37.7°

(Figures IV.14.1-2)

The sample was sheared at 0.09 mm/min and reached its slow peak strength of 32.1° after 0.98 mm of shear displacement, at which point its height had decreased by 0.01 mm. The measured strength decreased with further shearing until it reached its slow residual value of 31.4° after 46.2 mm of shear displacement, at which point the sample height had decreased by an additional 0.091 mm. It appeared from the shape of the stress ratio/displacement curve that the majority of particles within the shear zone of the sample had already been oriented during the shearing stages under the lower normal stress of 50 kPa. The slow shearing part of this test was terminated after 112.1 mm of shear displacement.

The rate of displacement was increased to 0.09 cm/sec for the first 10.7 cm of shearing at this fast shearing part of the test and the measured sample strength increased to 37.7° at zero associated displacement were it remained until the rate of displacement was further increased. After the first 10.7 cm of shearing at 0.09 cm/sec, the rate of displacement was increased to 1.68 cm/sec were it was remained for the following 111.6 cm of shearing. The measured strength also increased with the rate of displacement and varied between 41.3° and 44.1°. The sample height curve appeared to be undulating with an amplitude of 0.11 mm during the first two rotations of the sample. However, after a cumulative displacement of 82.5 cm the undulating shape of the shear zone flattened-out. After 122.3 cm of shearing, the rate of displacement was decreased to 0.09 cm/sec and the measured strength dropped to 40.6°. In the following 49.4 cm of shearing the rate of displacement was increased and decreased gradually from 0.09 cm/sec to 1.64 cm/sec and back in order to investigate the viscous characteristics of the sample. While the rate was being increased gradually, the measured strength of the sample jumped to 41.6°. This was not the case upon decreasing the rate of displacement where the measured sample

displacement. As it was expected, the sample exhibited a substantial amount of dilation upon shearing and its height had increased by 1.43 mm after 5.7 cm of displacement while the measured strength decreased to 36.2°. The rate of displacement was increased gradually to 1.47 cm/sec and then decreased to 0.21 cm/sec but no effect on the strength of the sample was observed. The test was terminated after 169.5 cm of shearing at which point the sample height had decreased by 0.17 mm because of particle crushing since no sample loss was experienced.

Test V.22

σ_n= 100 kPa

$\phi_{threshold}$= 25.2°

$\phi_{residual}$= 25.2°

(Figure V.22.1)

The sample was sheared at 0.072 cm/sec for the first 30.8 cm of this test. The measured strength reached immediately a threshold value of 25.2°. However, further shearing did not lead to a decrease in measured strength even though the sample appeared to be dilating as its height increased by 0.1 mm. The measured strength was not affected by any change in the rate of displacement which was increased gradually to 1.54 cm/sec after 75.5 cm of shearing and was decreased, in the same manner, to 0.225 cm/sec after 155.3 cm of shearing. At the end of the test the sample height had decreased by 0.06 mm because of further particle crushing.

Test V.23

σ_n= 200 kPa

$\phi_{threshold}$= 26.1°

$\phi_{residual}$= 26.1°

(Figure V.23.1)

The sample was sheared at 0.07 cm/sec for the first 30.4 cm of this test and its measured strength reached a threshold value of 26.1° at zero associated displacement while its height increased by 0.12 mm. Continued shearing was associated with a decrease in sample height due to particle crushing which, at the end of the test, after the sample had been sheared for 161.2 cm, dropped to 0.24 mm below its initial level. The rate of displacement was gradually increased to 1.58

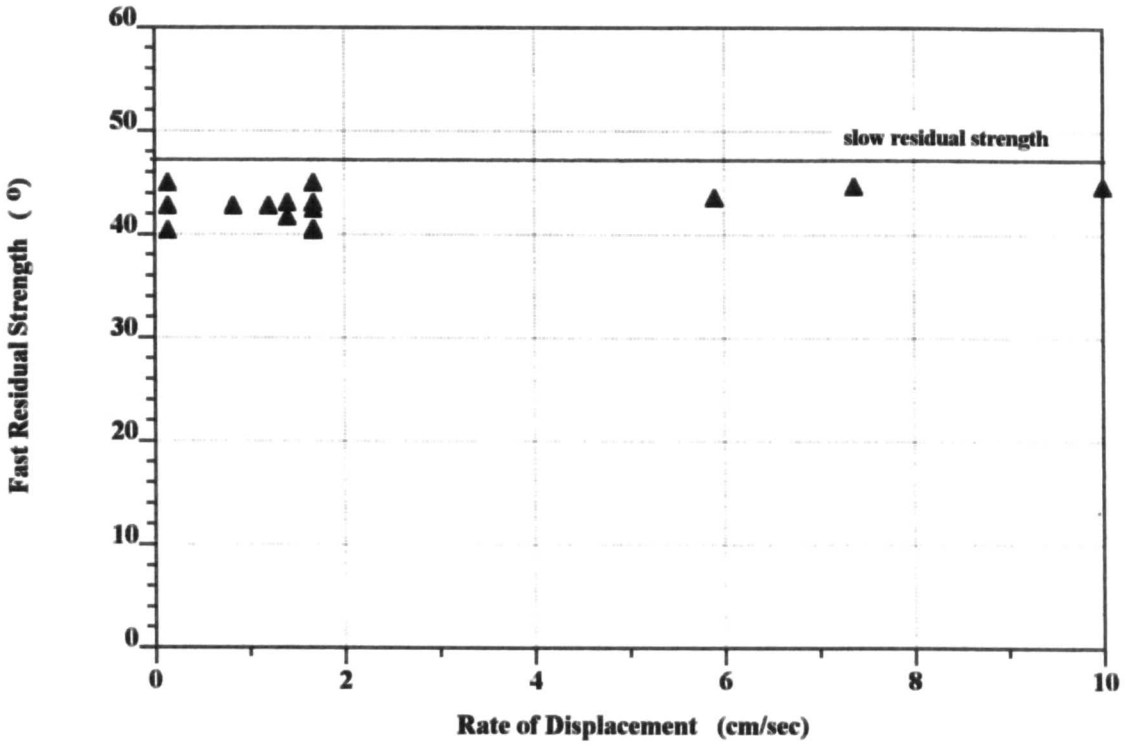


Fig. 6.4.15 Rate dependent characteristics of normally consolidated, submerged pure silica sample (50 kPa)

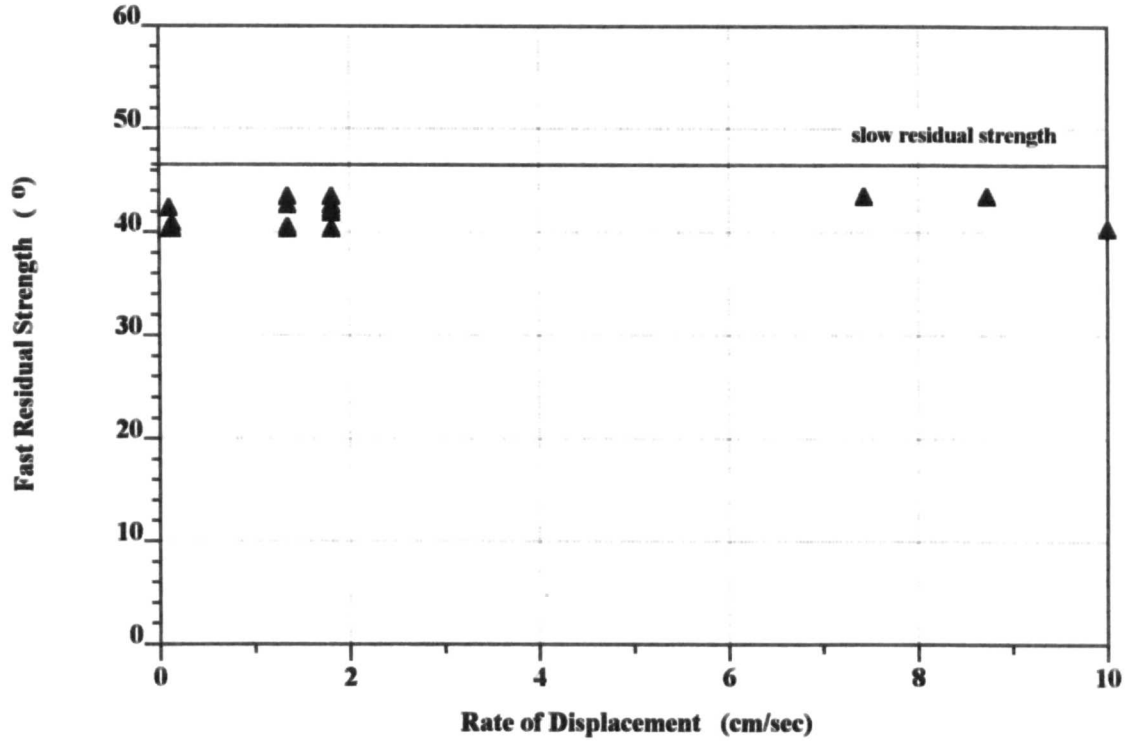


Fig. 6.4.16 Rate dependent characteristics of normally consolidated, submerged pure silica sample (100 kPa)

shearing part. It appeared from the shape of the sample height curve that an undulating shear zone had been formed with an initial amplitude of 1.88 mm which, with further shearing, decreased to 0.65 mm. The rate of displacement was gradually decreased to 0.28 cm/sec after 156.3 cm of shearing but this did not seam to affect the strength of the sample which was fluctuating between 9.02° and 16.8°. After 198.2 cm of shearing, the rate of displacement was suddenly increased to 1.87 cm/sec and the strength increased immediately to 14.8° before it dropped to 7.2°. The rate of displacement was increased and decreased gradually or instantly to levels which ranged between 1.9 cm/sec and 9.78 cm/sec until the test was terminated. During the second stage of this fast shearing part of the test the strength of the sample fluctuated between 4.04° and 7.2° while the amplitude of the undulating shear surface continued to decrease. Any change of the rate of displacement during the second stage of this testing procedure had no significant effect on the fast residual strength of the sample.

Test IV.19

σ_n= 200 kPa

$\phi_{residual}$= 31.4°

$\phi_{threshold}$= 40.4°

(Figures IV.19.1-2)

The sample was sheared at 0.09 mm/min and reached its residual state after 89.3 mm at which point the sample height had decreased by 0.3 mm. This decrease in sample height should be attributed to both soil loss and particle reorientation. However, if one assumes that the rate at which the soil was being lost through the gap of the ring was constant, one could argue that the major cause of this 0.3 mm of decrease in sample height is particle reorientation, since only an additional 0.02 mm decrease in sample height was recorded after the particle reorientation had ceased and before the slow shearing part of this test was terminated at 142.5 mm.

The rate of displacement was suddenly increased to 1.25 cm/sec and the measured strength reached its threshold value of 40.4° at zero associated displacement. Further shearing led to a continuous decrease in measured strength while the sample appeared to be dilating. After 20 cm of shearing at 1.25 cm/sec, the sample height had increased by 0.175 mm above its level at the end of the slow shearing part of this test and the measured sample strength had already dropped to its slow residual value of 31.4°. Furthermore, after 40 cm of shearing at the same rate of

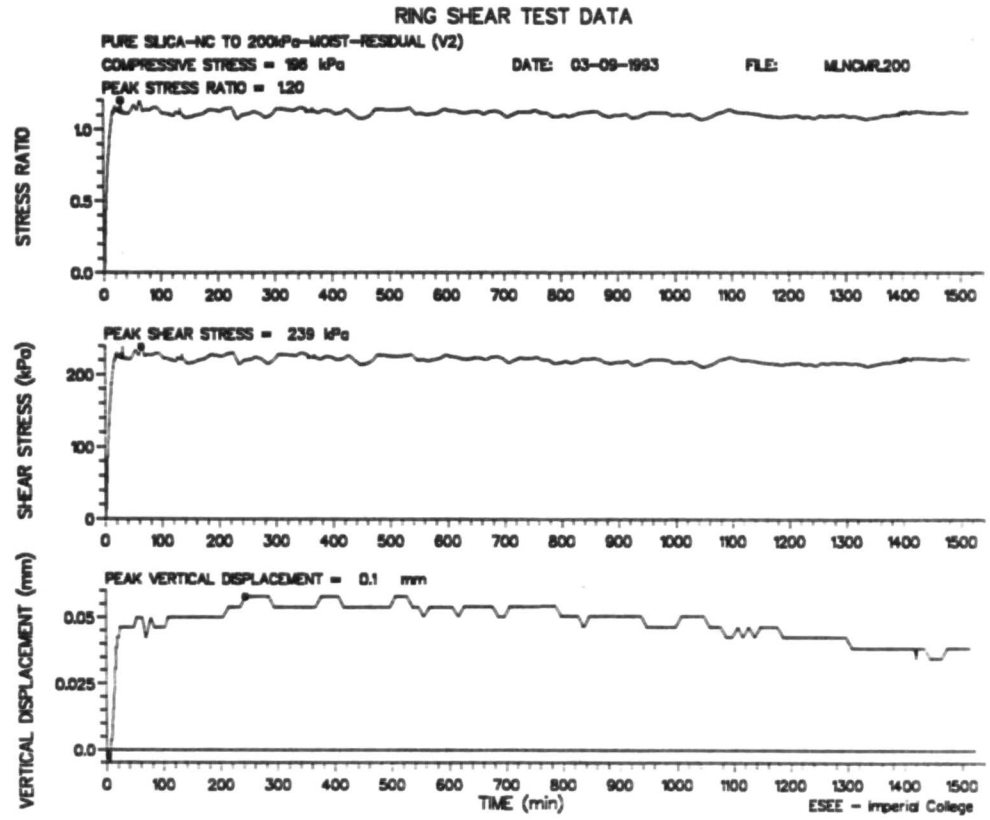


Fig. V.3.1 Slow shearing test on normally consolidated, non-submerged pure silica (200 kPa)

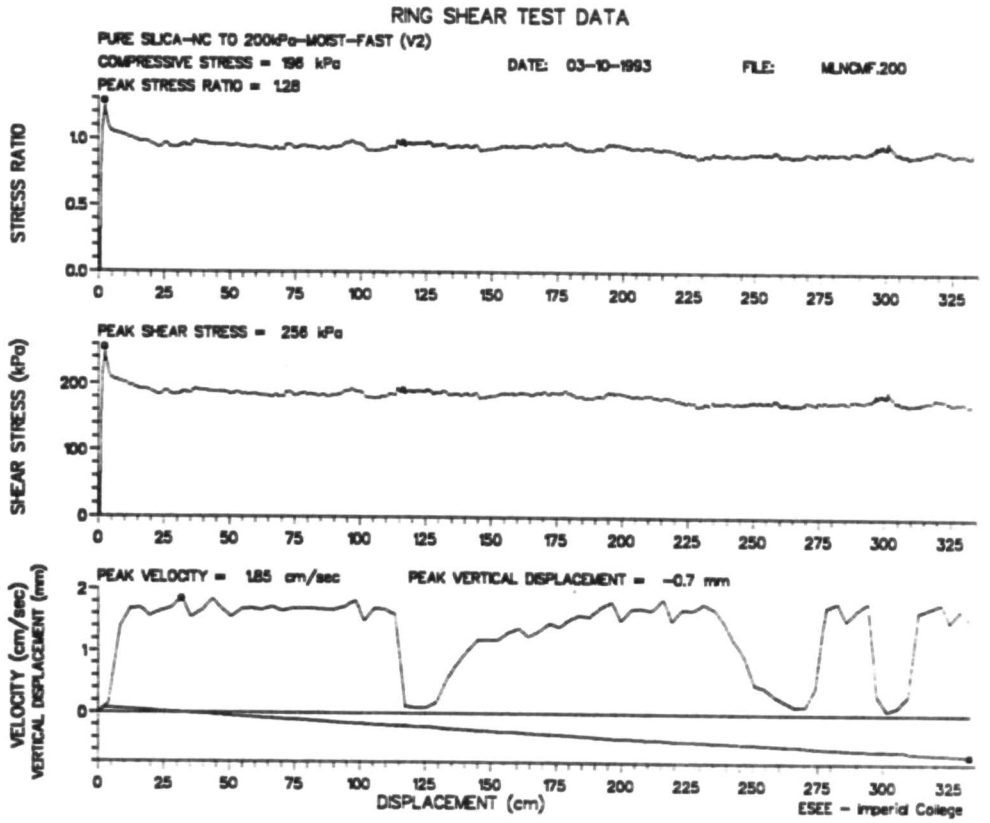


Fig. V.3.2 Fast shearing test on normally consolidated, non-submerged pure silica (200 kPa)

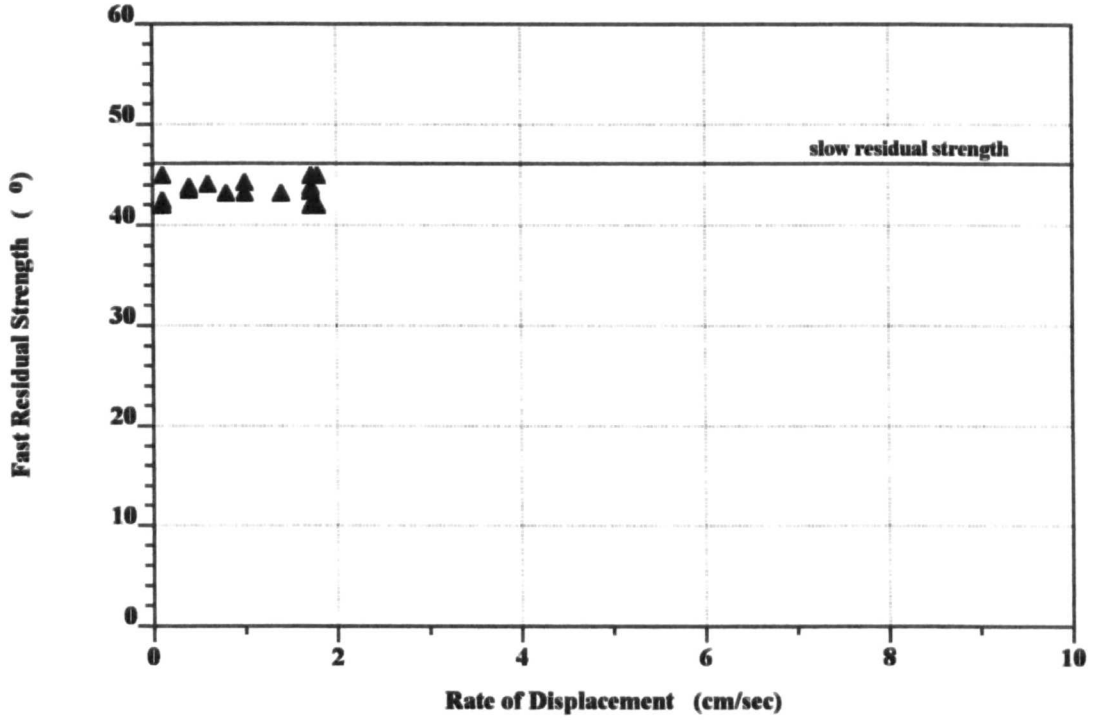


Fig. 6.4.25 Rate dependent characteristics of normally consolidated, non-submerged, sandy-silt sample (250 kPa)

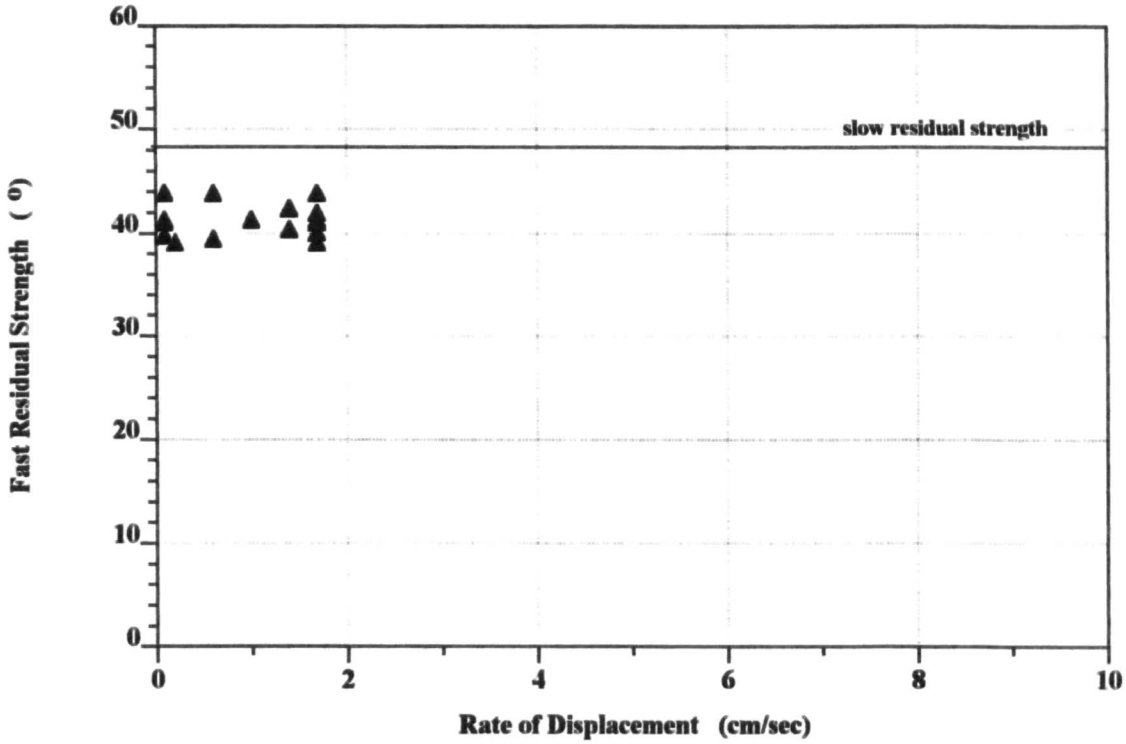


Fig. 6.4.26 Rate dependent characteristics of normally consolidated, submerged, sandy-silt sample (50 kPa)

of the sample had decreased to 10.5° after 62 cm of shearing and fluctuated between 7.3° and 10.5° before the rate of displacement was suddenly decreased to 0.06 cm/sec. It is important to note at this point that the measured sample strength appeared to be increasing from 7.3° to 11.3° while the sample was being sheared at 0.06 cm/sec as it was the case for all the slow stages of this fast shearing part of the test. Between 220.4 cm and 232.1 cm of cumulative shear displacement the sample was sheared at 0.08 cm/sec and its fast residual strength again increased from 9.5° to 12.7°, and between 256.3 cm and 271.4 cm it increased from 12.7° to 15.4°. It was suggested at that point that these increases in the measured fast residual strength of the sample may be attributed to pore pressure dissipation. The rate of displacement was increased and decreased gradually from 0.06 cm/sec to 1.71 cm/sec, but these changes clear effect on the measured strength. The test was terminated after 300 cm of shearing at which point the sample height had decreased by 1.8 mm. This decrease in sample height should be attributed to soil loss.

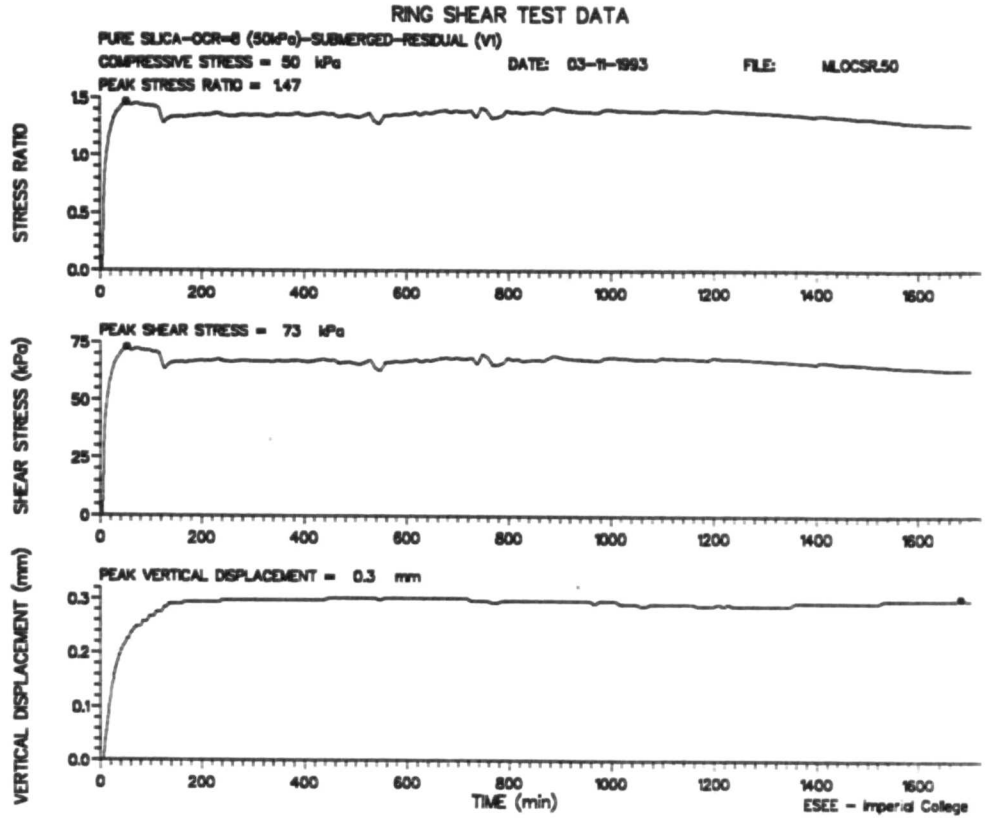


Fig. V.8.1 Slow shearing test on over-consolidated, submerged pure silica (50 kPa - OCR= 8)

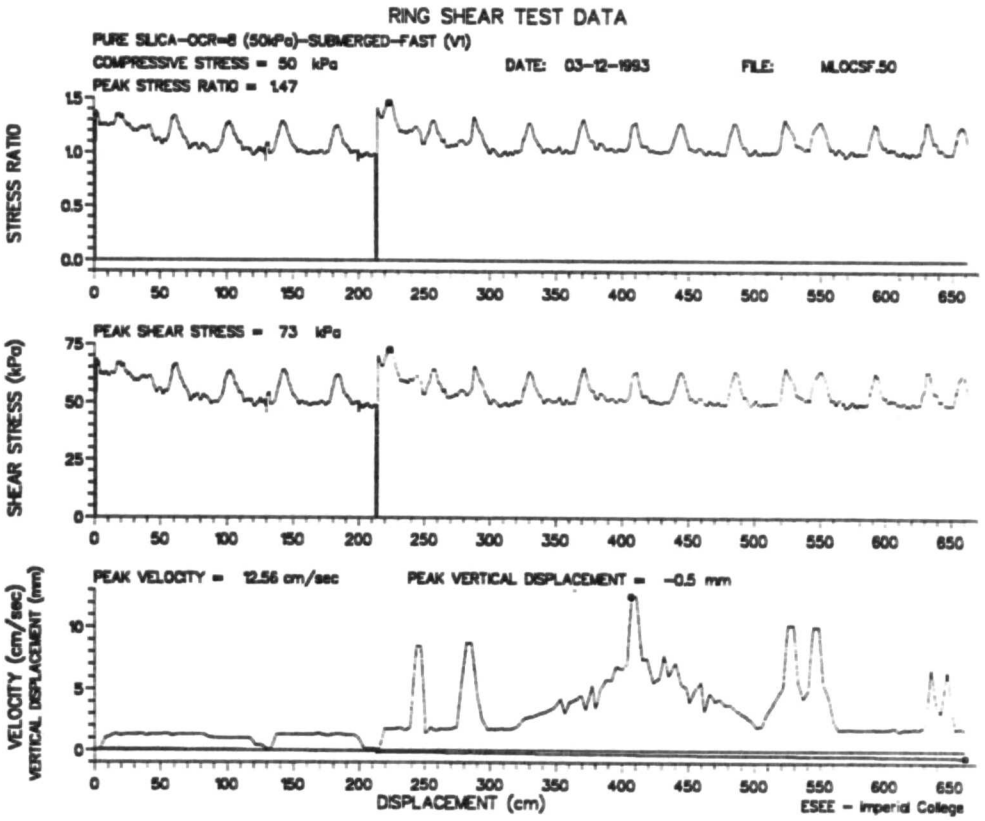


Fig. V.8.2 Fast shearing test on over-consolidated, submerged pure silica (50 kPa - OCR= 8)

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 The Apparatus

The ring shear apparatus is the only device which can produce large unidirectional shear displacements and estimate the residual strength of soils by uninterrupted shearing. Because of its ability to produce large shear displacements at varying rates of displacement, this apparatus has been widely used to investigate the behaviour of soils under fast shearing. However, the loss of soil through the gap of the confining rings of the apparatus limits the duration for which high rates of displacement can be sustained, and thus inevitably affects the experimental results. To minimise the loss of soil, and thus prolong the duration of fast shear testing, a new set of confining rings was designed and manufactured.

The new confining rings were evaluated for their efficiency in minimising the soil loss and their compatibility with the original rings in measuring the slow residual strength of soils. It is has been shown that the residual strength measured with the new design varies by 0 to 2 % from the residual strength measured using the original apparatus. Moreover, the new design results in as little as seven times less soil loss than the original apparatus. The reproducibility of the test results and the repeatable behaviour of the samples tested indicate the effectiveness of the new design. It is concluded that the new confining rings improve substantially the capabilities of the IC/NGI ring shear apparatus, particularly to investigate the behaviour of soils under fast rates of displacement.

In analysing the variation of the residual strength at increasing rates of displacement, the *fast residual strength* (*i.e.* the steady residual strength measured at high rates of displacement) is compared with the *slow residual strength* (*i.e.* the residual strength measured at 0.09 mm/min). It has been suggested (Lemos, 1986; Tika, 1989)

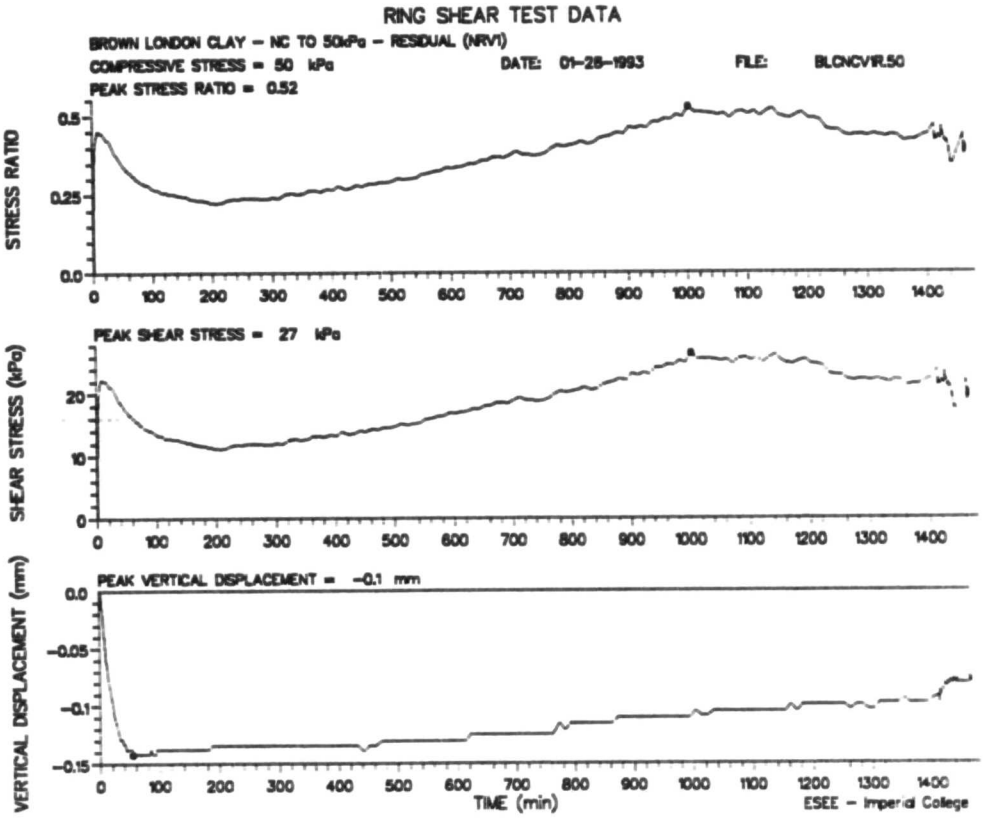


Fig. IV.5.1 Slow shearing test on normally consolidated, non-submerged brown London clay (50 kPa - second sample)

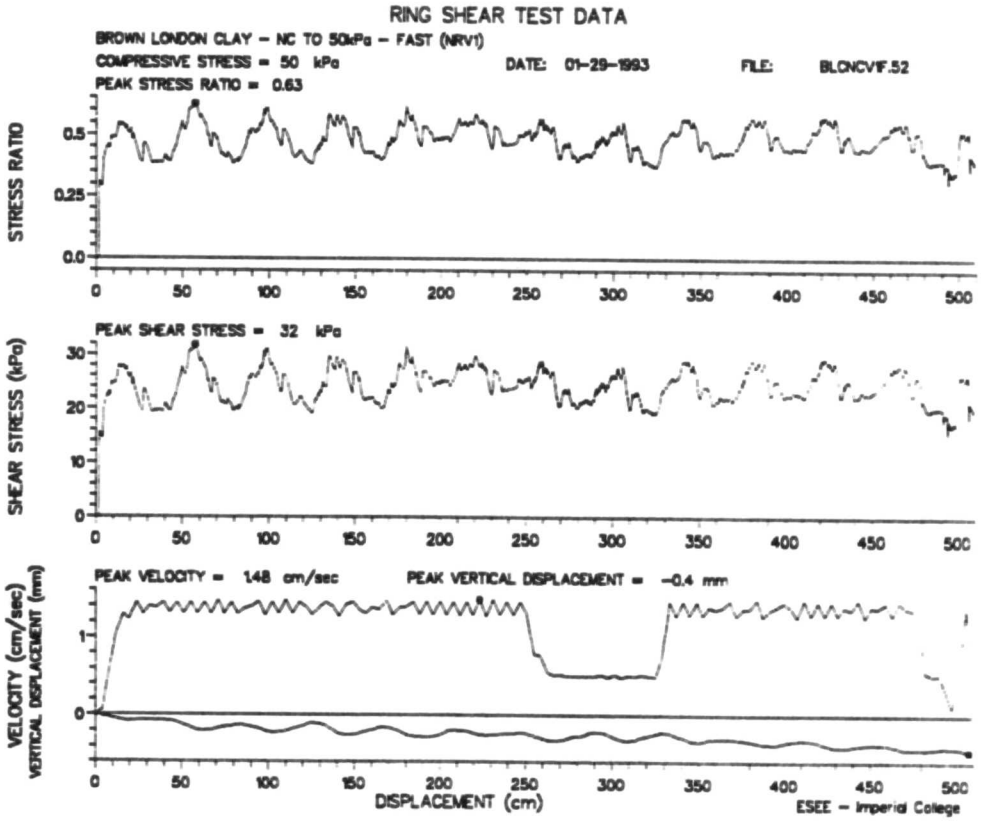


Fig. IV.5.2 Fast shearing test on normally consolidated, non-submerged brown London clay (50 kPa - second sample)

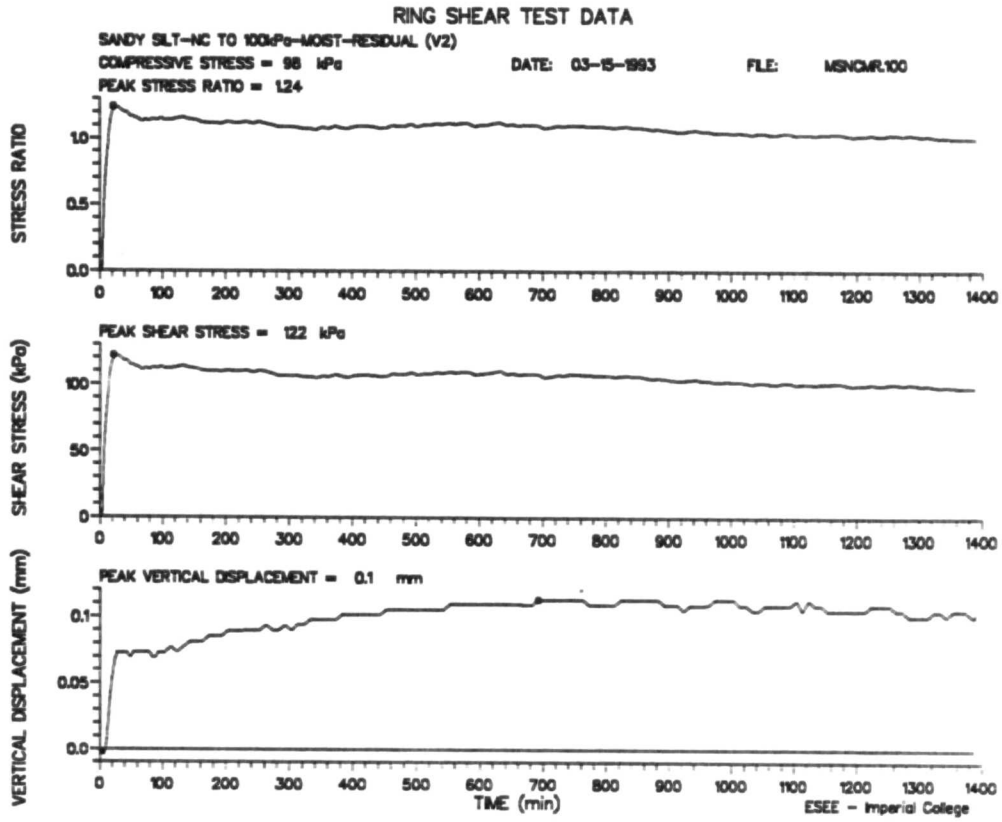


Fig. V.13.1 Slow shearing test on normally consolidated, non-submerged artificial sandy-silt (100 kPa)

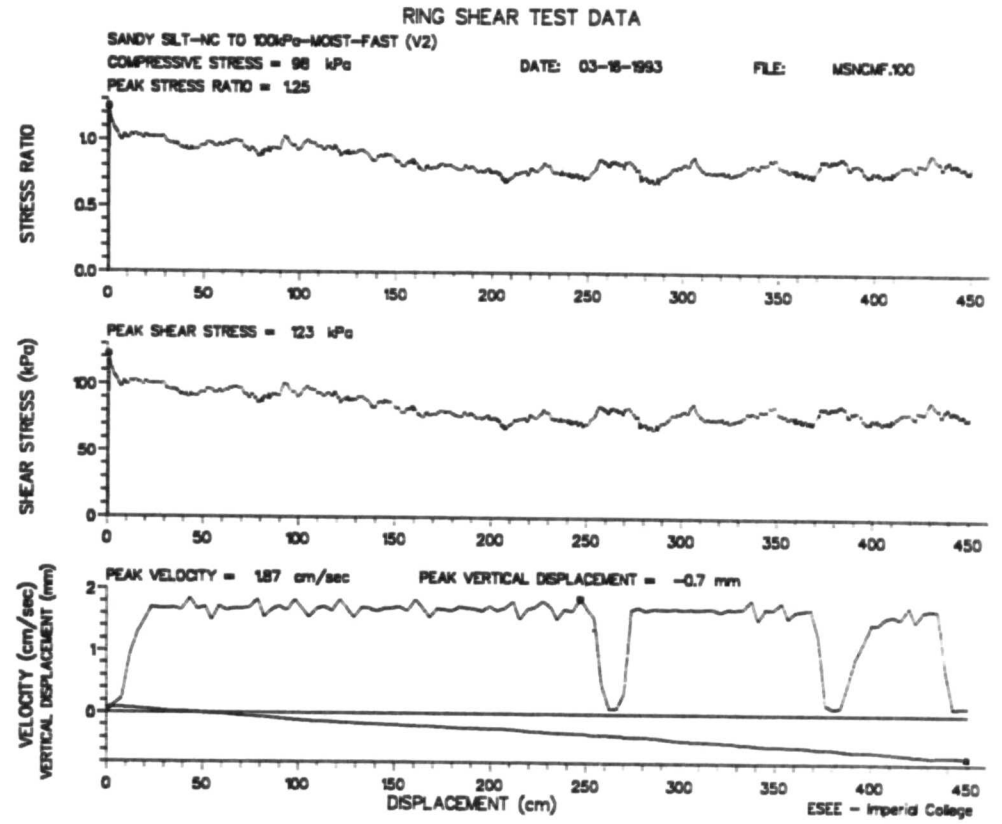


Fig. V.13.2 Fast shearing test on normally consolidated, non-submerged artificial sandy-silt (100 kPa)

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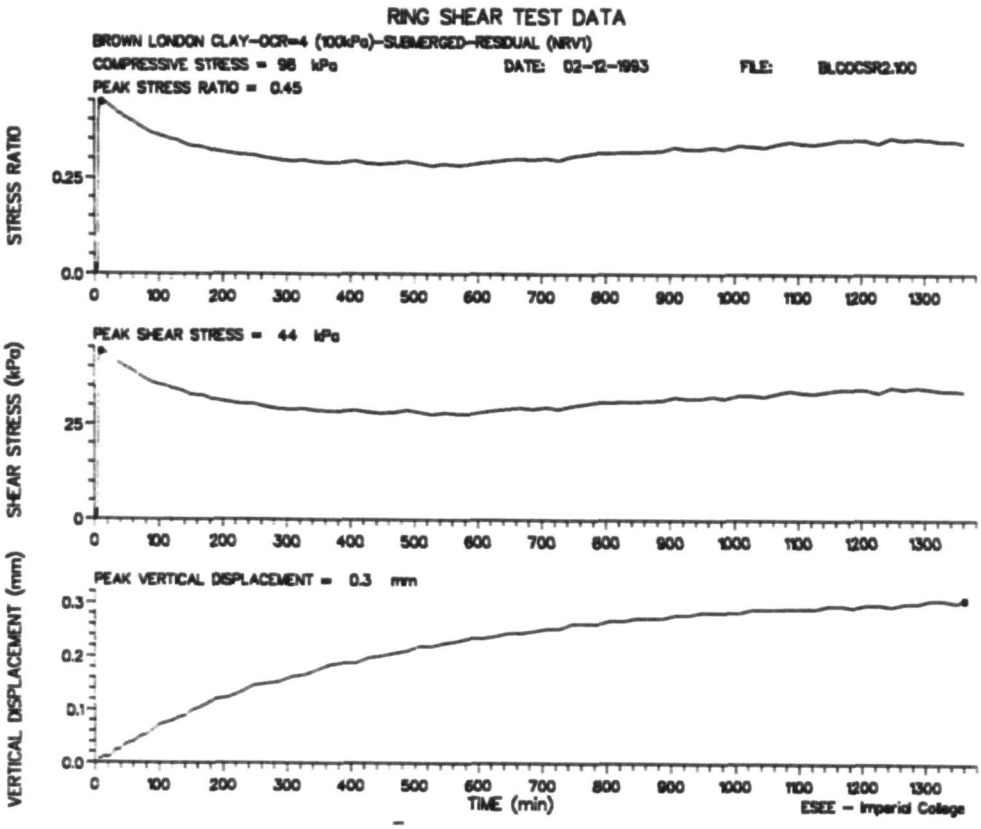


Fig. IV.10.1 Slow shearing test on over-consolidated, submerged brown London clay (100 kPa - OCR= 4)

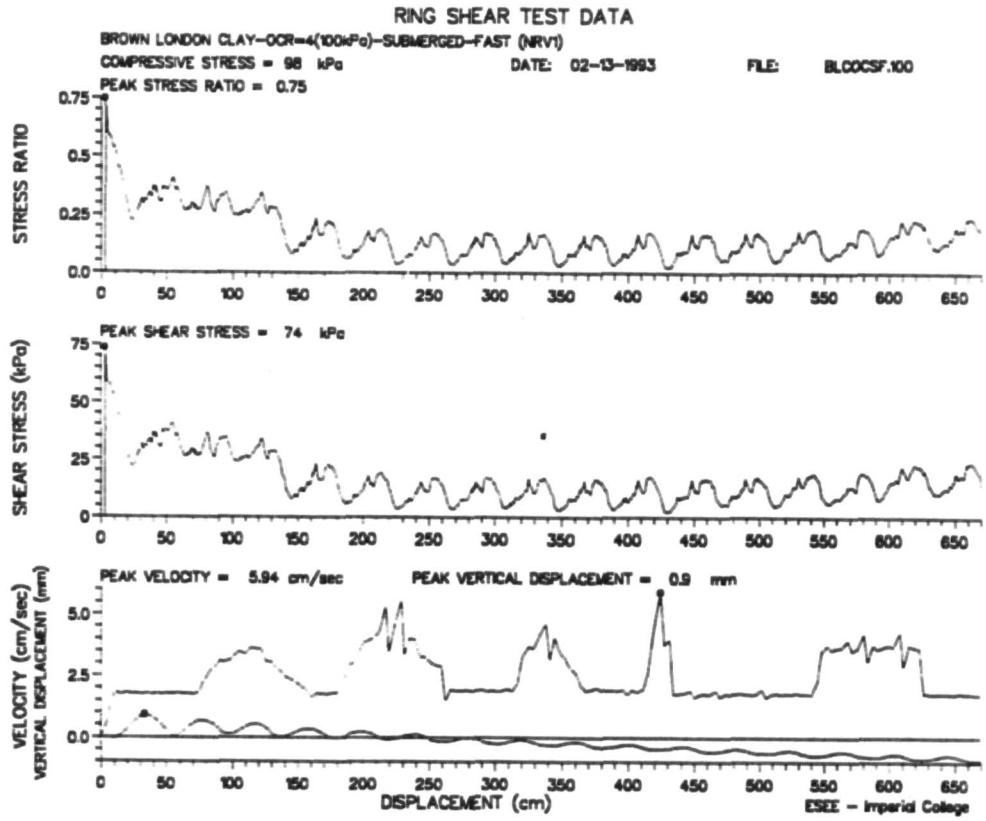


Fig. IV.10.2 Fast shearing test on over-consolidated, submerged brown London clay (100 kPa - OCR=4)

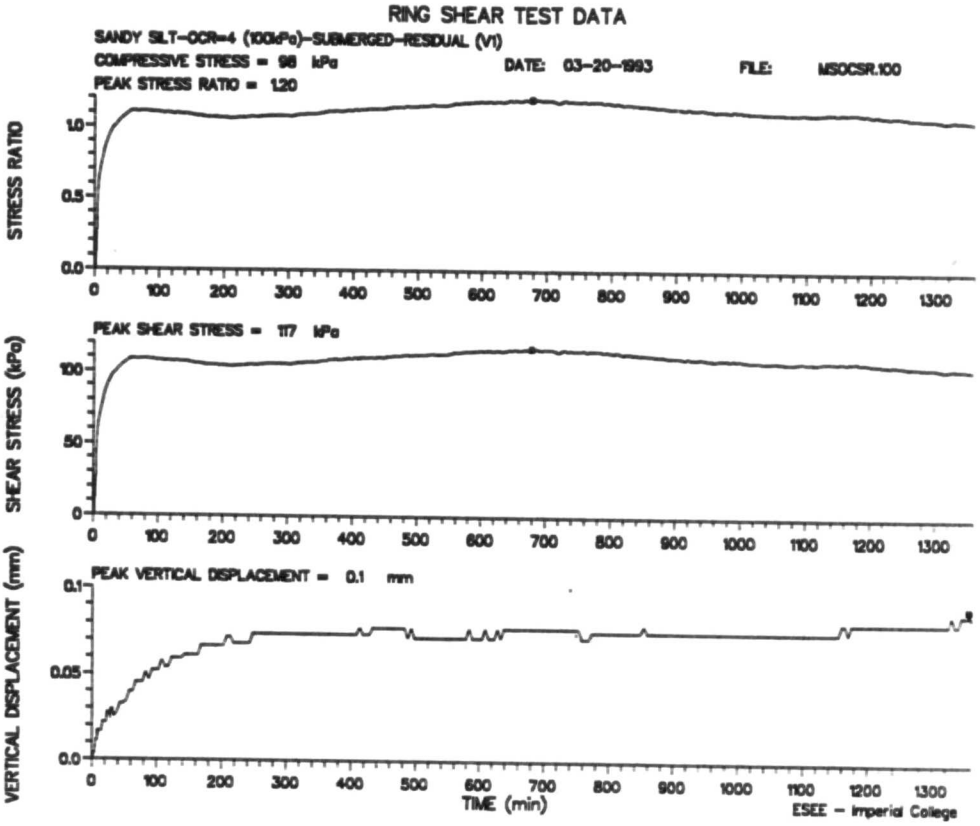


Fig. V.18.1 Slow shearing test on over-consolidated, submerged artificial sandy-silt (100 kPa - OCR= 4)

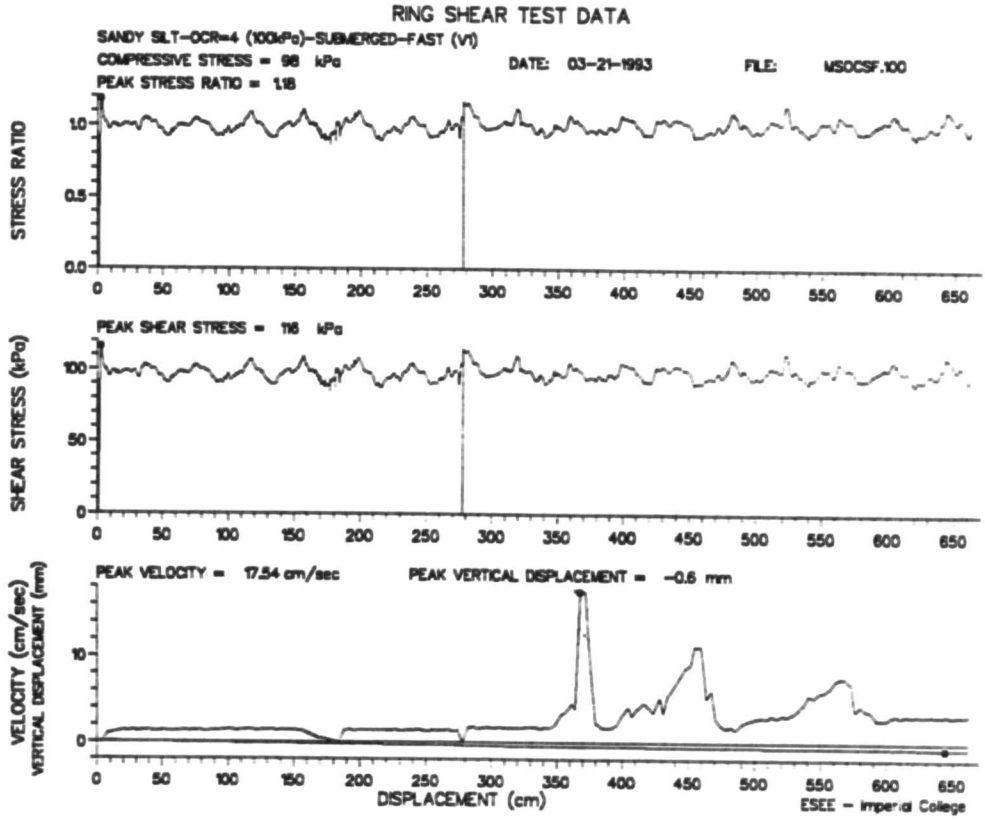


Fig. V.18.2 Fast shearing test on over-consolidated, submerged artificial sandy-silt (100 kPa - OCR= 4)

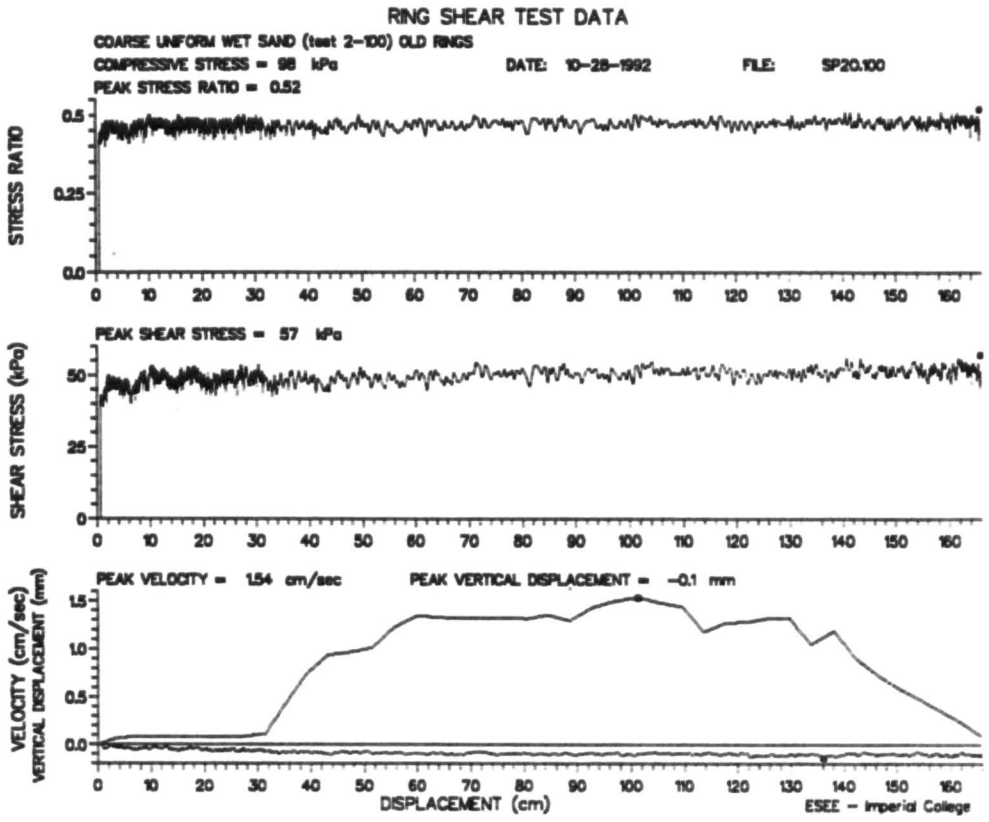


Fig. V.25.1 Fast shearing test on normally consolidated, wet, non-submerged medium-coarse sand (100 kPa)

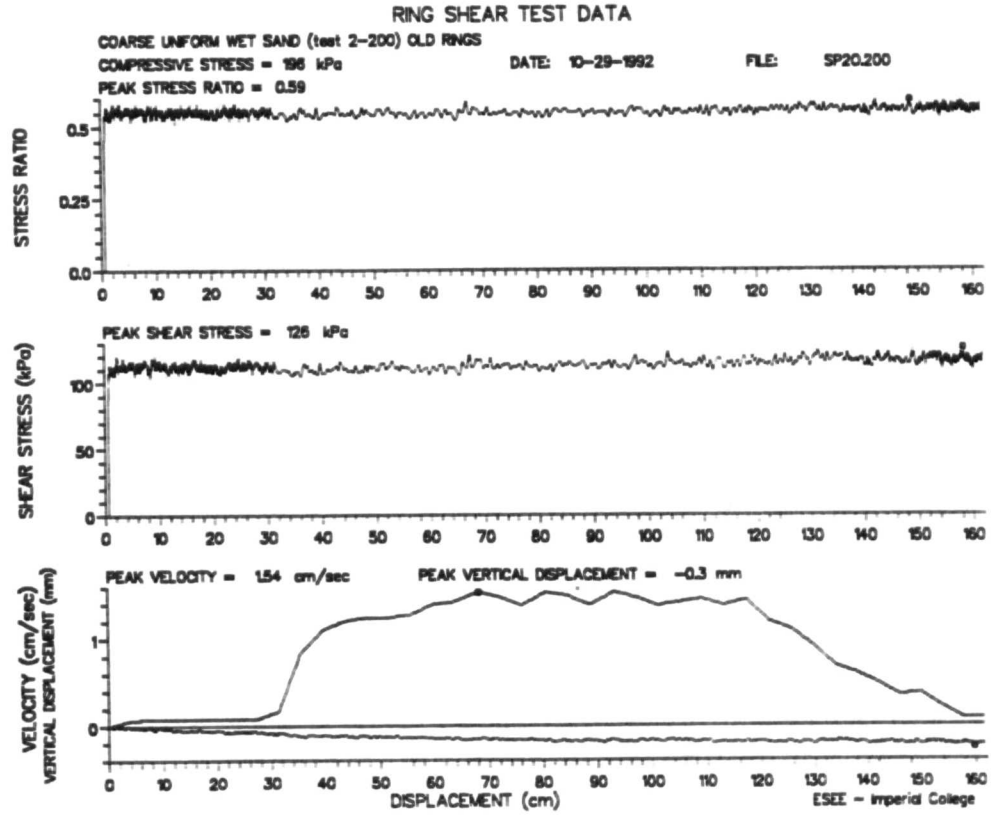


Fig. V.26.1 Fast shearing test on normally consolidated, wet, non-submerged medium-coarse sand (200 kPa)

Cowden till:

Test VI.3

σ_n= 100 kPa

$\phi_{residual}$= 30.7°

$\phi_{threshold}$= 41.0°

(Figures VI.3.1-3)

The sample was sheared at 0.09 mm/min and its measured strength reached its slow residual value of 30.7° after 110.2 mm of shearing. When this slow shearing part of the test was terminated after a displacement of 127.3 mm, the sample height had decreased by 0.4 mm.

The rate of displacement was increased suddenly to 1.39 cm/sec while the measured strength reached its threshold value of 41.0° at zero associated displacement. While shearing continued at the same rate of displacement, the measured strength appeared to be decreasing and the sample height to be increasing. After 31.1 cm of shearing the sample height had increased by 0.89 mm, and the measured strength had decreased to 27.0°. Further shearing led to further decrease in measured strength while the shape of the sample height curve indicated the formation of an undulating shear surface with a wave-length of 40 cm and an initial amplitude of 0.33 mm. After the sample had been sheared for 108.3 cm its strength had decreased to 15.9°. The rate of displacement was decreased suddenly from 1.39 cm/sec to 0.11 cm/sec, after the sample had been sheared for 200.4 cm, and for the following 24.6 cm of displacement it remained at that level while the strength appeared to be increasing from 16.3° to 21.9°. At that point the rate of displacement was increased again to 1.39 cm/sec. The measured strength stabilised within a range between 15.0° and 18.4° until the rate of displacement was decreased again to 0.11 cm/sec. The sample was sheared at 0.11 cm/sec between 295.7 cm and 303.8 cm of cumulative displacement while its strength appeared to be increasing from 16.3° to 21.9°. Subsequently, the rate of displacement was increased to 1.95 cm/sec and then to 8.20 cm/sec. The rate of displacement was changed, suddenly or gradually, from 1.95 cm/sec to 8.20 cm/sec and to 9.78 cm/sec before this fast shearing part of the test was terminated at 610 cm of cumulative displacement. However, no effect on the strength of the sample was observed. The latter had stabilised within a range between 15.0° and 18.4°. Meanwhile, the sample height curve gave evidence of the existence

However, as shearing at 1.15 cm/sec continued, the sample height appeared to increase while the measured strength decreased. After the sample had been sheared for 114.9 cm its strength stabilised within a range between 38.0° and 41.5°. The rate of displacement was changed, suddenly and gradually, from 1.15 cm/sec, to 0.125 cm/sec, to 1.68 cm/sec, to 10.0 cm/sec and to 2.5 cm/sec before the test was terminated, but no effect on the strength of the sample was observed.

Cowden till:

Normally Consolidated

Test VI.9

σ_n= 100 kPa

$\phi_{residual}$= 42.0°

$\phi_{threshold}$= 48.0°

(Figures VI.9.1-2)

The sample was sheared at 0.09 mm/min, and the measured strength reached its slow peak value of 45.8° after 4.88 mm of shearing while the sample height had decreased by 0.27 mm. Shearing continued at the same rate of displacement and the measured strength reached its slow residual value of 42.0° after the sample had been sheared for 78.7 mm. Meanwhile, the sample height had decreased by 0.49 mm. This part of the test was terminated after 9.95 mm of shearing.

The rate of displacement was increased suddenly to 1.72 cm/sec and the measured strength reached immediately its threshold value of 48.0°. Further shearing at the same rate of displacement led to an immediate decrease in strength which, after the sample had been sheared for 13.0 cm, stabilised within a range between 40.5° and 42.8°. The rate of displacement was decreased from 1.72 cm/sec to 0.09 cm/sec after the sample had been sheared for 126.1 cm and remained at that level for the following 21.5 cm while the strength of the sample fluctuated between 39.9° and 42.0°. After the sample had been sheared for 147.6 cm the rate of displacement was suddenly increased back to 1.72 cm/sec. This increase of the rate of displacement had initially no effect on the strength of the sample but after 36.3 cm of shearing at 1.72 cm/sec that strength decreased, and stabilised within a range between 36.9° and 42.0°. The rate of displacement varied between 1.72 cm/sec and 0.09 cm/sec before the test was terminated, but no effect on the measured strength was observed.

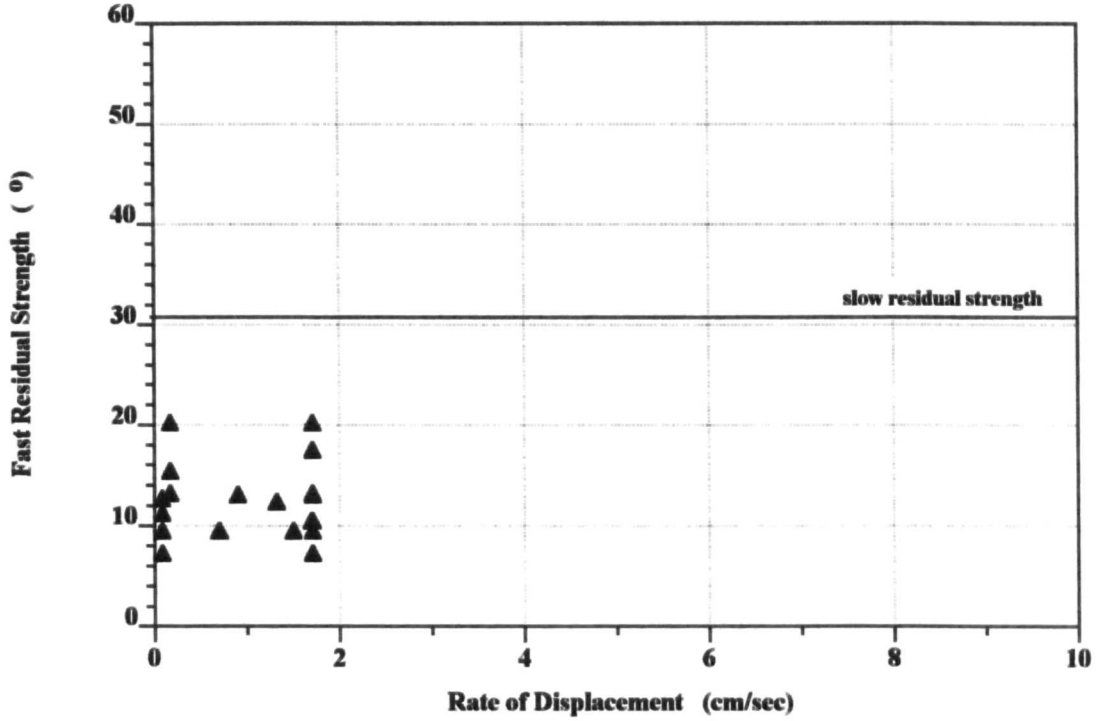


Fig. 6.3.45 Rate dependent characteristics of over-consolidated, submerged, Cowden till (300 kPa)

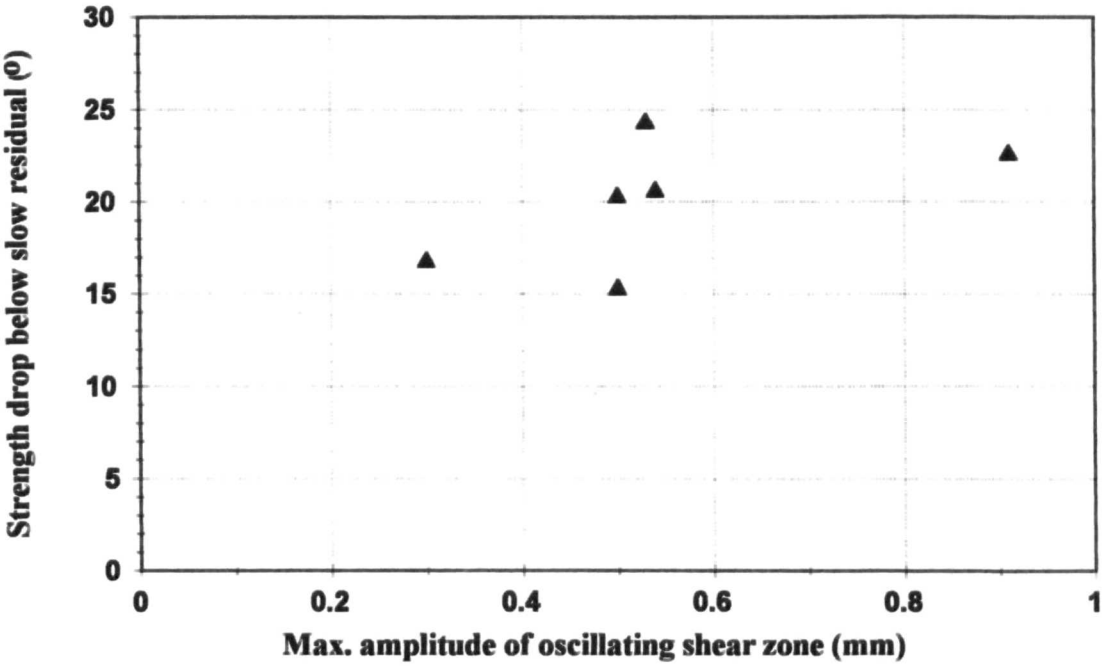


Fig. 6.3.46 Amplitude of oscillating shear zone effect on the drop of fast residual strength below slow residual strength

undulating shear surface which was reflected on the stress ratio/displacement curve and on the sample height curve. The undulating shear surface had an amplitude of 0.13 mm and a wave-length of 40.1 cm. The measured sample strength also oscillated around a mean value of 25.9° with an amplitude of 11.6°. It should be noted, however, that, as the rate of displacement was increased, the strength of the sample increased, and vice versa. When the rate of displacement was decreased to 0.083 cm/min, towards the end of the test, the strength dropped to 19.2° only to increase again when the rate was suddenly increased to 1.33 cm/min.

Normally Consolidated, Submerged

The soil which was tested in this series of tests was prepared identically with the soil which was tested in the previously described series of tests except for the fact that after the sample had been placed into the rings the assembly was submerged in water.

TEST IV. 6

σ_n= 50 kPa

$\phi_{residual}$= 14.5°

$\phi_{threshold}$= 29.8°

(Figures IV.6.1-2)

The sample was sheared for the first time to residual at the constant rate of displacement of 0.09 mm/min and after 1.99 mm of shearing it reached its slow peak strength of 29.7° as its height decreased by 0.075 mm. Continued shearing at the same rate of displacement led to a decrease in strength until it reached its slow residual value of 14.5° after a displacement of 70.2 mm and a further decrease in sample height by 0.04 mm. This slow shearing part of the test was terminated after 129.2 mm of shearing and an overall decrease in sample height of 0.125 mm below its level at the end of the consolidation stage.

The rate of displacement was increased to 0.63 cm/sec and the measured sample strength reached its threshold value of 29.8° while the sample height decreased by 0.067 mm below its level at the end of the slow shearing part of this test. After 17.6 cm of shearing at 0.63 cm/sec the sample had dilated by 0.15 mm and its strength had dropped to 13.1°. Nevertheless, further shearing led to a decrease in sample height probably because of contraction of the soil matrix within the shear zone

0.085 cm/sec and then back to 1.43 cm/sec until the sample had been sheared for a cumulative 280 cm but no effect on the strength of the sample was observed. Some internal ring friction continued to influence the shape of the stress ratio curve, but this should not be taken into account upon assessing the behaviour of the sample. After 280 cm of shearing, the rate of displacement was increased to 1.79 cm/sec and then gradually to 10.0 cm/sec and the measured strength reached the level of 47.1° at zero associated displacement before it stabilised at a range between 38.7° and 40.4°. Any change of the rate of displacement, sudden or gradual, from 1.79 cm/sec to 10 cm/sec had no effect on the strength of the sample. The test was terminated after 640 cm of shearing.

Test V.10

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 45.8°

$\phi_{threshold}$= 55.0°

(Figures V.10.1-2)

The sample was sheared at 0.09 mm/min and the measured strength reached its slow peak value of 48.2° while the sample height appeared to be increasing. Further shearing led to a decrease in strength which reached its slow residual value of 45.8° after a displacement of 11.4 mm. This part of the test was terminated after 133 mm of shearing.

The rate of displacement was increased suddenly to 0.59 cm/sec and the measured strength reached its threshold value of 55.0° at zero associated displacement. After 6.5 cm of shearing the strength had dropped to 45.8° and, while the rate of displacement was being increased to 1.32 cm/sec, it stabilised between 41.4° and 44.6°. The rate of displacement was varied between 1.32 cm/sec and 0.059 cm/sec before the sample reached a cumulative shear displacement of 223 cm, but this had no effect on the strength of the sample. The same was the case for the second stage of this fast shearing part of the test. The rate of displacement was increased suddenly to 1.76 cm/sec and the strength reached the value of 48.3° at zero associated displacement before it stabilised within a range between 41.4° and 44.6°. Any further changes of the rate of displacement, sudden or gradual, from 1.76

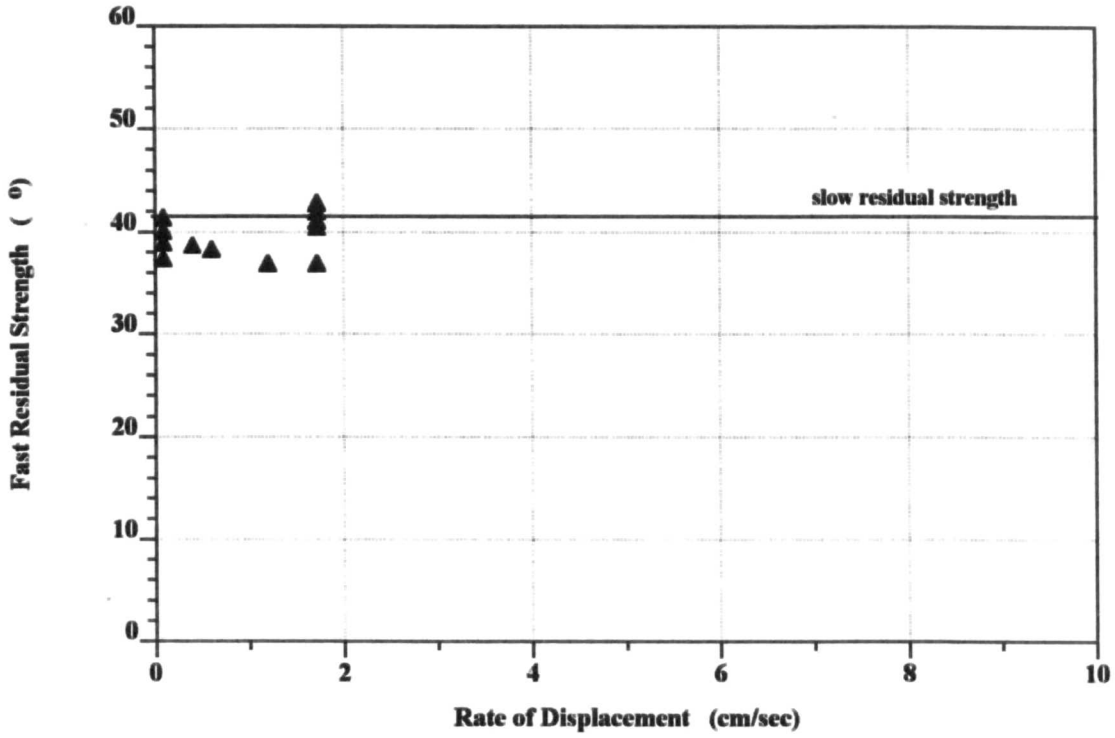


Fig. 6.3.55 Rate dependent characteristics of normally consolidated, completely dry, Cowden till (100 kPa)

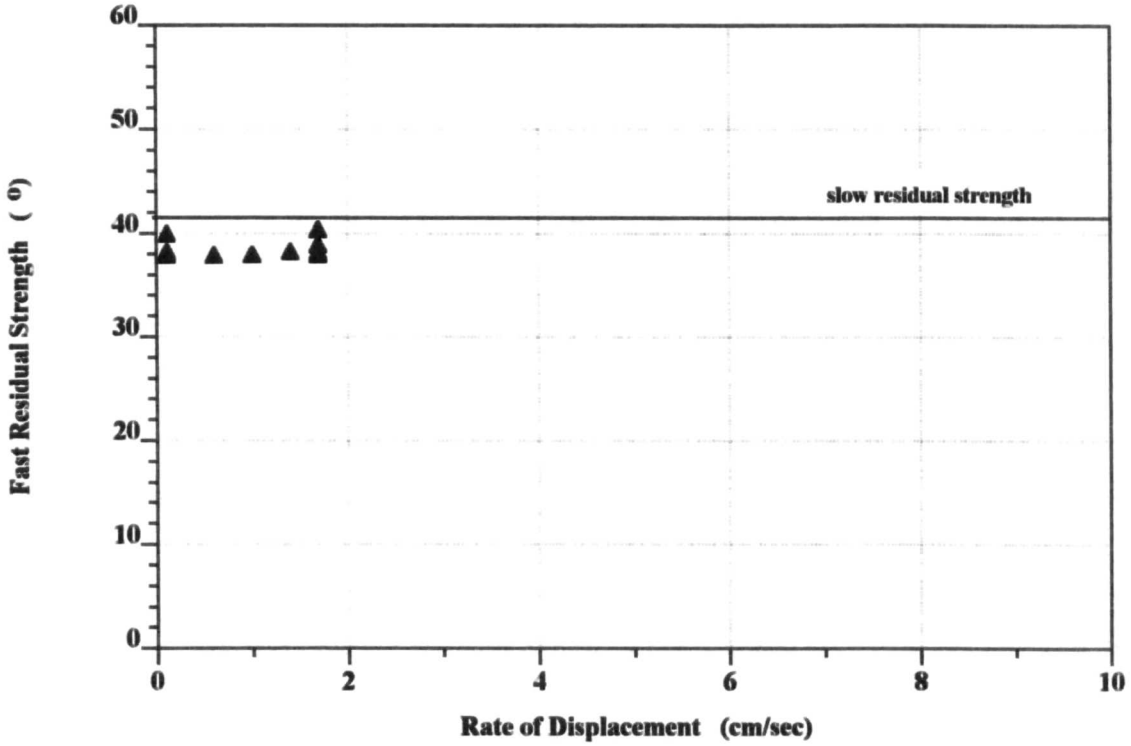


Fig. 6.3.56 Rate dependent characteristics of normally consolidated, completely dry, Cowden till (200 kPa)

shearing at the same rate led to further decrease in strength to 12.6° after a shear displacement of 23.8 cm while the sample height increased by 0.25 mm. After 34.4 cm of shearing at 1.76 cm/sec the sample height had increased by 0.88 mm and its strength to 18.4°. While shearing was continued, an undulating shear surface appeared to be forming with an initial amplitude of 0.85 mm and a wave-length of 41.3 cm. The amplitude of this undulating shear surface gradually decreased and after 431.2 cm of shearing it reached its minimum value of 0.18 mm while its wave-length remained constant throughout the test. The strength of the sample was not affected when the rate of displacement was increased gradually to 3.55 cm/sec and stabilised around 16.7° until it dropped below its slow residual value after 130.2 cm of cumulative shearing. Continued shearing at rates of displacement which varied between 1.76 cm/sec and 5.5 cm/sec led to further decrease in sample strength which oscillated between 9.7° and 1.7°. After 450 cm of shearing, while the amplitude of the undulating shear surface decreased, the measured strength of the sample appeared to increase and the measured strength varied between 12.8° and 6.2°. The test was terminated after 670 cm of shearing and it should be emphasised at this point that any increase in the rate of displacement above 1.76 cm/sec did not influence the measured strength. Furthermore, any change in the rate of displacement had no effect on the measured strength after it had dropped below its slow residual value. As far as the new ring design is concerned, it should be pointed out that the soil loss associated with only 0.9 mm decrease in the sample height after 670 cm of fast shearing was considered a major improvement on the apparatus.

TEST IV.11

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 11.4°

$\phi_{threshold}$= 24.7°

(Figures IV.11.1-2)

The sample was sheared to residual at 0.09 mm/min and reached its slow peak strength of 14.6° after 0.55 mm of shearing. After 42.2 mm of shearing at the same rate of displacement the sample had reached its slow residual strength of 11.4° while its height had decreased by 0.107 mm. This decrease in sample height should be attributed to particle orientation and soil loss through the gap of the rings. Further

shearing continued at the same rate of displacement, the sample height increased to 0.09 mm, and, after a displacement of 9.4 cm, the measured strength decreased to 47.7°. Further shearing at the same rate of displacement led to a continuous decrease of the strength of the sample which stabilised within a range between 39.1° and 43.9° after the sample had been sheared for 51.4 cm. Any further change of the rate of displacement, sudden or gradual, from 1.69 cm/sec to 0.09 cm/sec had no effect on the strength of the sample. By the time this test was terminated, after a displacement of 480 cm, the sample height had decreased by 1.4 mm because of excessive soil loss.

Test V.17

$\sigma_n = 100$

$\phi_{residual} = 48.0°$

$\phi_{threshold} = 52.9°$

(Figures V.17.1-2)

The sample exhibited again the same behaviour with the non-submerged sample under the same normal stress and dilated upon shearing at 0.09 mm/min. After 1.9 mm of shearing the measured strength had reached its slow peak value of 51.8° while the sample height increased by 0.06 mm. Further shearing was associated with an additional 0.025 mm increase in sample height and the measured strength reached its slow residual value of 48.0° after a displacement of 32.3 mm. This part of the test was terminated after 115.9 mm of shearing and the decrease in sample height which was observed after the residual state had been established was attributed to soil loss.

The rate of displacement was increased suddenly to 1.68 cm/sec and the measured strength reached immediately its threshold value of 52.9°. Shearing continued at the same rate of displacement and the sample height increased by 0.11 mm while the strength of the sample decreased and stabilised within a range between 42.0° and 46.7°. The rate of displacement was decreased suddenly to 0.11 cm/sec after the sample had been sheared for a cumulative 210.7 cm and after a further 18.3 cm it was increased gradually back to 1.68 cm/sec. However, these changes in the rate of displacement had no effect on the strength of the sample. The test was terminated after 379.8 cm of shearing because the sample height had decreased by 1.1 mm, due to excessive soil loss, and there was danger of damaging the load cells.

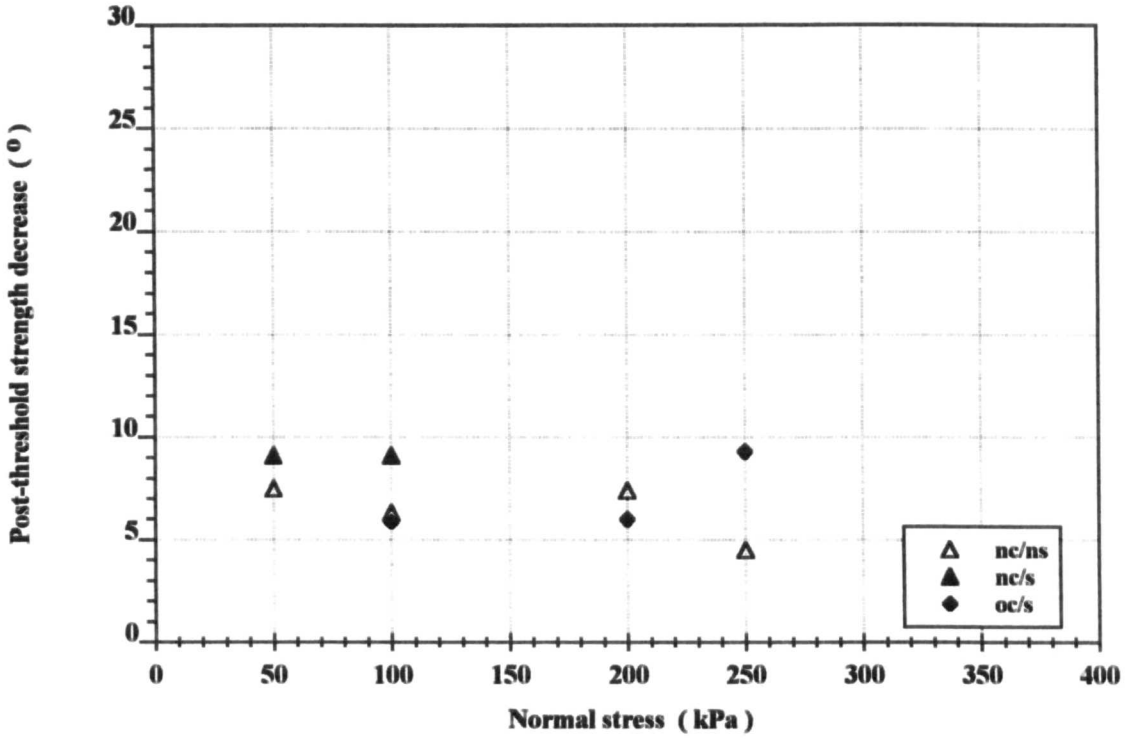


Fig. 6.4.7 Normal stress effect on post-threshold strength decrease of sandy-silt samples

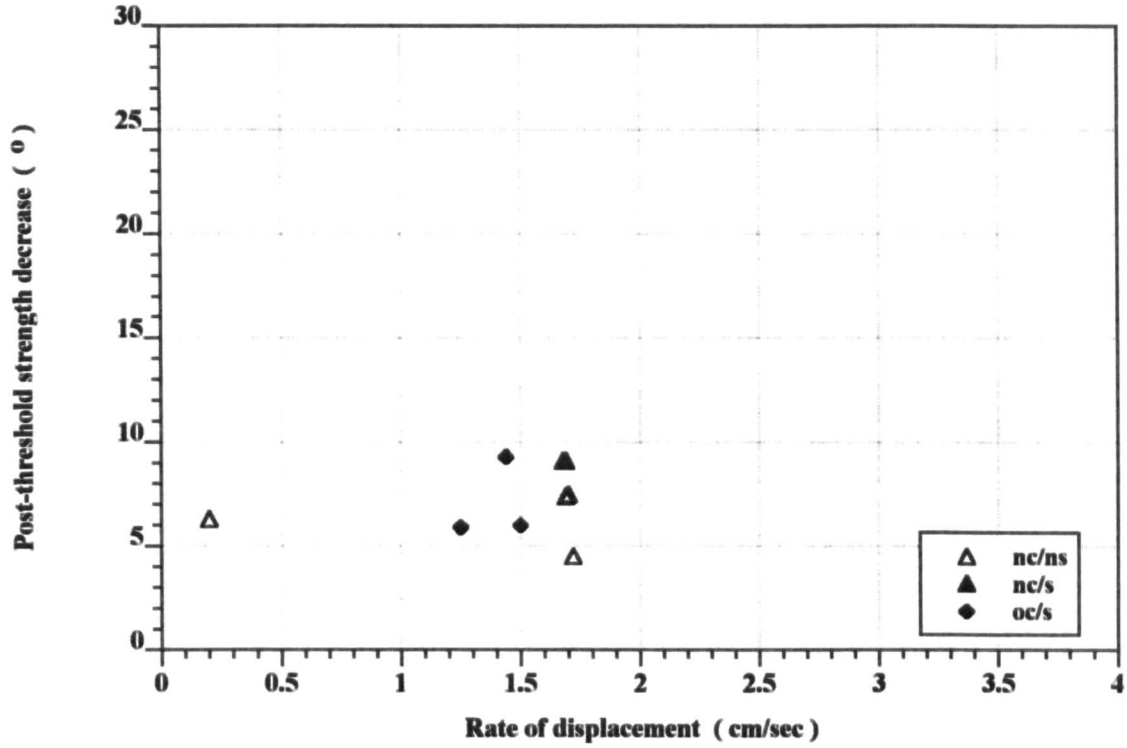


Fig. 6.4.8 Rate of displacement effect on post-threshold strength decrease of sandy-silt samples

strength decreased gradually and exhibited viscous characteristics. At 0.24 cm/sec the measured strength was 39.4° and at 0.09 cm/sec it was 38.3°. It should be noted at this point that the fast residual of this sample was not affected by any change in the rate of displacement between 0.9 cm/sec and 1.64 cm/sec. The final sequence of increasing/decreasing of the rate of displacement in this test gave similar results with the previous sequences. When sheared at 1.64 cm/sec the fast residual strength of this sample varied between 42.7° and 41.8° while when sheared at 0.09 cm/sec it stabilised around 39.9°. The test was terminated after 340 cm of shearing.

Test IV.15

σ_n= 200 kPa

$\phi_{residual}$= 31.4°

$\phi_{threshold}$= 39.5°

(Figures IV.15.1-2)

The sample was sheared at 0.09 mm/min and reached its slow residual strength of 31.4° after 123.5 mm of shearing, at which point its height had decreased by 0.138 mm.

After 251.8 mm of shearing at 0.09 mm/min the rate of displacement was increased suddenly to 1.78 cm/sec and the sample reached its threshold strength of 39.5° at zero associated shear displacement. Further shearing led to a decrease in measured strength to 32.9° while the sample height increased by 0.011 mm. As shearing continued at the same rate of displacement the sample appeared to be contracting and its strength increased to 38.5° after 75 cm of shear displacement. At that point the rate of displacement was decreased gradually to 0.114 cm/sec and the measured strength decreased to 36.3°. However, the measured strength began to respond to the decrease in the rate of displacement only after the latter had dropped below 1.2 cm/sec. The rate of displacement was increased to 1.78 cm/sec for the following 20.3 cm of shearing and the strength increased to 38.5°. The rate of displacement was subsequently dropped to 0.061 cm/sec and the measured strength decreased to 34.3°. The remainder of the test was targeted to investigate the rate of displacement above which the measured sample strength remains constant. The rate of displacement was increased gradually and the sample exhibited positive rate effects. At 0.25 cm/sec the measured strength was 36.2° and at 0.72 cm/sec it was 37.4°. The same values were measured upon decreasing the rate of displacement.

cm/sec after 101.1 cm of shearing and then decreased, in the same manner, to 0.1 cm/sec but these changes had no effect on the measured strength.

Normally Consolidated, Moist

Test V.24

σ_n= 50 kPa

$\phi_{threshold}$= 36.9°

$\phi_{residual}$= 35.8°

(Figure V.24.1)

The sample was sheared at 0.075 cm/sec for the first 26 cm of this test and the measured strength reached immediately a threshold value of 36.9°. Further shearing was associated with a decrease in measured strength which reached its residual value of 35.8° after 2.2 cm of shear displacement while the sample height had increased by 0.43 mm. As shearing continued, the sample height decreased due to particle crushing and at the end of the test it had dropped 0.07 mm below its initial level. The rate of displacement was increased gradually to 1.60 cm/sec after the sample had been sheared for 70.7 cm and then decreased, in the same manner, to 0.11 cm/sec, but no effect on the strength of the sample was observed.

Test V.25

σ_n= 100 kPa

$\phi_{threshold}$= 25.0°

$\phi_{residual}$= 25.0°

(Figure V.25.1)

The same testing procedure which was used in the previously described tests was followed in this test as well. However, contrary to what has been the case for all the previously described tests on sand, the sample height decreased from the beginning of the test, and, after the sample had been sheared for 165.4 cm, it had dropped by 0.125 mm. The sample was sheared at 0.078 cm/sec for the first 26.6 cm of the test while the measured strength reached its residual value of 25.0°. The rate of displacement was increased gradually to 1.54 cm/sec and then decreased, in the same manner, to 0.094 cm/sec but no effect on the measured strength was observed.

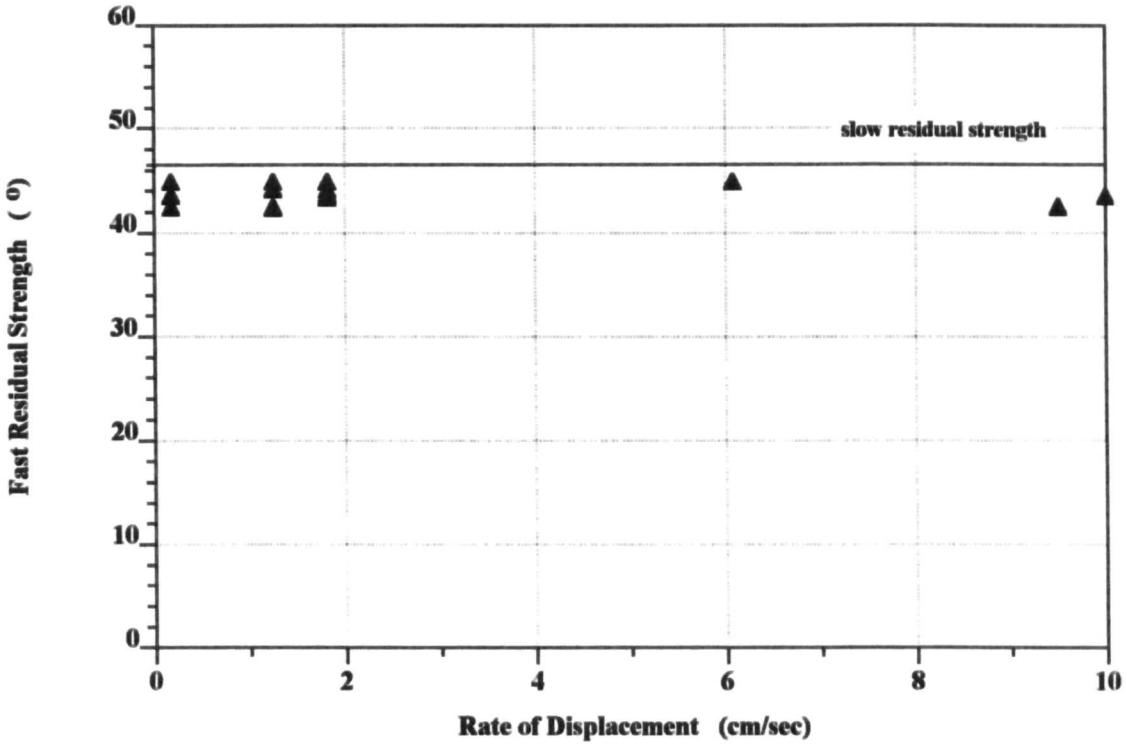


Fig. 6.4.17 Rate dependent characteristics of normally consolidated, submerged pure silica sample (200 kPa)

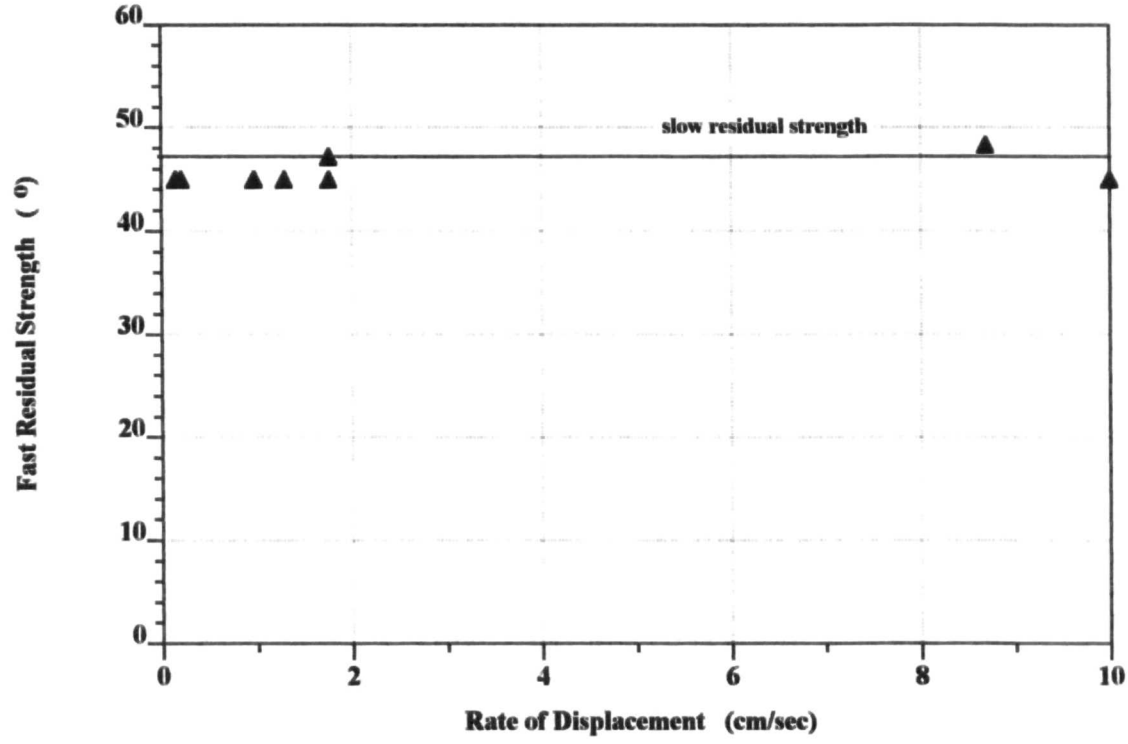


Fig. 6.4.18 Rate dependent characteristics of over-consolidated, submerged pure silica sample (50 kPa)

displacement, the sample height had increased by a further 0.117 mm and the measured strength had dropped to 23.2°. The measured strength continued to decrease while it appeared from the shape of the sample height curve that an undulating shear surface with an initial amplitude of 0.525 mm and a wave-length of 40 cm had been formed. After 70 cm of shearing the measured strength had dropped to 13.6° and stabilised at 7.02°. During the first 180.8 cm of shearing of this test the rate of displacement varied from 0.07 cm/sec to 1.25 cm/sec but this had no profound effect on the fast residual strength of the sample which, from the early stages of this test, had dropped below its slow residual value. During the second part of this test, between 196 cm and 570 cm of shearing, the rates of displacement which were applied on the sample varied between 2.0 cm/sec and 10 cm/sec. From 196 cm to 311.6 cm of shear displacement the fast residual strength of the sample varied between 4.9° and 9.03°, from 311.6 cm to 432.5 cm it varied between 9.03° and 12° and during the last 137.5 cm of this test it varied between 12° and 15.5°. Again it should be noted that no rate of displacement effect was observed on the measured sample strength once it had stabilised below its slow residual value.

Test IV.20

σ_n= 300 kPa

$\phi_{residual}$= 31.7°

$\phi_{threshold}$= 52.0°

(Figures IV.20.1-2)

The residual state of this sample was established after 83.6 cm of shearing at 0.09 mm/min, at which point the sample height had decreased by 0.08 mm. Further shearing led to an additional decrease in sample height by 0.02 mm before this slow shearing part of the test was terminated after 126 mm of shear displacement.

The rate of displacement was suddenly increased to 1.35 cm/sec and the sample reached its threshold strength of 52.0° at zero associated displacement. Further shearing at the same rate of displacement was associated with a substantial decrease in measured strength, while the sample appeared to be dilating. After 16.6 cm of shearing, the measured strength had dropped to its slow residual value of 31.7° as the sample height had increased by 0.5 mm. The strength continued to decrease until it reached its minimum value of 10.2° after 68.2 cm of shearing. From the shape of the sample height curve it was concluded that an undulating shear surface with an

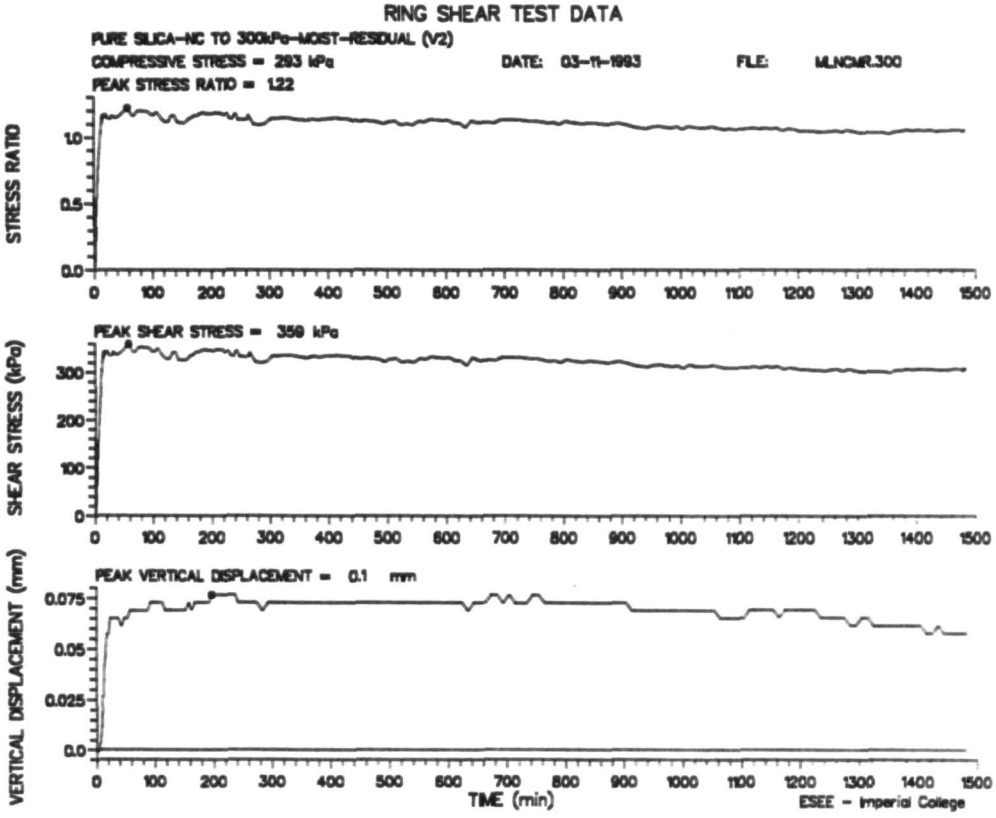


Fig. V.4.1 Slow shearing test on normally consolidated, non-submerged pure silica (300 kPa)

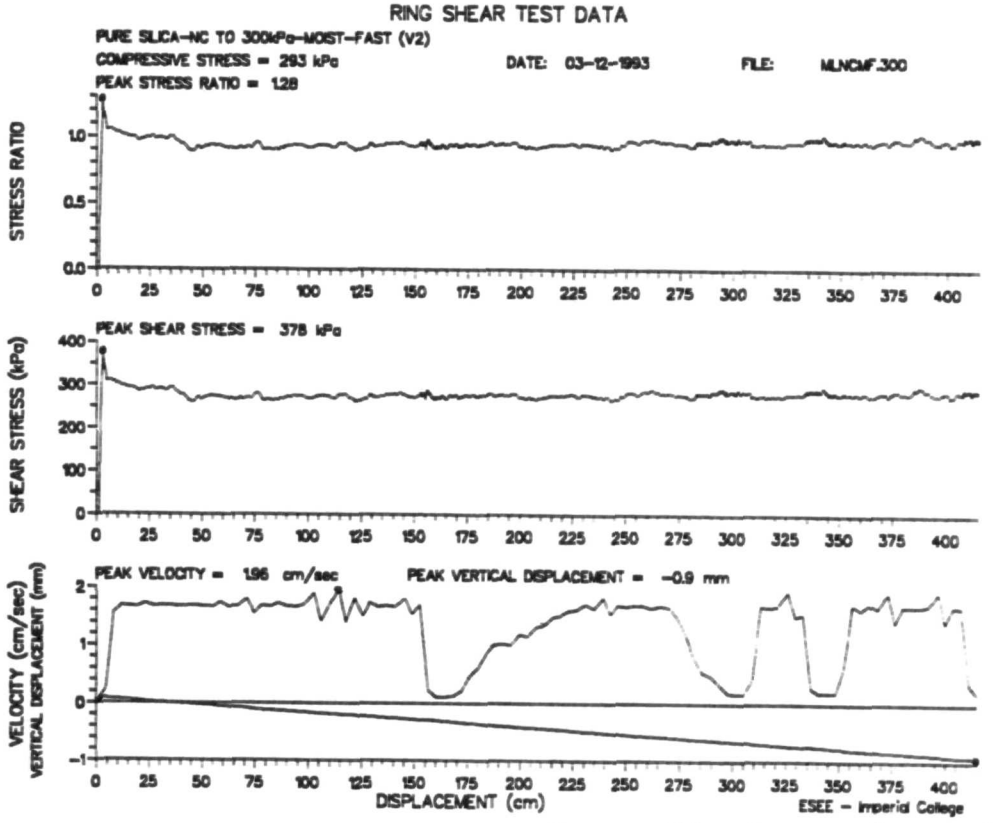


Fig. V.4.2 Fast shearing test on normally consolidated, non-submerged pure silica (300 kPa)

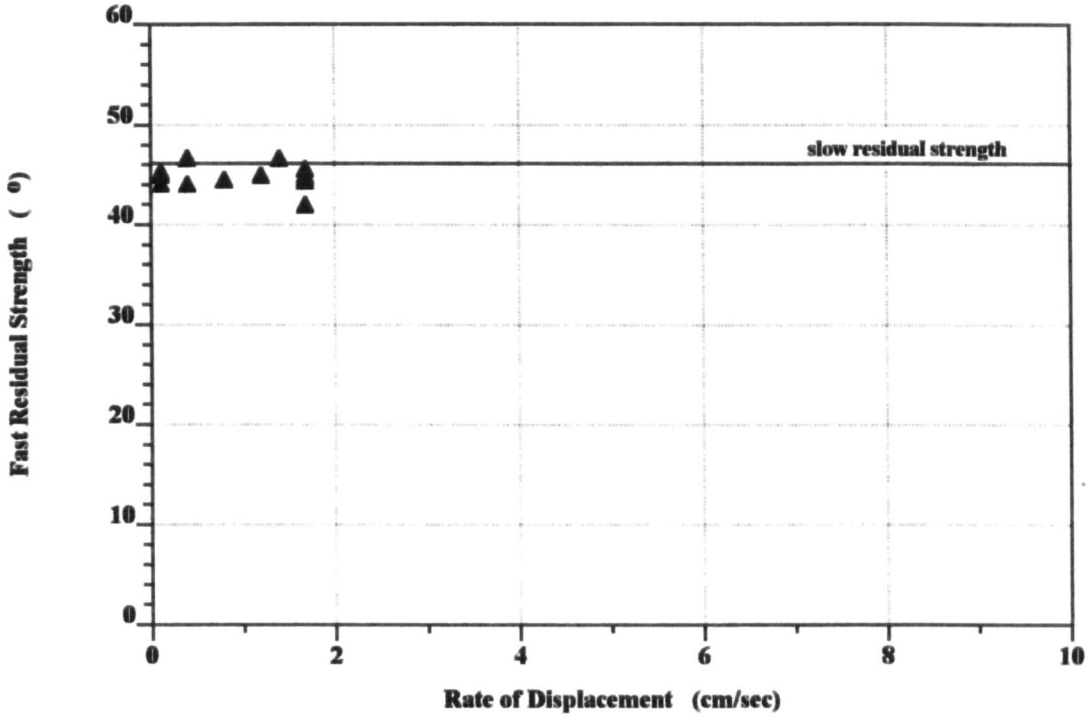


Fig. 6.4.27 Rate dependent characteristics of normally consolidated, submerged, sandy-silt sample (100 kPa)

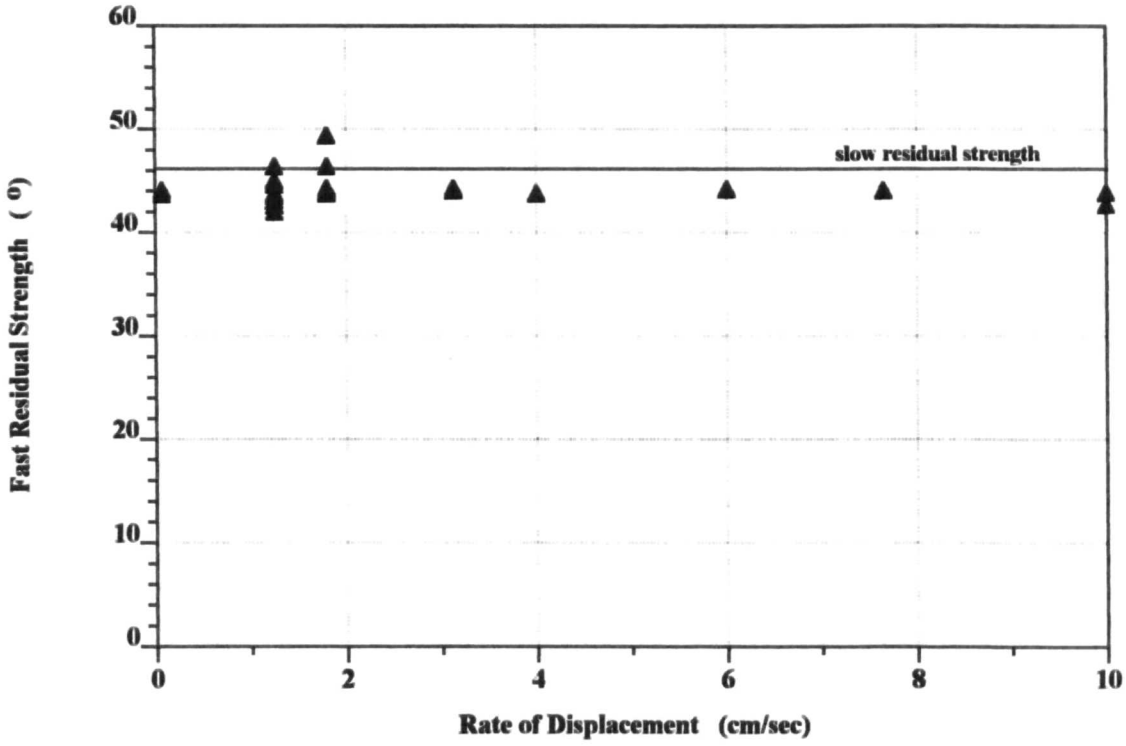


Fig. 6.4.28 Rate dependent characteristics of over-consolidated, submerged, sandy-silt sample (100 kPa)

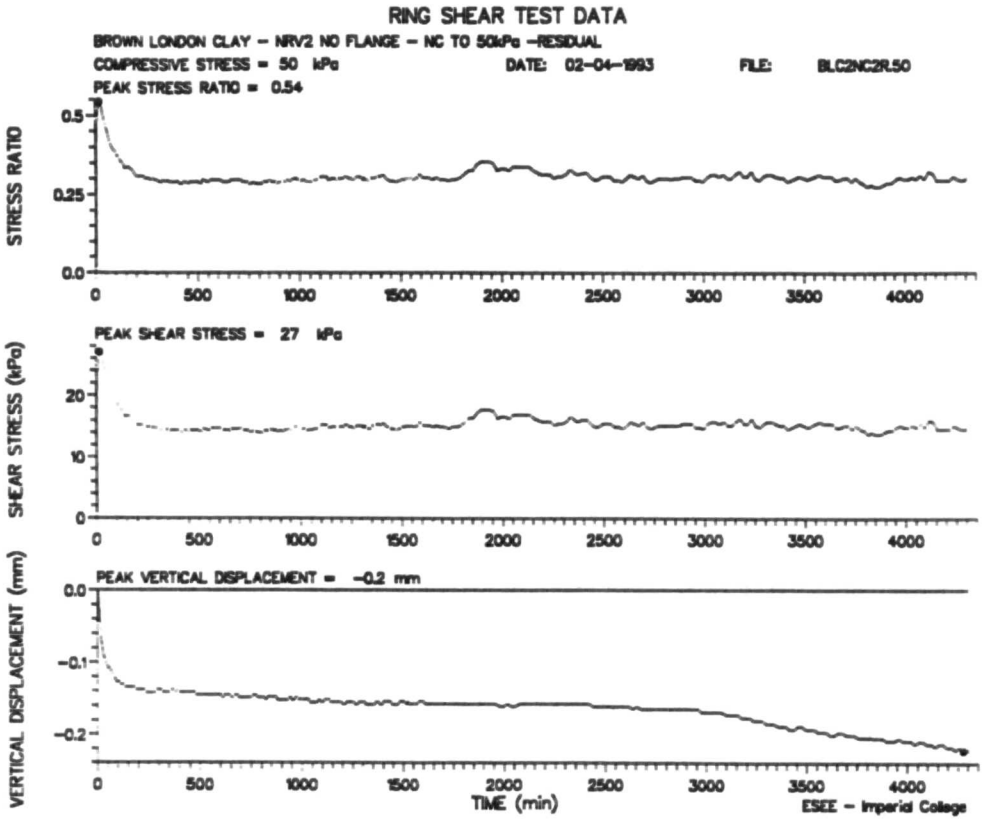


Fig. IV.1.1 Slow shearing test on normally consolidated, non-submerged brown London clay (50 kPa)

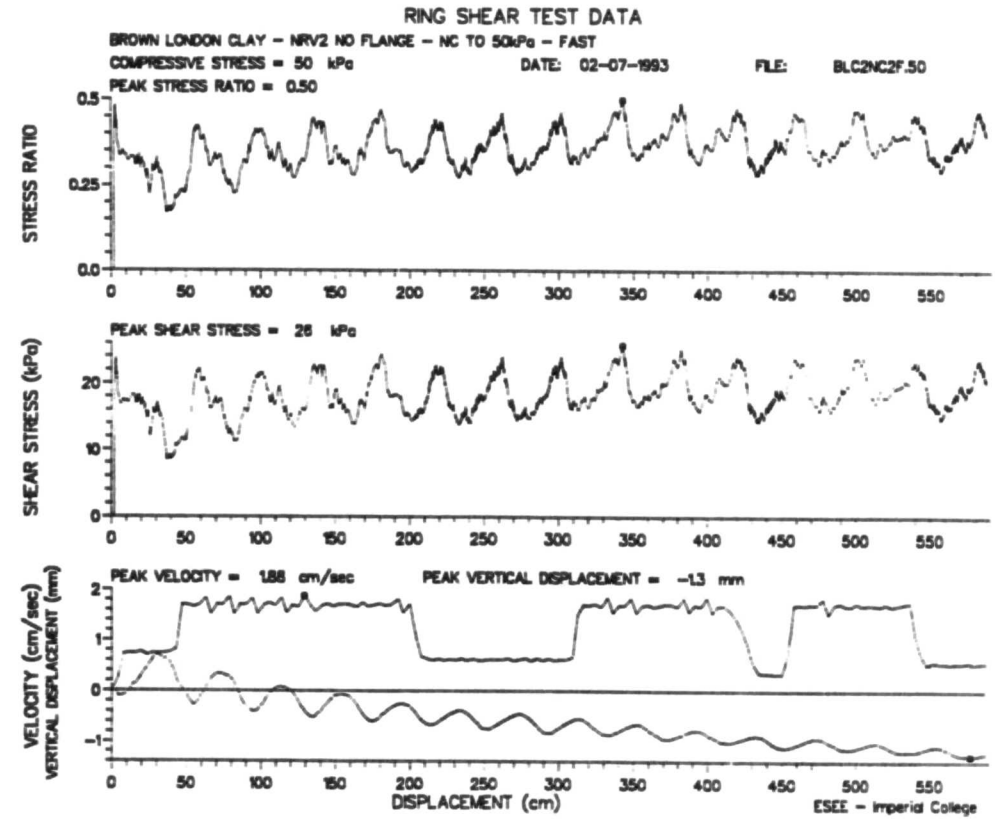


Fig. IV.1.2 Fast shearing test on normally consolidated, non-submerged brown London clay (50 kPa)

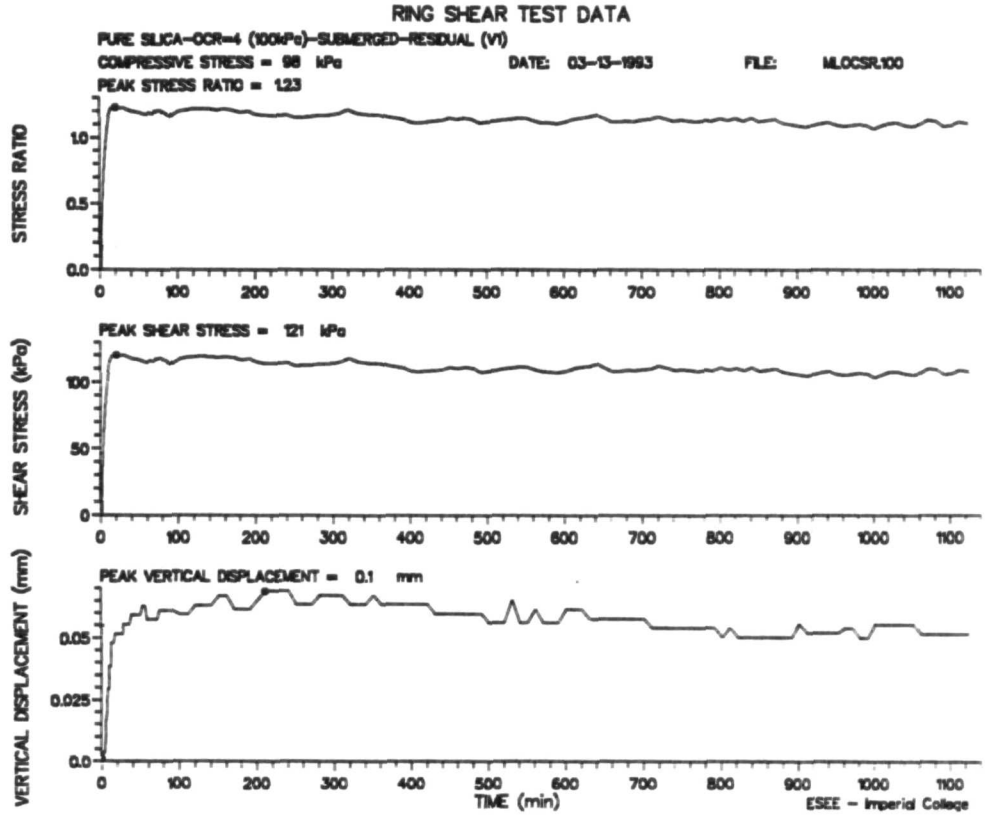


Fig. V.9.1 Slow shearing test on over-consolidated, submerged pure silica (100 kPa - OCR= 4)

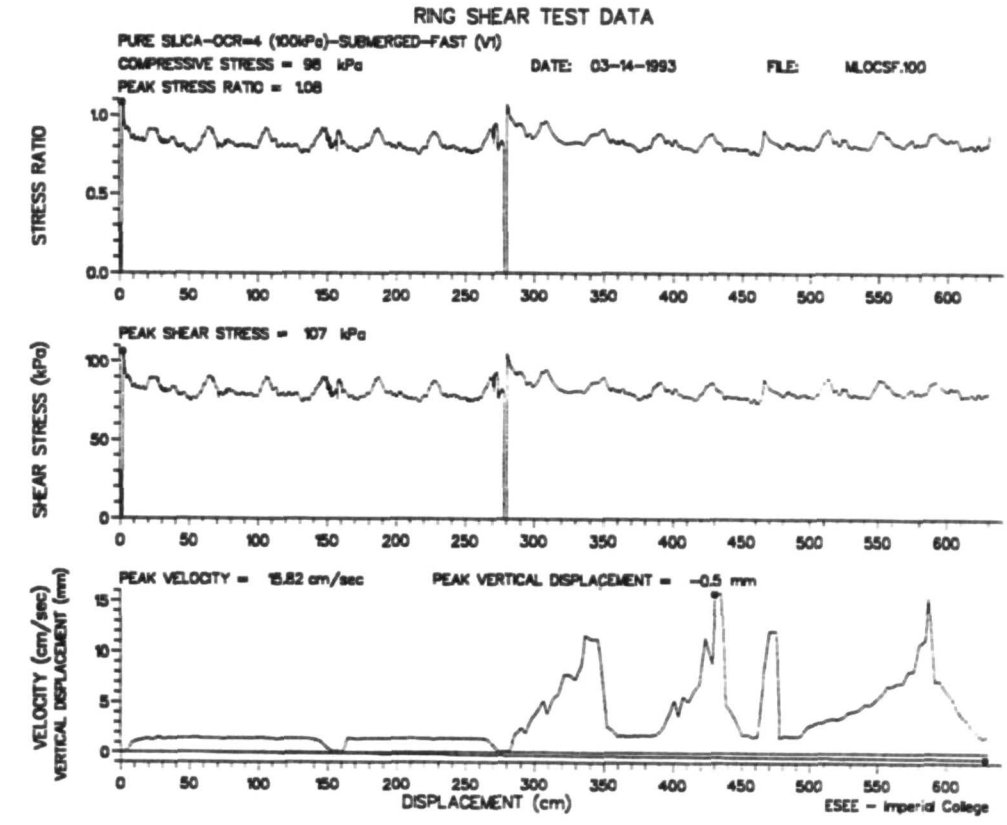


Fig. V.9.2 Fast shearing test on over-consolidated, submerged pure silica (100 kPa - OCR= 4)

that there are five types of variation of the residual strength with increasing rates of displacement depending on whether the fast residual strength stabilises above, below, or at, the slow residual strength. However, the present work suggests that the shear displacements in fast tests conducted with the original confining rings are not sufficient for a steady residual strength to be achieved, and that only in fast tests conducted with the new confining rings is the displacement sufficient for the fast residual strength of soils to be established.

7.1.2 Test Results

Two principal types of soil behaviour during fast shearing are identified [Fig. 6.5.1] from the results of this investigation. These are defined as **positive** and **negative rate effects**, depending on whether the fast residual strength stabilises above or below the slow residual strength, respectively. Also, both the *threshold strength* and the *post-threshold strength decrease* (defined as the decrease in measured strength below its threshold value and observed immediately after the threshold strength has been achieved) are observed. The post-threshold strength decrease is associated with an increase in sample height and may be related to the dilation or increase in porosity of the shearing matrix (Lemos, 1986; Tika, 1989), the development of a gap (or void) between the rotating and stationary parts of the sample, and an increase in pore pressure immediately after the application of the fast rates of displacement (Yagi *et al*, 1991).

The behaviour of soils during fast shearing is controlled by their plasticity, the shape of their shear surface, and the presence of water in their environment.

The positive rate effect [*Type I* behaviour] occurs with plastic soils (PI>12 %, LL>27 %, and CF>30 %) which are not submerged in water, regardless of the shape of their shear surface, and with plastic soil samples which are submerged in water and have a nearly planar shear surface. In these cases, the fast residual strength stabilises above the slow residual strength, increases with increasing rate of displacement, and reaches its maximum value when the rate of displacement approaches 1.5 cm/sec. The results from this investigation indicate that the fast residual strength of plastic soil samples which exhibit a positive rate effect may be 150 to 250 % higher than their slow residual strength. Any increase in the rate of displacement above 1.5 cm/sec does not appear to

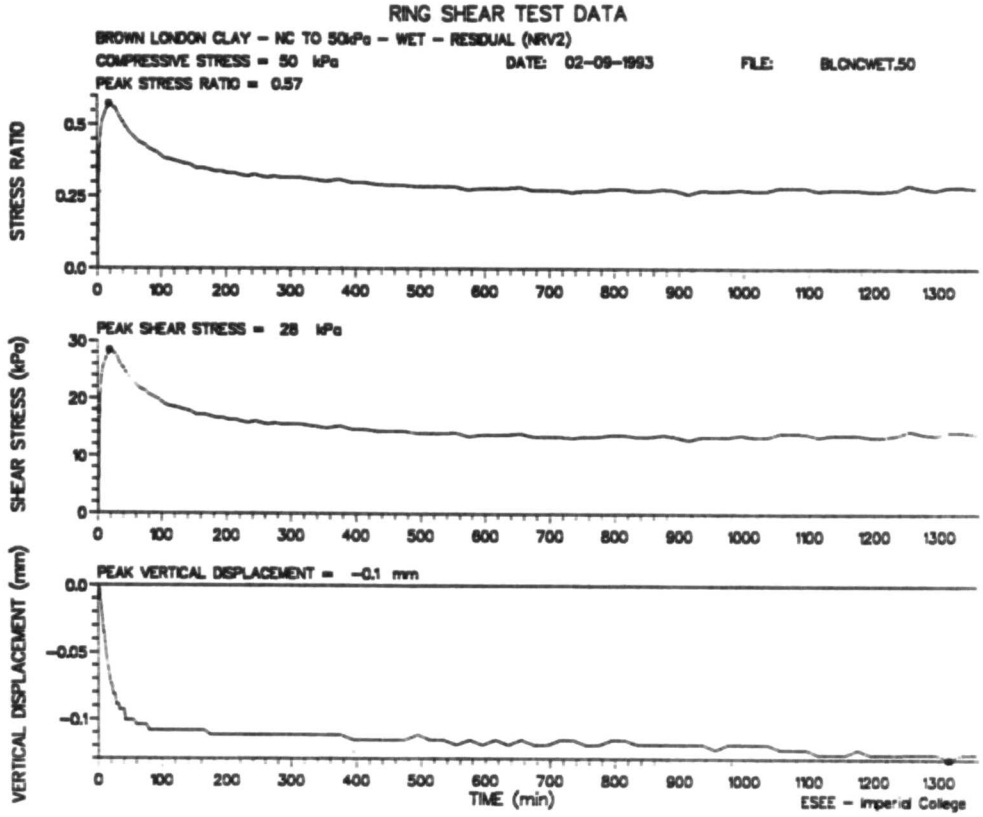


Fig. IV.6.1 Slow shearing test on normally consolidated, submerged brown London clay (50 kPa)

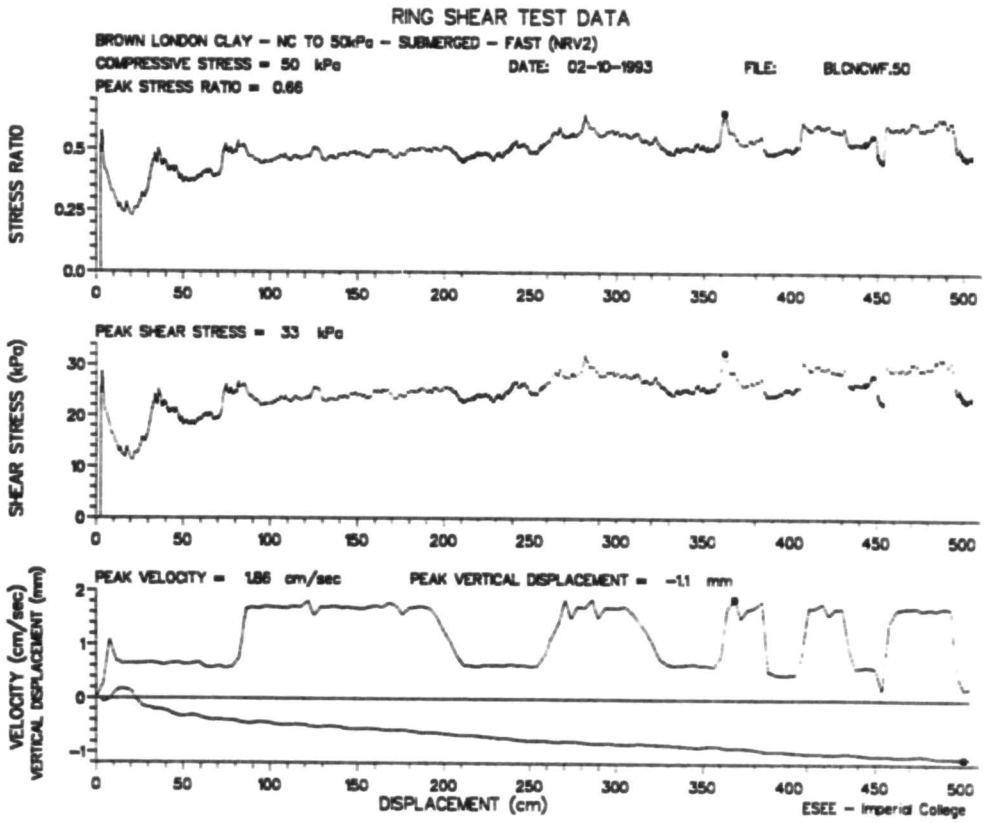


Fig. IV.6.2 Fast shearing test on normally consolidated, submerged brown London clay (50 kPa)

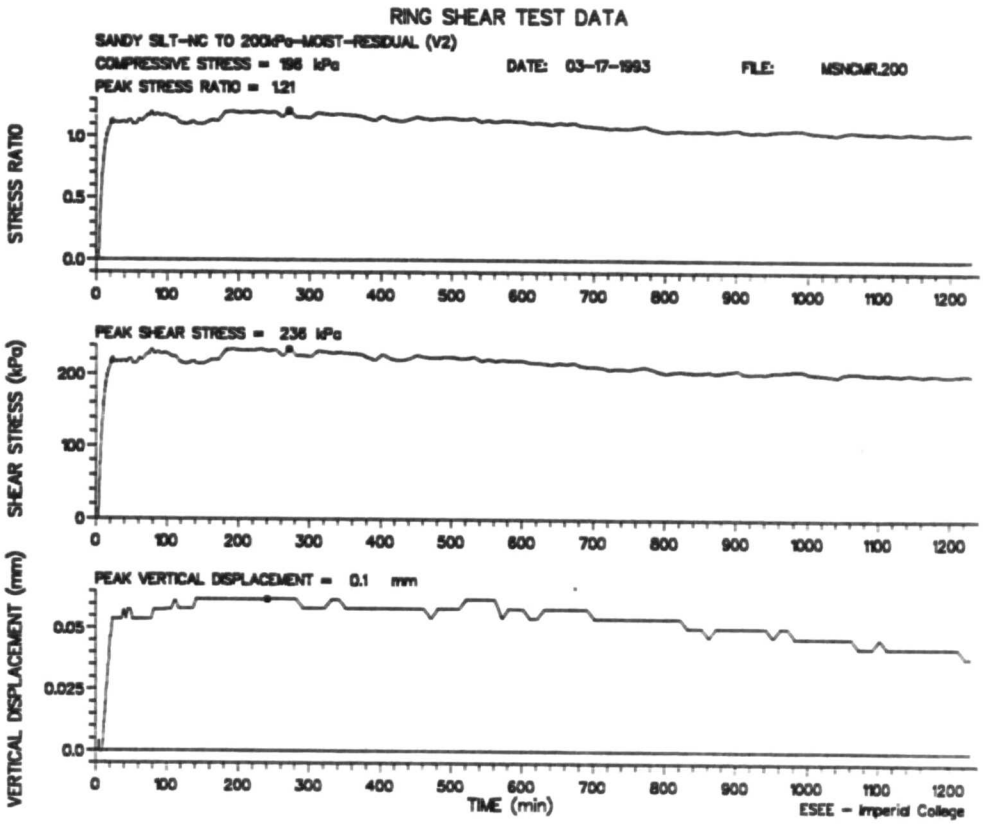


Fig. V.14.1 Slow shearing test on normally consolidated, non-submerged artificial sandy-silt (200 kPa)

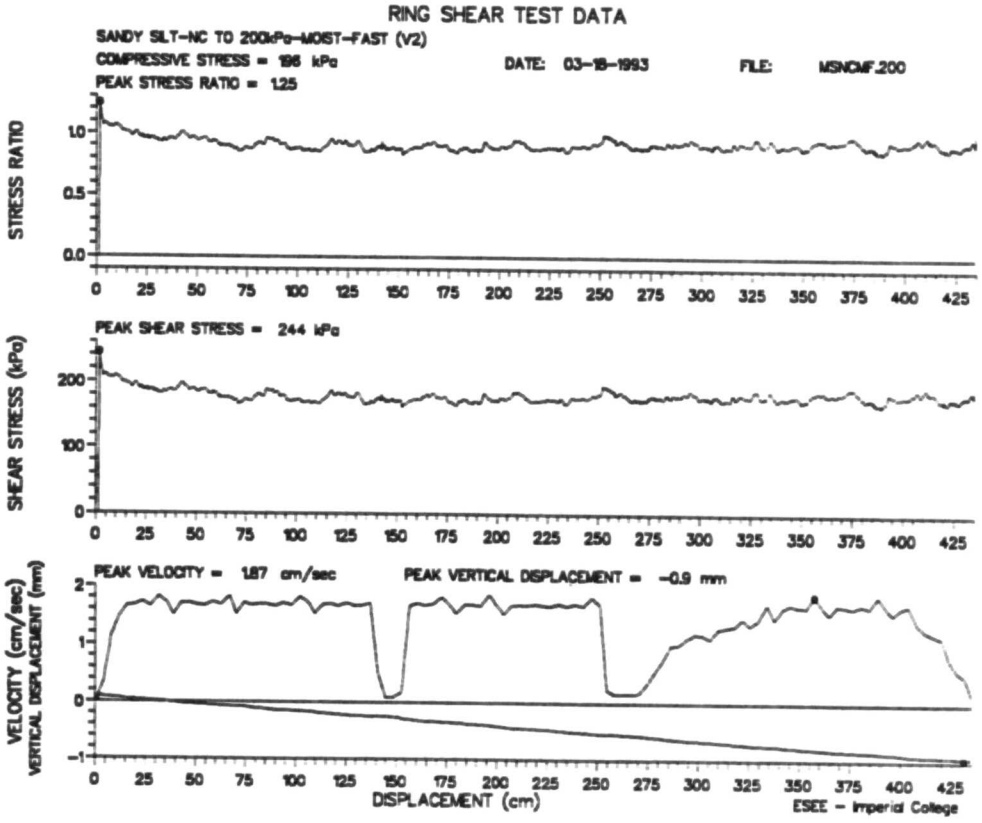


Fig. V.14.2 Fast shearing test on normally consolidated, non-submerged artificial sandy-silt (200 kPa)

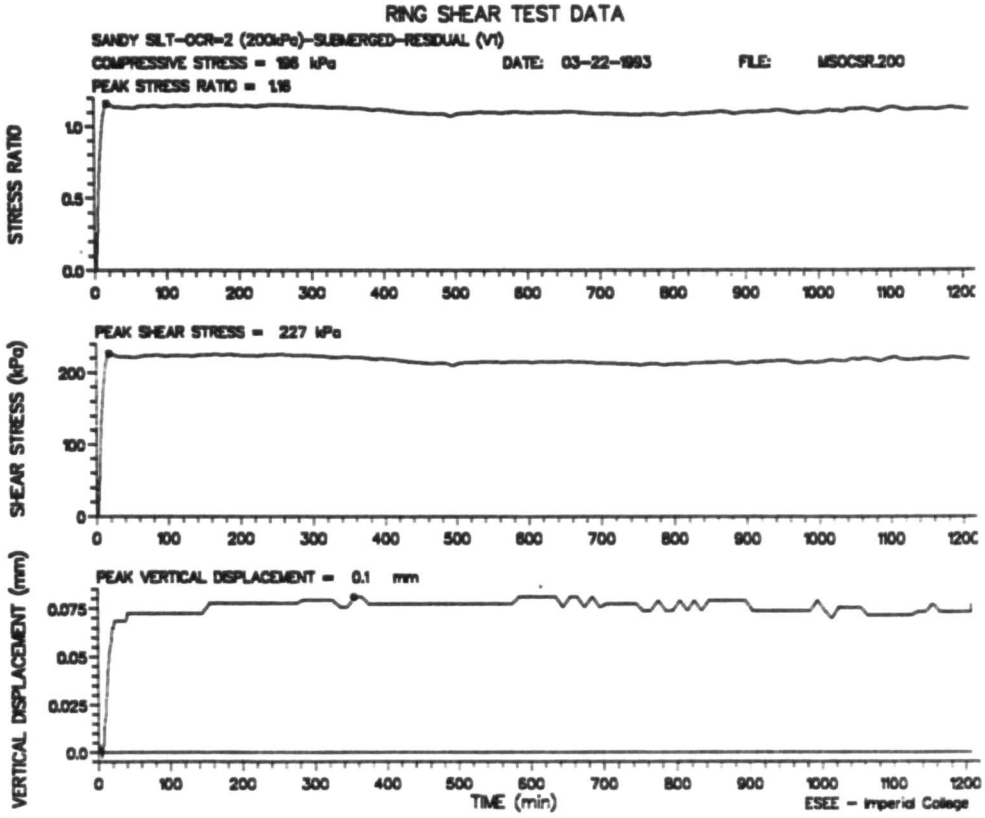


Fig. V.19.1 Slow shearing test on over-consolidated, submerged artificial sandy-silt (200 kPa - OCR= 2)

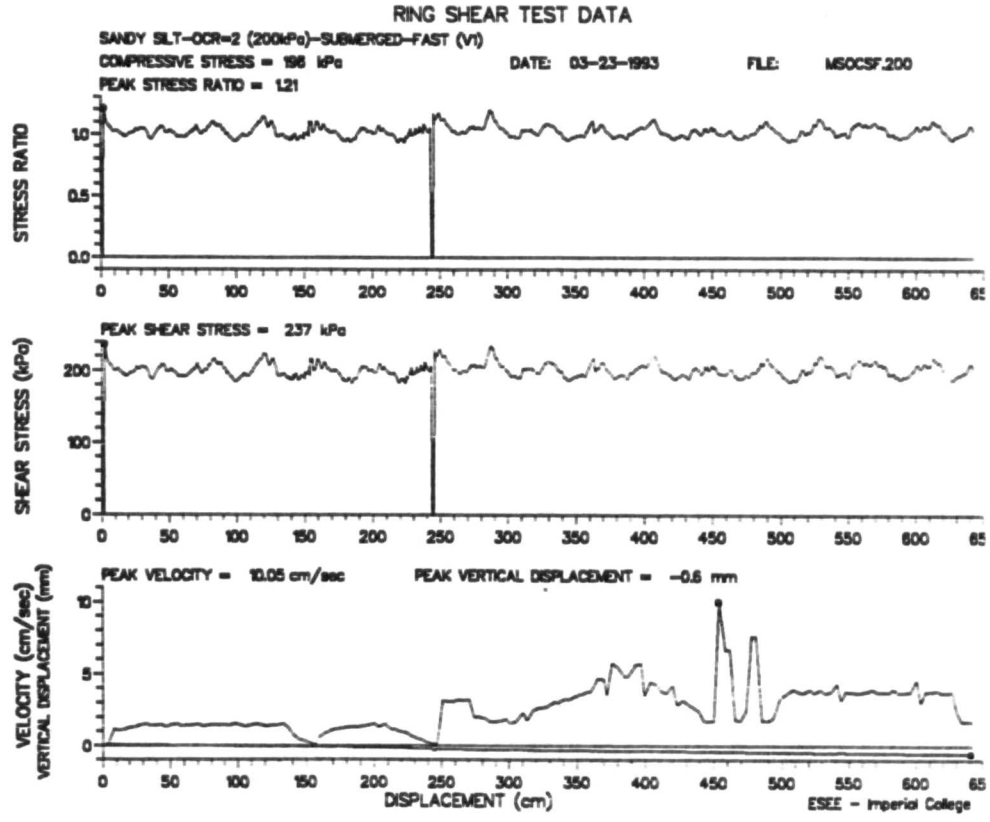


Fig. V.19.2 Fast shearing test on over-consolidated, submerged artificial sandy-silt (200 kPa - OCR= 2)

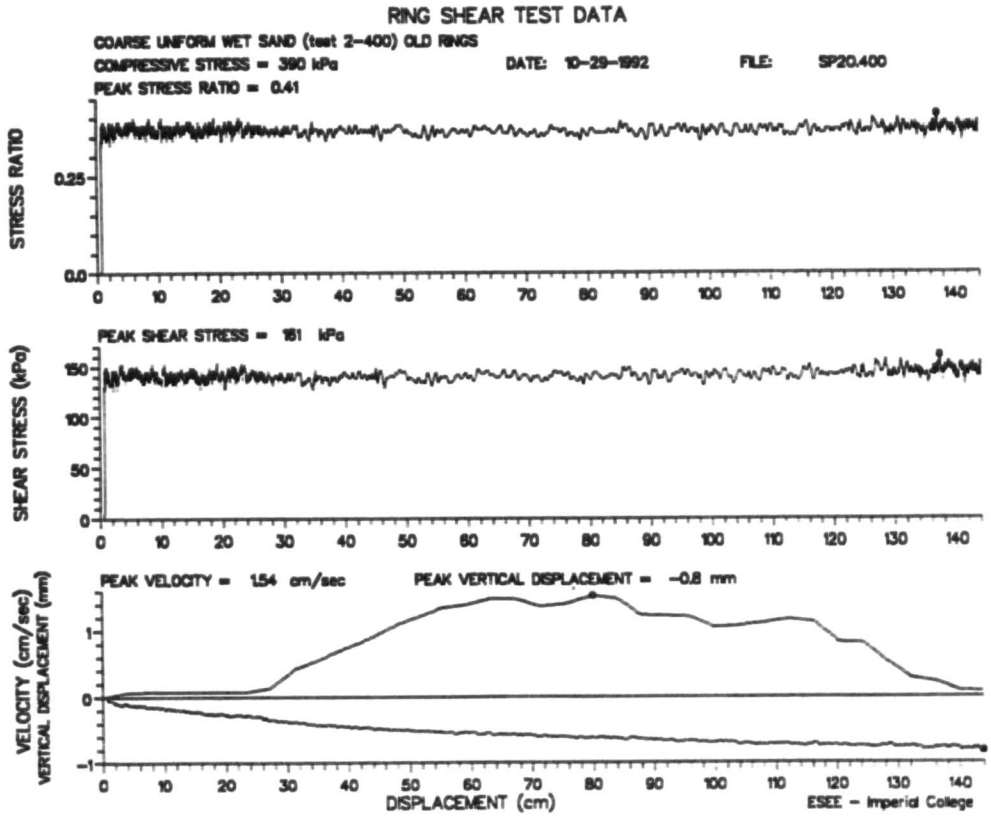


Fig. V.27.1 Fast shearing test on normally consolidated, wet, non-submerged medium-coarse sand (400 kPa)

of an undulating shear surface whose amplitude did not seem to decrease substantially with shearing.

The third part of this test consisted of shearing the sample at 0.09 mm/min until the measured strength stabilised at its slow residual value of 30.7°.

Test VI.4

σ_n= 200 kPa

$\phi_{residual}$= 30.9°

$\phi_{threshold}$= 47.1°

(Figures VI.4.1-3)

The sample was sheared at 0.09 mm/min and the measured strength reached its slow residual value of 30.9° after 25.2 mm of shear displacement while the sample height had decreased by 0.11 mm. This slow shearing part of the test was terminated after the sample had been sheared for 40.9 mm and while the sample height had decreased by an additional 0.02 mm due to soil loss.

The rate of displacement was suddenly increased to 1.29 cm/sec, and the measured strength reached immediately its threshold value of 47.1°. Further shearing at the same rate of displacement led to a continuous decrease in measured strength while the sample height curve revealed the formation of an undulating shear surface. After the sample had been sheared for 24.9 cm at the same rate of displacement, its strength had dropped to 30.8° and its height had increased by 0.11 mm. Further shearing was associated with further decrease in strength while the sample height increased by an additional 0.17 mm. As shearing at 1.29 cm/sec continued, the sample height curve suggested the existence of an undulating shear surface which had an initial amplitude of 0.29 mm and a wave-length around 40 cm. The measured strength stabilised within a range between 13.8° and 19.4° until the rate of displacement was decreased to 0.09 cm/sec after 176 cm of shearing. The rate of displacement remained at that level for a further 33.6 cm of shearing, during which, the measured strength increased from 18.4° to 26.0°. The rate of displacement was again increased to 1.29 cm/sec while the strength of the sample dropped and stabilised within a range between 10.8° and 18.5° until the sample had been sheared for a cumulative 277.7 cm. At that point, the rate of displacement was again decreased to 0.09 cm/sec, and the strength of the sample increased from 14.0° to 23.0°. This fast shearing part of the test was terminated after a displacement of 292.3 cm and throughout its duration the

Test VI.10

σ_n= 200 kPa

$\phi_{residual}$= 41.4°

$\phi_{threshold}$= 42.0°

(Figures VI.10.1-2)

The measured strength reached its slow residual value of 41.4° after the sample had been sheared for only 8.4 mm, and the slow shearing part of the test was terminated after the sample had been sheared for 125.4 mm.

The rate of displacement was increased suddenly to 1.69 cm/sec and the measured strength reached immediately its threshold value of 42.0°. Further shearing led to a decrease in strength which, from the early stages of this part of the test, stabilised within a range between 38.0° and 41.4°. Furthermore, the sample height appeared to be increasing slightly at the beginning of the test but, as shearing progressed, it decreased continually due to sample loss. The rate of displacement varied between 1.69 cm/sec and 0.11 cm/sec before the test was terminated at 480 cm of shear displacement, but no effect on the strength of the sample was observed.

Overconsolidated

Test VI.11

σ_n= 100 kPa

OCR= 4

$\phi_{residual}$= 42.6°

$\phi_{threshold}$= 44.3°

(Figures VI.11.1-2)

The same sample which was used in the previous test was consolidated to 400 kPa, unloaded to 100 kPa, and sheared at 0.09 mm/min. The measured strength reached its slow peak value of 52.4° while the sample height increased as the overconsolidated structure relaxed. Further shearing led to a decrease in the measured strength which reached its slow residual value of 42.6° when the sample had been sheared for 122.5 mm and while the sample height had increased by 0.055 mm. This part of the test was terminated after 167.2 mm of shearing.

The rate of displacement was increased suddenly to 1.71 cm/sec and the measured strength reached immediately its threshold value of 44.3°. While shearing at the same rate of displacement continued, the sample appeared to dilate and its

and continuing loss of soil through the gap of the confining rings. After 500 cm of displacement, at the end of the test, the sample height had decreased, overall, by 1.1 mm. Furthermore, as the sample height started to decrease and while the rate of shearing was still 0.63 cm/sec, the strength increased to 26.4° after 36 cm of shearing before it stabilised at 21.4°. The rate of displacement was once again increased suddenly to 1.68 cm/sec and the strength of the sample increased immediately to 26°. After 11.7 cm of shearing at the same rate, the strength decreased and stabilised at 25.5°. For the remainder of the test any decrease in the rate of displacement was followed by a decrease in the strength of the sample and vice versa.

TEST IV.7

σ_n= 400 kPa

$\phi_{residual}$= 11.3°

$\phi_{threshold}$= 20.3°

(Figures IV.7.1-2)

The sample was sheared at 0.09 mm/min and reached its slow peak strength of 17.2° after 3.11 mm of shear displacement while its height dropped 0.066 mm below its level at the end of the consolidation stage. Continued shearing at the same rate of displacement led to a decrease in strength until it reached its slow residual value of 11.3° after 24.7 mm of shearing and a further 0.264 mm decrease in height. This part of the test was terminated after 129.2 mm of shearing and a further 0.27 mm decrease in sample height.

The rate of displacement was increased suddenly to 1.31 cm/sec and the strength reached its threshold value of 20.3° at zero associated displacement. After 15 cm of shearing the strength had decreased to 14°, and the sample had dilated by 0.05 mm. As shearing continued at the same rate of displacement, the sample height decreased, and the strength stabilised around 16.3°. Any decrease in the rate of displacement was followed by a decrease in the measured strength, and the sample exhibited viscous characteristics. When the rate of shearing was increased or decreased gradually, the strength of the sample increased or decreased gradually as well. When the rate of shearing was changed suddenly, the strength of the sample also increased or decreased immediately. At the lowest rate of displacement of 0.075 cm/sec the strength of the sample decreased below its residual value at 10.1° but this decrease was not persistent every time the rate approached its lowest value. The

cm/sec, to 8.64 cm/sec, to 7.9 cm/sec and to 10.0 cm/sec had no effect on the strength of the sample. The test was terminated after 670 cm of shearing.

Test V.11

σ_n= 300 kPa

OCR= 1.5

$\phi_{residual}$= 47.7°

$\phi_{threshold}$= 62.8°

(Figures V.11.1-2)

The irregular shape of the stress ratio curve of the slow shearing part of this test indicated the effect of the contamination of the rings. However, the measured sample strength apparently reached its slow residual value of 47.7° after 87.7 mm of shearing while the sample appeared to dilate. This part of the test was terminated after 117.8 cm of shearing.

The rate of displacement was increased suddenly to 1.69 cm/sec and the measured sample strength reached its threshold value of 62.8° at zero associated displacement. Further shearing at the same rate of displacement led to a decrease in strength to 43.5°. The rate of displacement was changed, suddenly and gradually, from 1.69 cm/sec, to 6.18 cm/sec and to 3.9 cm/sec before the test was terminated after 1,000 cm of shearing but this had no effect on the strength of the sample.

Tests on Artificially Mixed Sandy-Silt

General:

The samples which were used for this set of tests were artificial mixtures of fine clean sand and pure silica and were prepared at the Soil Mechanics laboratory of Imperial College. The mixtures, in their dry state, consisted of 25 % sand and 75 % silica.

Normally Consolidated, Non-Submerged

Test V.12

σ_n= 50 kPa

$\phi_{residual}$= 50.6°

shearing at the same rate of displacement led to additional decrease in sample height while the measured strength remained around its slow residual value. Shearing at 0.09 mm/min was terminated after 131.1 mm of displacement at which point the sample height had dropped by an additional 0.063 mm, probably because of sample loss through the gap of the rings. The apparent increase in measured strength towards the end of this slow stage should be attributed to contamination of the rings.

The rate of displacement was immediately increased to 3.77 cm/sec and the sample reached its threshold strength of 24.7°. Further shearing at the same rate of displacement was associated with a dramatic decrease in strength below its slow residual value while the sample height increased by 0.56 mm. As shearing continued at 3.77 cm/sec, the sample continued to dilate and its strength to decrease. After 27.3 cm of shearing the sample height had increased by a further 0.29 mm and the strength had dropped to 5.1°. For the following 230 cm of shearing the rate of displacement varied between 2.11 cm/sec and 4.13 cm/sec, the strength of the sample stabilised around 4.1° and the undulating shear surface which had been established (as it was concluded from the shape of the sample height curve) had an amplitude of 0.29 mm after the first full rotation was completed which steadily decreased and after 230 cm of shearing it measured only 0.077 mm. The rate of displacement remained at 2.11 cm/sec until 248 cm of shearing but the measured strength of the sample appeared to be increasing while the amplitude of the undulating shear zone decreased. The rate of displacement was increased to 7 cm/sec after 300 cm of cumulative shearing but no profound effect on the measured strength was observed. Similarly, no profound effect on the measured strength was observed when the rate of displacement was increased to 9.41 cm/sec or 8.0 cm/sec. After 447.5 cm of shearing, the measured strength of the sample had increased to its slow residual value of 11.4° and at the end of the test, after 560 cm of shearing, it had reached the value of 12.1°.

TEST IV.12

σ_n= 300 kPa

OCR= 1.5

$\phi_{residual}$= 13.3°

$\phi_{threshold}$= 24.6°

ϕ_{peak}= 26.6°

(Figures IV.12.1-2)

Overconsolidated, Submerged

Test V.18

σ_n= 100 kPa

OCR= 4

$\phi_{residual}$= 46.4°

$\phi_{threshold}$= 49.7°

(Figures V.18.1-2)

The sample dilated on shearing at 0.09 mm/min and the measured strength reached its slow peak value of 47.9° after a shear displacement of 5.7 mm while the sample height had increased by 0.04 mm. As shearing progressed, the sample height continued to increase and the measured strength to decrease. The sample reached its slow residual strength of 46.4° after 19.5 mm of shearing while its height increased by an additional 0.027 mm. This part of the test was terminated after 129.2 mm of shearing.

The rate of displacement was increased suddenly to 1.25 cm/sec and the measured strength reached immediately its threshold value of 49.7°. Further shearing at the same rate of displacement led to a decrease in strength while the sample height increased by 0.14 mm. The rate of displacement was decreased from 1.25 cm/sec to 0.07 cm/sec after 160 cm of shearing while the strength of the sample had stabilised within a range between 42.0° and 45.9°. It should be noted at this point that the actual variation of the measured strength was indeed smaller and that same internal ring friction, which could not be taken into account, must have influenced the test results. After 277.2 cm of shearing the motor was stalled, the slow gear was disengaged and the rate of displacement was suddenly increased to 1.8 cm/sec while the measured strength reached the level of 49.4° at zero associated displacement. As shearing continued at the same rate of displacement the measured strength decreased and stabilised within a range between 42.0° and 45.9° until the test was terminated after a shear displacement of 670 cm. Any changes of the rate of displacement, sudden or gradual, from 1.8 cm/sec, to 10.0 cm/sec and to 7.65 cm/sec had no effect on the strength of the sample.

Test V.19

σ_n= 200 kPa

Again it was observed that the measured sample strength remained constant when the rate of displacement increased above 1.2 cm/sec. This fast shearing part of the test was terminated after 265 cm of cumulative displacement.

Test IV.16

σ_n= 400 kPa

$\phi_{residual}$= 31.4°

$\phi_{threshold}$= 38.7°

(Figure IV.16.1-2)

The sample was sheared at the drained rate of displacement of 0.09 mm/min and reached its state of residual after 75 mm of shearing. At that point the sample height had dropped by 0.196 mm below its level at the end of the consolidation stage and this should be attributed to both particle reorientation and sample loss. The sample was sheared for a cumulative 140.6 mm at the same rate of displacement before the test was terminated.

The rate of displacement was suddenly increased to 1.69 cm/sec and remained at that level for the first 190.8 cm of the fast shearing stage. The measured strength reached immediately its threshold value of 38.7° and further shearing led to a its decrease while the sample appeared to be dilating. After 16.1 cm of shearing at 1.69 cm/sec the sample height had increased by 0.34 mm and the measured strength had dropped to 27.1°. After 21.1 cm of shearing at the same rate of displacement the sample height was still 0.31 mm above its level at the end of the slow shearing part of this test and the measured strength had dropped further to 25.6°. As shearing continued at the same rate of displacement the sample appeared to be contracting and its strength increased to 36.2°. From the irregularities of the sample height curve it can be argued that the shear surface was not planar. Furthermore, the measured sample strength varied between 33.8° and 36.2°, always above the sample's slow residual value. After 190.8 cm of shearing, the rate of displacement was suddenly decreased to 0.023 cm/sec, however, the effect of this change on the measured strength was not clear because it was influenced by a 0.32 mm decrease of the sample height. The remainder of this test was aimed to investigate the viscous characteristics of the sample, the existence of a critical rate of displacement above which the fast residual strength of the sample remains constant, and whether the fast residual strength of this sample depended on shear displacement. The rate of displacement

Test V.26

σ_n= 200 kPa

$\phi_{threshold}$= 28.8°

$\phi_{resifual}$= 28.8°

(Figure V.26.1)

Apart from the fact that the decrease of the sample height along the duration of this test had almost doubled in comparison with that of the previously described test, this procedure gave no new evidence on the behaviour of the sample on fast shearing. The rate of displacement was gradually increased to 0.084 cm/sec and remained at that level until the sample had been sheared for 26.6 cm when the rate was increased, again gradually, to 1.54 cm/sec. Before the test was terminated, after a displacement of 161 cm, the rate was decreased to 0.077 cm/sec. The measured strength reached from the beginning of the test its residual value of 28.8° and was not affected by any further changes of the rate of displacement.

Test V.27

σ_n= 400 kPa

$\phi_{threshold}$= 20.3°

$\phi_{residual}$= 20.3°

(Figure V.27.1)

The sample was sheared at 0.082 cm/sec for the first 24 cm of this test while the measured strength reached its residual value of 20.3°. This test was terminated after the sample had been sheared for 144 cm and its height had decreased by 0.82 mm because of particle crushing. The rate of displacement was increased gradually to 1.54 cm/sec and then decreased back to its initial value, but no effect on the measured strength was observed.

initial amplitude of 0.5 mm and a wave-length of 40 cm had formed. Continued shearing at rates of displacement which varied between 1.35 cm/sec and 0.08 cm/sec gave no profound evidence about the effect of the changes in the rate of displacement on the fast residual strength of this sample as the measured sample strength fluctuated between 11.3° and 18°. The second stage of this fast shearing part of the test -from 106.8 cm of shearing to 265 cm of shearing- was associated with sudden or gradual increases and decreases of the rate of displacement. There was no clear evidence about the effects of changing rates of displacement on the fast residual strength of the sample.

Overconsolidated, Submerged

Test IV.21

σ_n= 100 kPa

OCR= 4

$\phi_{residual}$= 33.8°

$\phi_{threshold}$= 40.6°

(Figure IV.21.1-2)

The sample had been consolidated under 400 kPa and then unloaded to 100 kPa before it was sheared at 0.09 mm/min for a cumulative displacement of 261.25 mm. The measured strength reached its "slow" peak value of 50.4° after 2.79 mm of shearing while the sample height increased by 0.13 mm above its level at the end of the consolidation stage because of the relaxation of the overconsolidated structure. Further shearing led to a decrease in strength while the structure continued to relax. After 55.8 mm of shearing the sample height had increased by 0.193 mm and the strength had decreased to 41.9°. The sample reached its residual state after 247.0 mm of shearing.

The rate of displacement was suddenly increased to 1.68 cm/sec but the threshold value was not as evident as in previous tests. After 3.33 cm of shearing the measured strength reached a "threshold" value of 40.6°, the sample appeared to be dilating, and after 20.3 cm of shearing its height had increased by 0.27 mm and the measured strength had decreased to 35.2°. After that point, the sample height started to decrease the measured strength increased. After 70.3 cm of shearing at 1.68 cm/sec the rate of displacement was suddenly decreased to 0.096 cm/sec and the measured strength dropped from 40.9° to 37.7°, but, nevertheless, it remained above

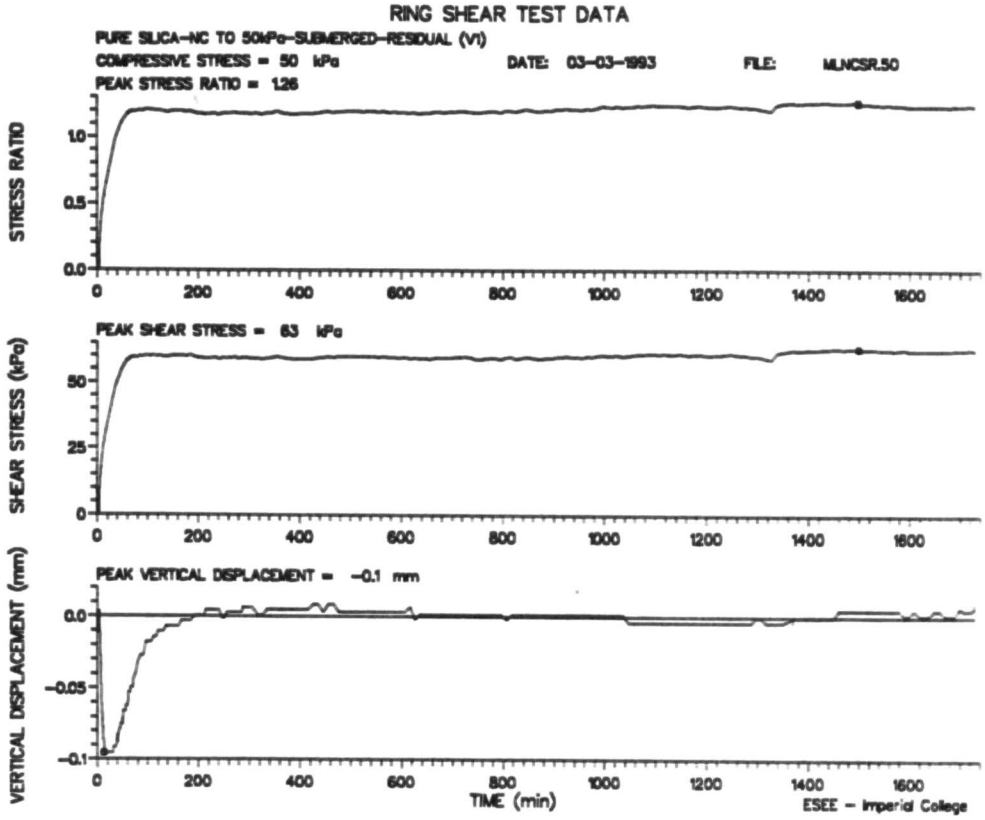


Fig. V.5.1 Slow shearing test on normally consolidated, submerged pure silica (50 kPa)

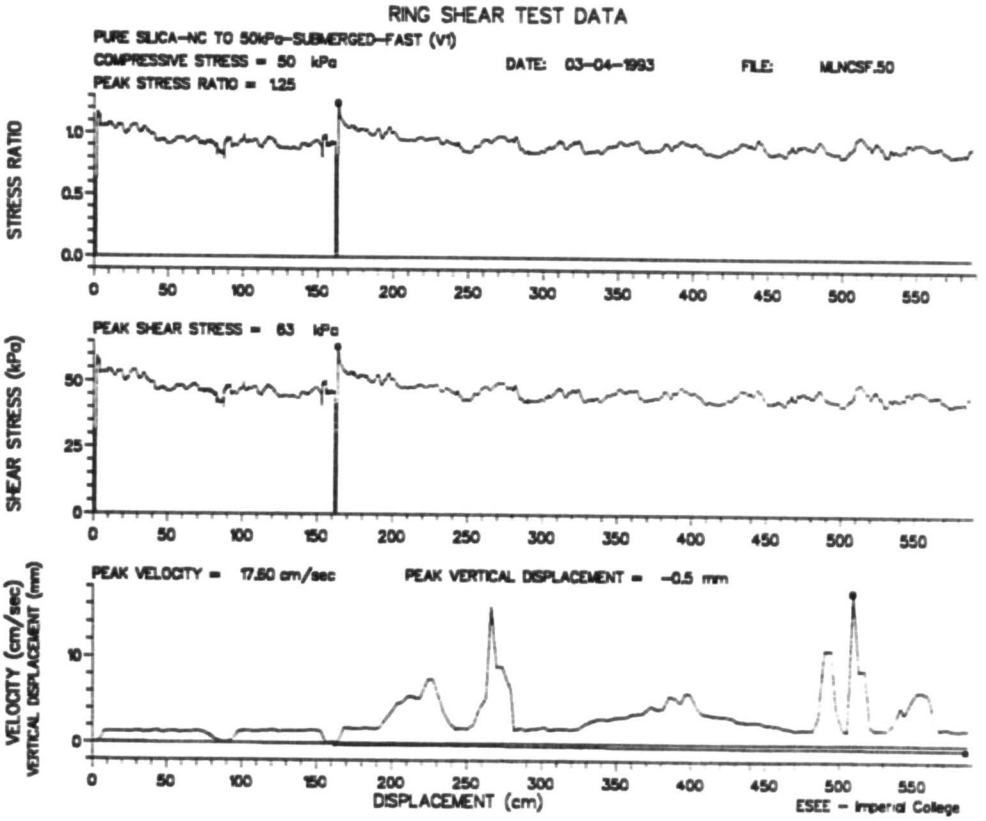


Fig. V.5.2 Fast shearing test on normally consolidated, submerged pure silica (50 kPa)

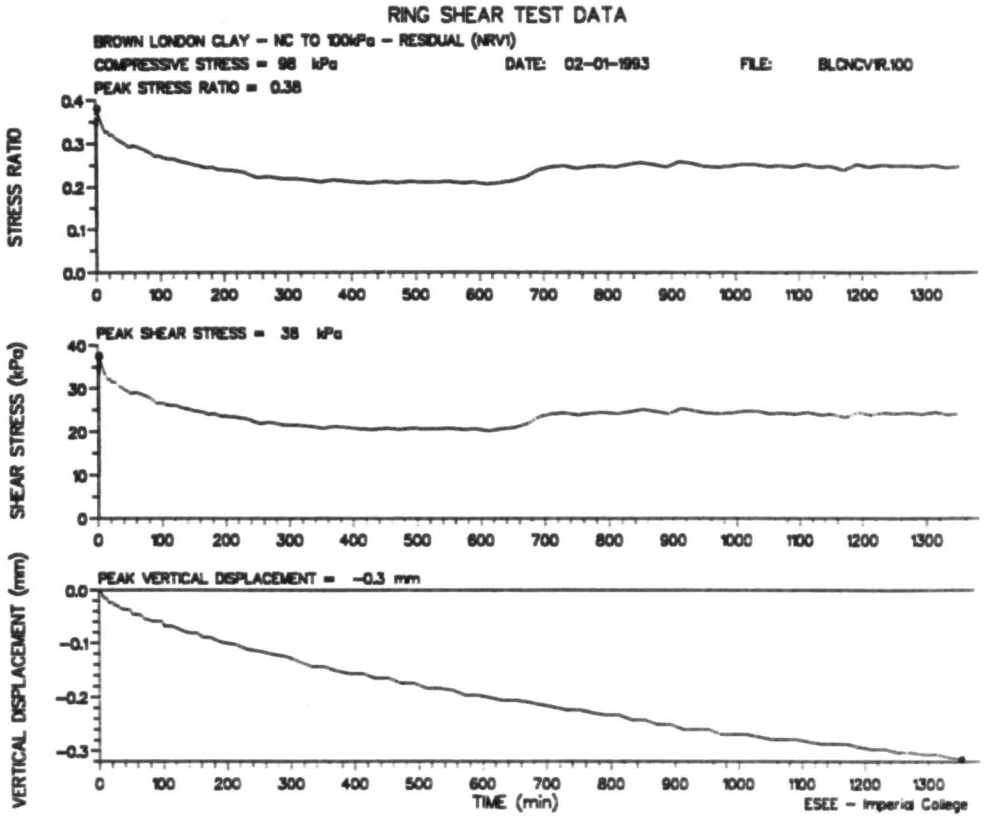


Fig. IV.2.1 Slow shearing test on normally consolidated, non-submerged brown London clay (100 kPa)

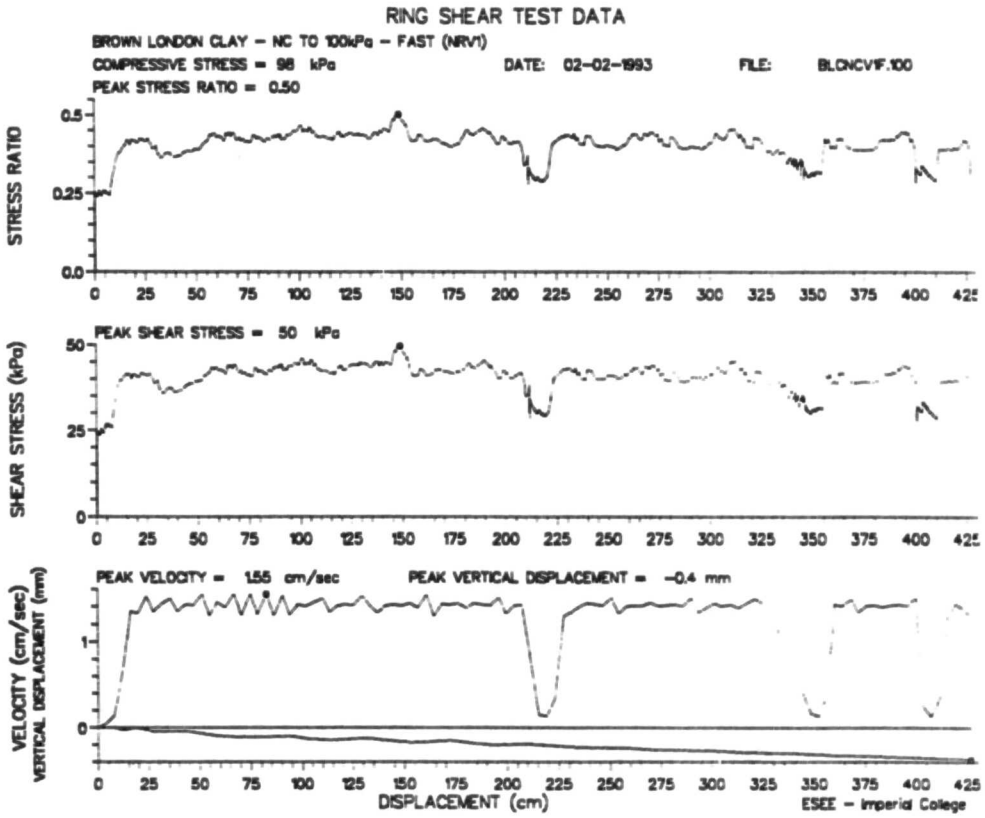


Fig. IV.2.2 Fast shearing test on normally consolidated, non-submerged brown London clay (100 kPa)

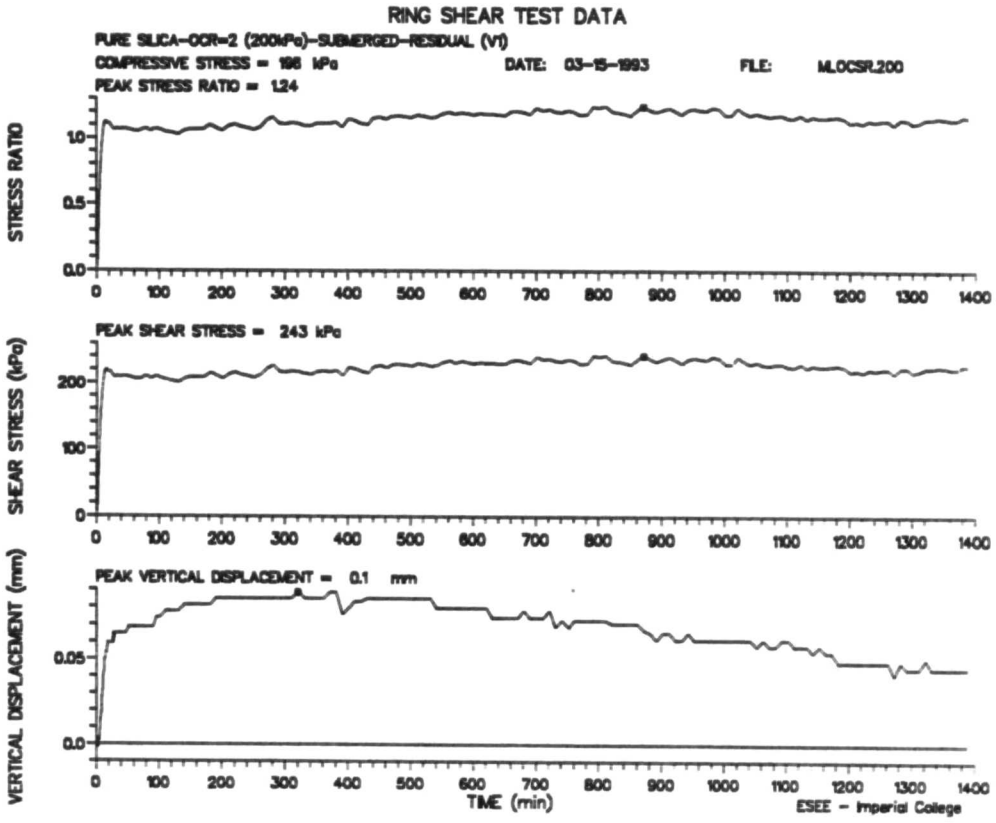


Fig. V.10.1 Slow shearing test on over-consolidated, submerged pure silica (200 kPa - OCR= 2)

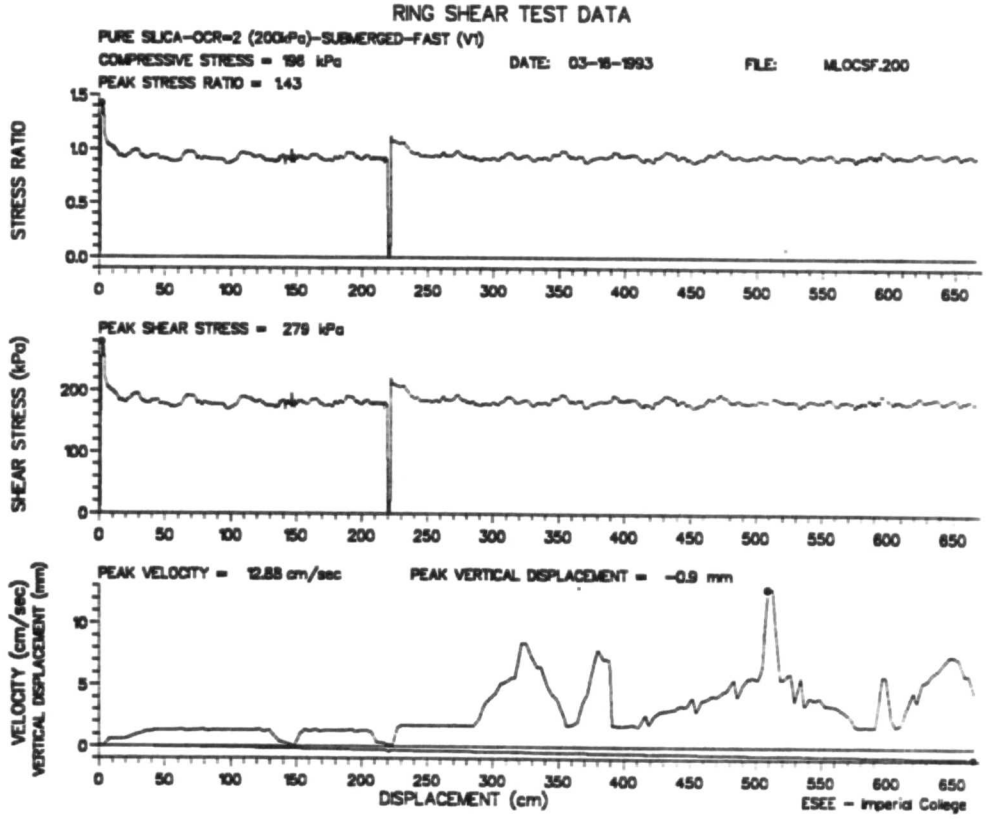


Fig. V.10.2 Fast shearing test on over-consolidated, submerged pure silica (200 kPa - OCR= 2)

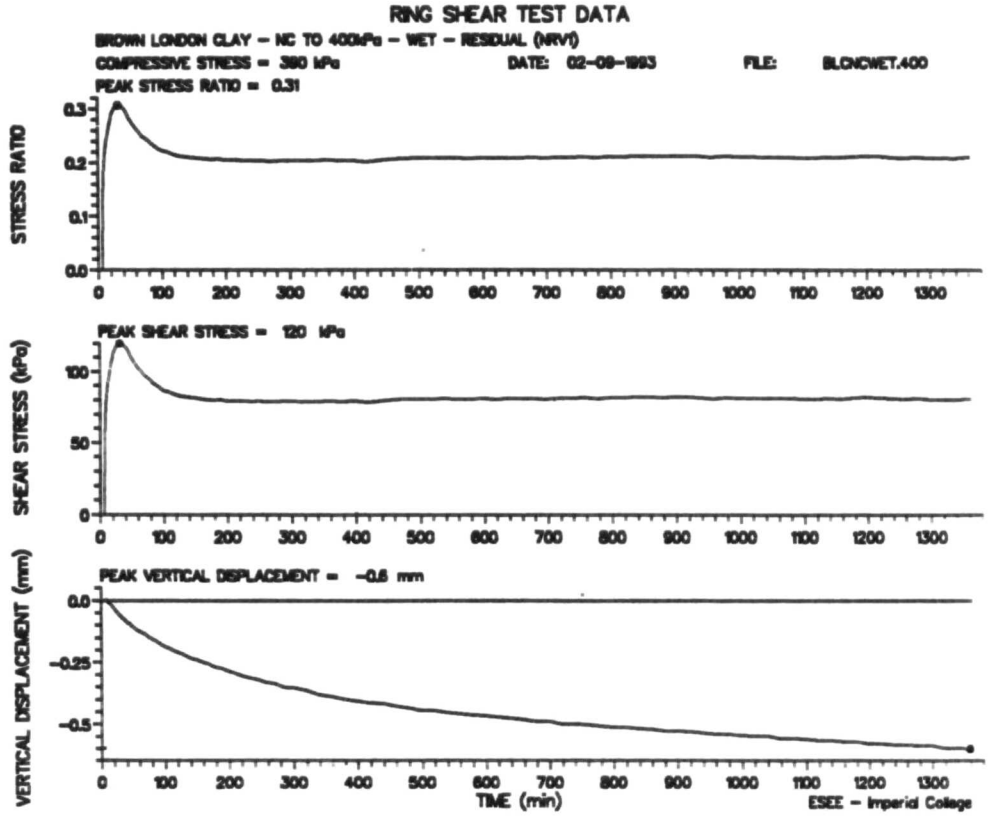


Fig. IV.7.1 Slow shearing test on normally consolidated, submerged brown London clay (400 kPa)

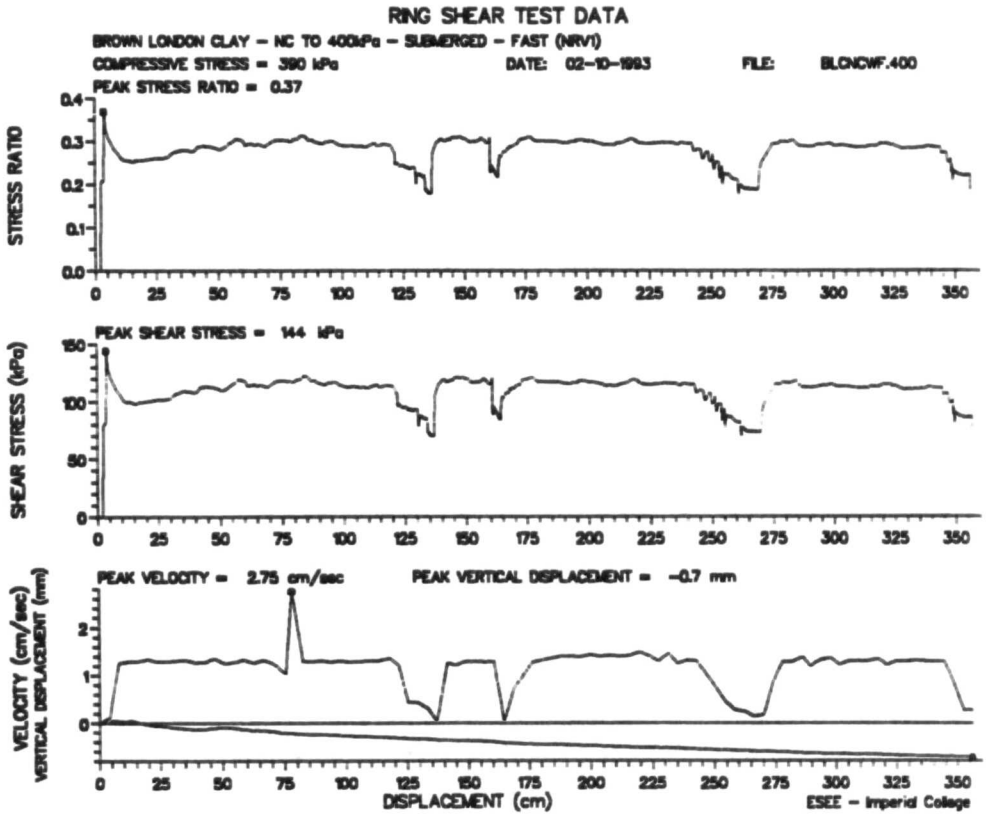


Fig. IV.7.2 Fast shearing test on normally consolidated, submerged brown London clay (400 kPa)

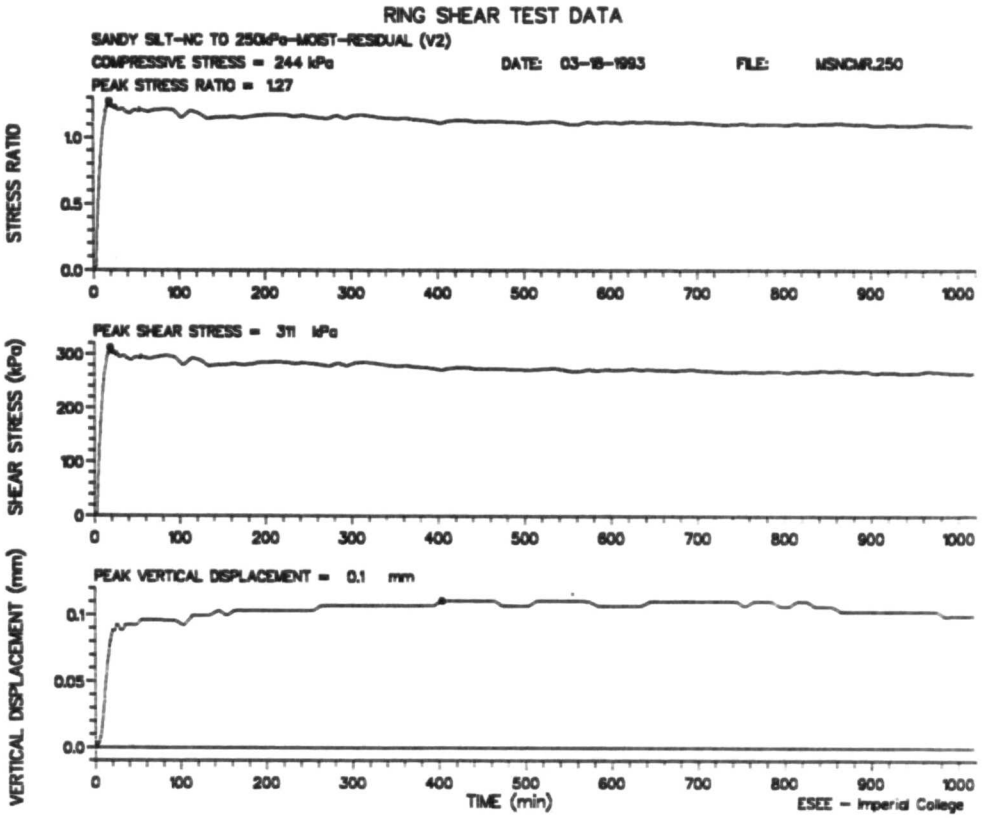


Fig. V.15.1 Slow shearing test on normally consolidated, non-submerged artificial sandy-silt (250 kPa)

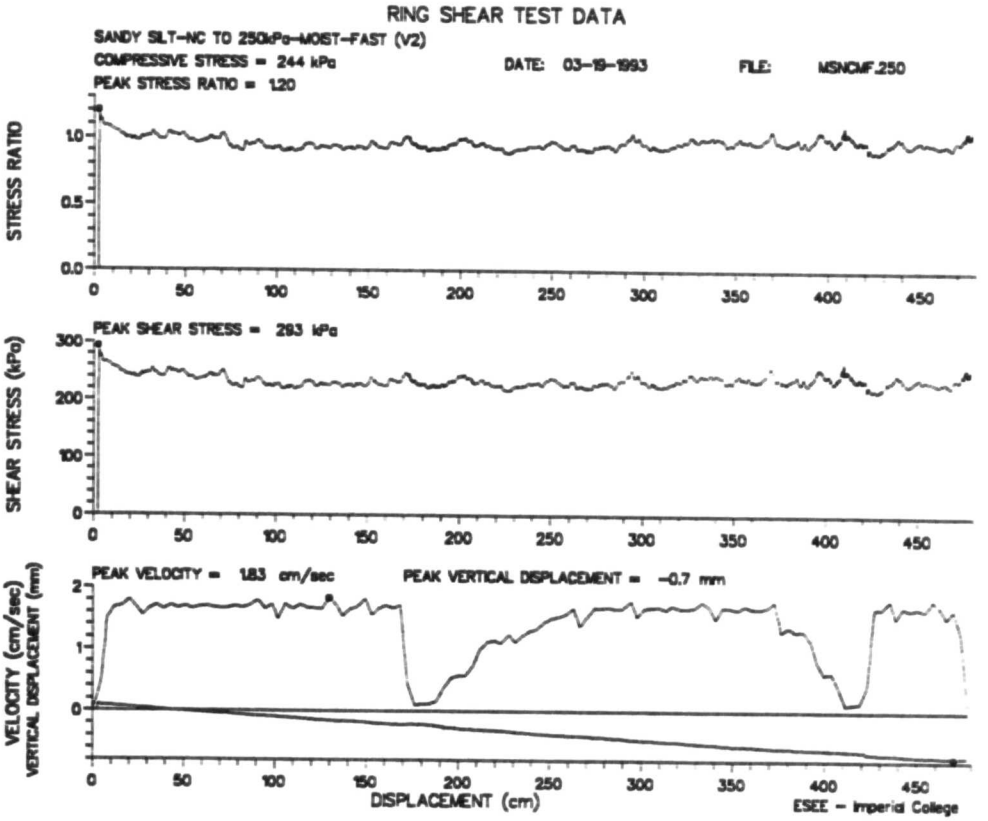


Fig. V.15.2 Fast shearing test on normally consolidated, non-submerged artificial sandy-silt (250 kPa)

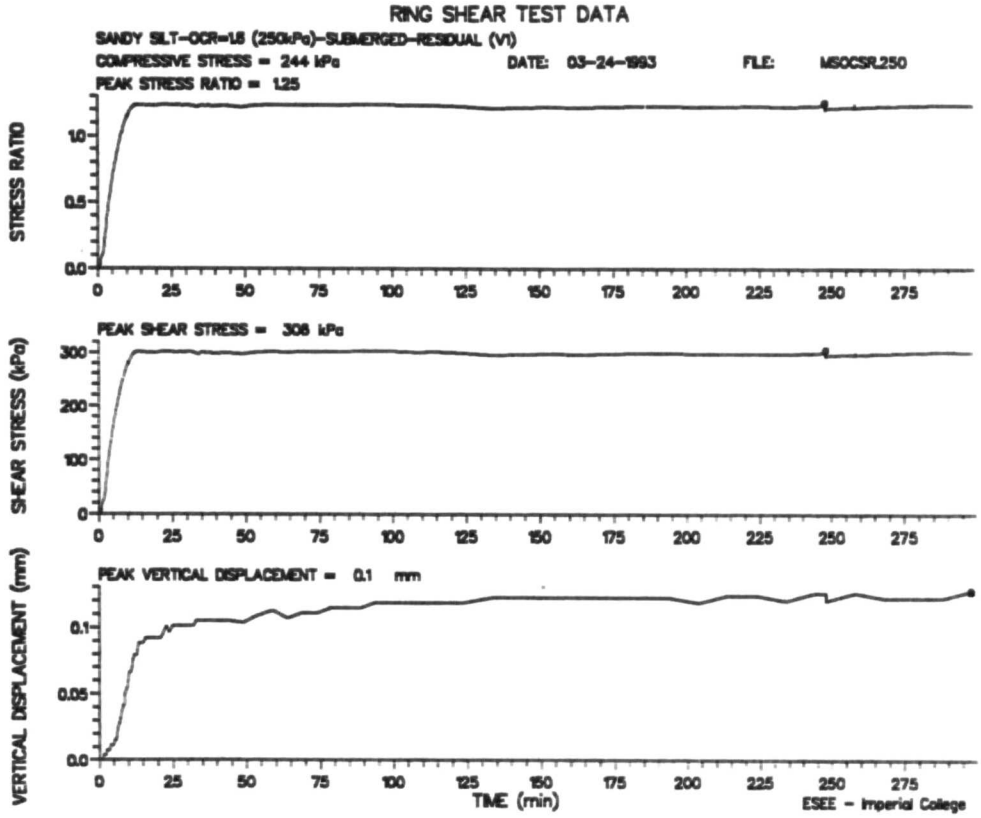


Fig. V.20.1 Slow shearing test on over-consolidated, submerged artificial sandy-silt (250 kPa - OCR= 1.6)

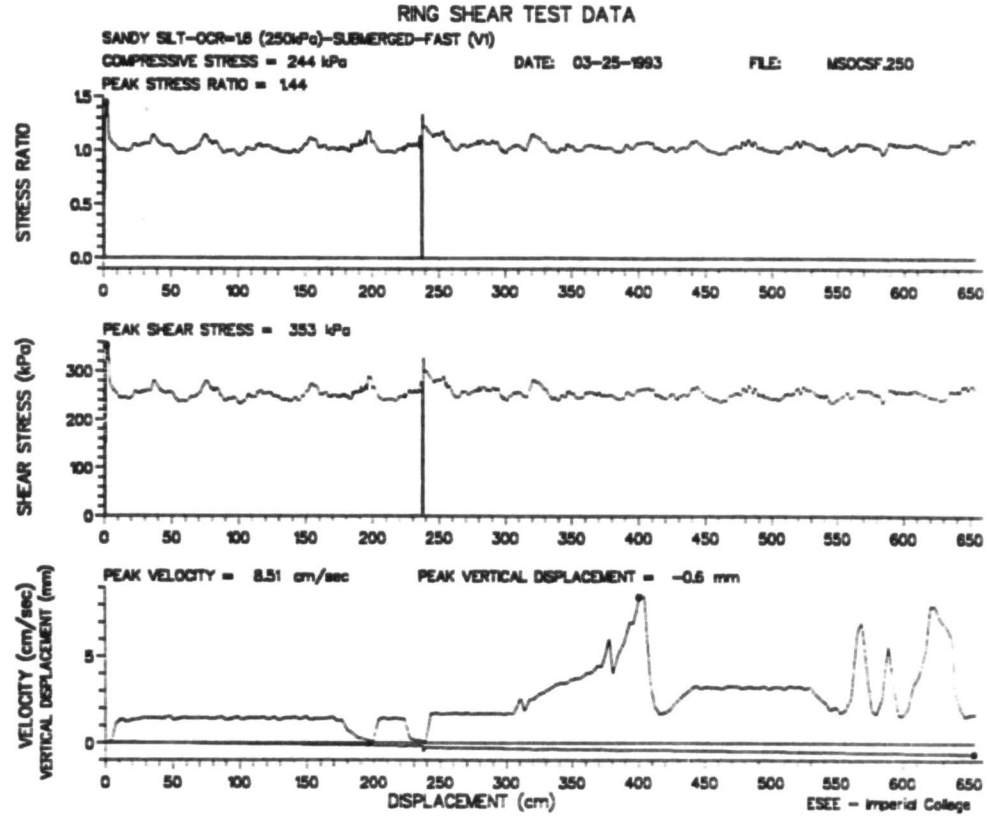


Fig. V.20.2 Fast shearing test on over-consolidated, submerged artificial sandy-silt (250 kPa - OCR= 1.6)

APPENDIX VI

DESCRIPTION OF ADDITIONAL TESTS ON PLASTIC SOILS

Tests to Investigate the Decrease of Fast Residual Strength Below Slow Residual

Brown London clay:

Test VI.1

σ_n= 100 kPa

OCR= 4

$\phi_{residual}$= 10.9°

$\phi_{threshold}$= 27.0°

(Figures VI.1.1-3)

The sample was consolidated to 400 kPa, unloaded to 100 kPa, and sheared at 0.09 mm/min. After 1.8 mm of shear displacement the measured strength reached its slow peak value of 37.6°, and the sample height had increased by 0.037 mm. Continued shearing led to further in sample height and decrease in the measured sample strength. After 107.4 mm of shearing the sample height had increased by 0.44 mm, and the measured strength had reached its slow residual value of 10.9°. This part of the test was terminated after 243 mm of shearing.

The rate of displacement was suddenly increased to 1.5 cm/sec and the measured strength reached immediately its threshold value of 27.0°. Further shearing was associated with a decrease in measured strength while the sample appeared to dilate. After 29.1 cm of shearing at the same rate of displacement the measured strength had decreased to 11.7° and the sample height had increased by 0.25 mm. Continued shearing at 1.5 cm/sec led to an increase in measured strength while the sample height appeared to be decreasing. After the sample had been sheared for 43.3 cm its height had dropped back to its level at the end of the slow shearing part of this test and the measured strength had increased to 15.3°. The strength of the sample fluctuated between 14.1° and 19.3° until the rate of displacement was decreased to

undulating (but declining due to sample loss) character of the sample height curve remained steady.

Subsequently, the sample was sheared at 0.09 mm/min and the pore pressures which had increased during the previous part of this test were allowed to dissipate. The measured strength continued to increase during this part of the test, and reached its slow residual value of 30.9° after 8.7 mm of shearing.

Tests on Completely Dry Samples

Brown London clay:

Normally Consolidated

Test VI.5

$\sigma_n = 100$

$\phi_{residual} = 37.4°$

$\phi_{threshold} = 43°$

(Figures VI.5.1-2)

The sample was sheared at 0.09 mm/min in order to avoid any structural changes which may have been induced by shearing at larger rates of displacement and also to form a basis for comparison between the remoulded and the dry samples. The measured strength reached its slow peak value of 42.9° after 9 mm of shearing while the sample height decreased by 0.54 mm. Subsequently, the measured strength began to decrease and reached its slow residual value of 37.4° after the sample had been sheared for 140.6 mm. Meanwhile the sample height had decreased by an additional 0.41 mm because of particle reorientation.

The rate of displacement was suddenly increased to 0.15 cm/sec and then, gradually, to 1.08 cm/sec before it was stabilised at 1.26 cm/sec. The measured strength reached immediately its threshold value of 43°, and, as shearing continued, decreased and reached the value of 30.3° after a shear displacement of 12 cm. Further shearing at the same rate of displacement led to an apparent increase in strength which had stabilised within a range between 34.0° and 37.4° before the rate of displacement was decreased to 0.85 cm/sec and then to 0.11 cm/sec. The decrease in the rate of displacement had no effect on the measured strength which remained within the same range. When the rate of displacement was increased again to 1.26

measured strength decreased before it stabilised within a range between 35.5° and 38.5°. After the sample had been sheared for 103.8 cm at 1.71 cm/sec, a periodic increase in the measured strength started to appear with wave-length of almost two full rotations and continually increasing magnitude. It was concluded that these increases of the measured strength were due to contamination of the rings from soil which had been progressively extruded through the gap during the previous two tests. Meanwhile, the rate of displacement varied from 1.71 cm/sec, to 0.2 cm/sec, to 0.08 cm/sec before the test was terminated after 590 of shearing, no effect on the strength of the sample was observed. The latter fluctuated between 35.5° and 42.6° during the final 140 cm of shearing in this test.

Test VI.12

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 37.8°

$\phi_{threshold}$= 42.9°

(Figures VI.12.1-2)

The sample was sheared at 0.09 mm/min, and the measured strength reached its slow residual value of 37.8° after 2.7 mm of shear displacement. This part of the test was terminated after the sample had been sheared for 115.0 mm.

The rate of displacement was increased suddenly to 1.70 cm/sec and the measured strength reached immediately its threshold value of 42.9°. The rate of displacement was decreased suddenly from 1.70 cm/sec to 0.073 cm/sec after the sample had been sheared for 294.6 cm, increased back to its former level after the sample had been sheared for 339.5 cm, and decreased again to 0.073 cm/sec just before the test was terminated after 560 cm of shearing. Meanwhile, the periodic increase in measured strength, encountered during the previously described test, was observed during this test as well, and was again attributed to internal ring friction caused by soil contamination. Besides these increases of the measured strength, the strength of the sample appeared to have stabilised within a range between 37.8° and 41.8° throughout the duration of this test and it was not affected by any variations of the rate of displacement.

monotonic and smooth sample height curve indicated the absence of an undulating shear surface and this was also reflected on the non-undulating shape of the stress ratio curve. The test was terminated after 360 cm of shearing and four cycles of increasing/decreasing the rate of displacement between 0.075 cm/sec and 1.31 cm/sec.

TEST IV.8

σ_n= 100 kPa

$\phi_{residual}$= 11.3°

$\phi_{threshold}$= 22.8°

(Figures IV.8.1-2)

Another sample was put together and tested under 100 and 200 kPa in order to complete this series of tests on normally consolidated, submerged brown London clay. This sample was sheared at 0.09 mm/min and it reached its slow peak strength of 24.7° after 2.28 mm of shearing while its height decreased by 0.115 mm. Continued shearing at the same rate led to a decrease in strength until the latter reached its slow residual value of 11.3° after a displacement of 92.5 mm. As the sample reached its residual state, its height decreased by 0.235 mm and this should be attributed to particle orientation and sample loss through the gap of the rings. The additional 0.015 mm decrease in sample height from the point at which the sample had reached its residual state until the end of this slow part of the test, after 112.1 mm of shearing at 0.09 mm/min, should be attributed solely to soil lost through the gap.

The rate of displacement was increased to 0.35 cm/sec for the following 6.87 cm of shearing and the strength increased immediately to 19.8° before it stabilised at 18.5°. The increase in the rate of displacement was associated with an apparent contraction of the sample or the material which was involved in shearing. After 6.87 cm of shearing, and while the rate of displacement was increased further to 1.71 cm/sec, the sample had contracted by 0.05 mm and its strength increased further to 22.8°. Continued shearing at 1.71 cm/sec led to dilation of the sample and an associated decrease in measured strength. After 31.6 cm of shearing the sample had dilated by 0.21 mm and its strength had dropped to 13.3°. Shearing at a constant 1.71 cm/sec was continued for 180 cm at which point the rate of displacement was decreased to 0.12 cm/sec. It was observed that an undulating shear surface had

$\phi_{threshold}$= 55.6°

(Figures V.12.1-2)

After 1.6 mm of shearing at 0.09 mm/min the sample height had decreased by 0.075 mm and the measured strength had reached the level of 37.5°. After 9.5 mm of shearing the sample height had rebounded to 0.02 mm above its initial level and the measured strength had reached its slow peak value of 51.8°. The measured strength reached its slow residual value of 50.6° after 45.6 mm of shearing. This part of the test was terminated after a displacement of 108.3 mm.

The rate of displacement was increased suddenly to 1.70 cm/sec and the measured strength reached its threshold value of 55.6° at zero associated displacement. Continued shearing at the same rate of displacement led to a decrease in strength while the sample appeared to dilate. After 5 cm of shearing the sample height had increased by 0.135 mm and the measured strength had dropped to 50.6°. The strength of the sample stabilised between 44.9° and 48.1° after 60 cm of cumulative displacement and it remained within this range until the test was terminated after 425 cm of shearing. Before the test was terminated, the rate of displacement was changed from 1.70 cm/sec to 0.11 cm/sec and back to 1.70 cm/sec, suddenly and gradually, but no effect on the strength of the sample was observed.

Test V.13

σ_n= 100 kPa

$\phi_{residual}$= 45.0°

$\phi_{threshold}$= 51.3°

(Figures V.13.1-2)

After 2.09 mm of shearing at 0.09 mm/sec the sample height had increased by 0.06 mm while the measured strength had reached its slow peak value of 51.1°. The sample reached its slow residual strength of 45.0° after 113.1 mm of shearing while its height had increased by 0.102 mm. This slow shearing part of the test was terminated after a displacement of 133.0 mm.

The rate of displacement was increased suddenly to 0.2 cm/sc and the measured strength reached immediately its threshold value of 51.3°. After 7.1 cm of shearing at the same rate of displacement, the strength had decreased to 45.0° while the sample height had increased by 0.105 mm. The rate of displacement was increased again to 1.69 cm/sec and the strength of the sample continued to decrease

The sample was sheared at 0.09 mm/min and reached its slow peak strength of 19.3° after only 0.8 mm of shearing. From the shape of the stress ratio/displacement curve it could be argued that the residual structure had already been established, however, it was concluded that the sample reached its slow residual strength of 13.3° after 117.8 mm of shearing while the height of the sample had decreased by 0.2 mm.

The rate of displacement was immediately increased to 1.2 cm/sec and the sample reached its threshold strength of 24.6° at zero associated shear displacement. The rate of displacement was continuously increased until it reached 7.8 cm/sec after 19.8 cm of shear displacement. At that point the shear strength of the sample had reached its fast peak value of 26.6° and the height of the sample had increased by 0.38 mm. The rate of displacement was suddenly decreased to 2.0 cm/sec and the measured strength dropped to its slow residual value after 46 cm of cumulative shearing. The measured strength continued to decrease with further shearing and oscillated between 4.4° and 11.3° after 90 cm of shearing. From the sample height curve it can be concluded that ring contamination was mainly responsible for the undulating shape of the stress ratio/displacement curve. As it was expected at this stage of the experiment a significant amount of soil must have been lost through the gap contaminating the rings and partially influencing the results. Any change of the rate of shearing between 2.0 and 10.0 cm/sec did not influence the measured soil strength after the latter had dropped below its slow residual level. The test was terminated after 480 cm of shear displacement at which point the sample height had decreased by 0.8 mm.

Tests on Cowden Till:

Normally Consolidated, Non-Submerged

Test IV.13

σ_n= 50 kPa

$\phi_{residual}$= 35.4°

$\phi_{threshold}$= 47.3°

(Figures IV.13.1-2)

The sample was sheared at 0.09 mm/min and after 7.22 mm of shear displacement it reached its slow peak strength of 36.5° while its height decreased by

OCR= 2

$\phi_{residual}$= 46.9°

$\phi_{threshold}$= 50.4°

(Figures V.19.1-2)

The sample was sheared at 0.09 mm/min and the measured strength reached its slow peak value of 49.2° after 1.9 mm of shearing while the sample height had increased by 0.065 mm. The latter had increased by a further 0.01 mm when the measured strength reached its slow residual value of 46.9° after 70.3 cm of shearing. This part of the test was terminated after the sample had been sheared for 115.9 mm.

The rate of displacement was suddenly increased to 1.5 cm/sec and the measured strength reached immediately its threshold value of 50.4°. Further shearing was associated with a decrease in measured strength while the sample appeared to dilate. After 29.2 cm of shearing, the sample height had increased by 0.075 mm, and the measured strength had stabilised within a range between 43.7° and 45.9°. As it was the case for the previously described test, some internal ring friction must have been responsible for the magnitude of the variation of the measured strength. After 245.5 cm of shearing the rate of displacement was suddenly increased to 3.15 cm/sec and the measured strength reached immediately the value of 49.3°. The rate of displacement was changed from 3.15 cm/sec, to 1.83 cm/sec, to 5.3 cm/sec and to 10 cm/sec, suddenly and gradually, but no effect on the measured strength was observed. The test was terminated after 650 cm of shearing.

Test V.20

σ_n= 250 kPa

OCR= 1.6

$\phi_{residual}$= 49.0°

$\phi_{threshold}$= 55.2°

(Figures V.20.1-2)

The sample reached its slow peak strength of 51.1° after 1.28 cm of shearing while its height had increased by 0.08 mm and its slow residual strength of 49.0° after 23.8 cm of shearing.

The rate of displacement was suddenly increased to 1.44 cm/sec and the measured strength reached immediately its threshold value of 55.2°. Further shearing at the same rate of displacement was associated with an increase in sample height by

was increased steadily from 0.023 cm/sec to 1.65 cm/sec and then decreased accordingly to 0.08 cm/sec. At 0.023 cm/sec the measured strength of this sample was 32.1°, at 0.83 cm/sec it was 35.4°, at 1.1 cm/sec it was 36.0°, at 1.26 cm/sec it was 36.3° and at 1.4 cm/sec and above it remained constant at 37.3°. Upon decreasing the rate of displacement the measured strength of the sample was not affected until the rate of displacement had dropped below 0.49 cm/sec. This test was terminated after 418 cm of cumulative shearing.

Normally Consolidated, Submerged:

Test IV.17

σ_n= 50 kPa

$\phi_{residual}$= 31.8°

$\phi_{threshold}$= 45.3°

(Figures IV.17.1-2)

The sample was sheared at 0.09 mm/min and reached its slow residual strength of 31.8° after 192.9 mm of displacement. This was the first time that this particular sample was sheared and it was expected that the residual state would involve shearing at a large displacement. The 0.282 mm of decrease in sample height as the sample approached its residual state should be attributed to particle reorientation and sample loss through the gap. Any further decrease in sample height should be solely attributed to sample loss. The increase in measured strength toward the end of this part of the test should be attributed to either ring contamination or formation of an undulating shear surface. Shearing at 0.09 mm/min was terminated after 2,650 mm.

The rate of displacement was increased instantly to 1.29 cm/sec and the measured strength reached its threshold value of 45.3° at zero associated displacement. While shearing continued at the same rate of displacement the sample appeared to be dilating and its measured strength decreased. After 26.9 cm of shearing, the sample height had increased by 1.08 mm and the measured strength had dropped to 32.0°. After the sample had been sheared for 42.3 cm, its height had increased to 1.8 mm above its level at the end of the slow shearing part of this test and its measured strength had decreased to 28.3°. Further shearing was associated with a decrease in sample height as the material which was involved in shearing might have been contracting. This decrease in sample height was accompanied by an

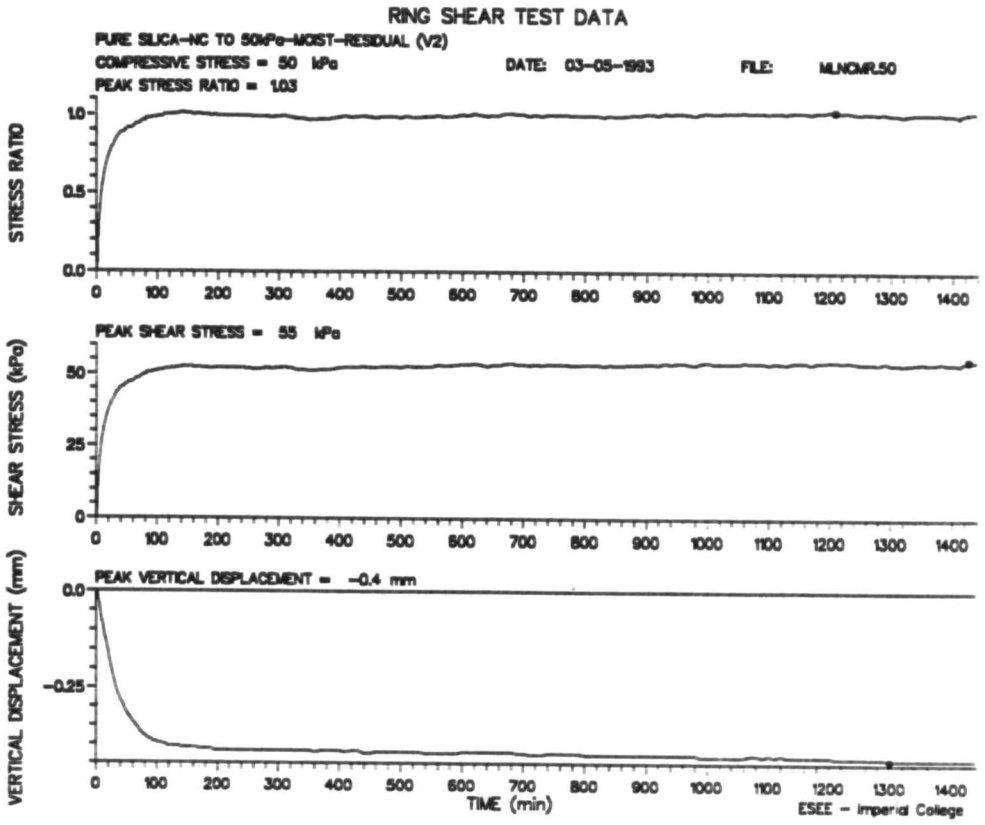


Fig. V.1.1 Slow shearing test on normally consolidated, non-submerged pure silica (50 kPa)

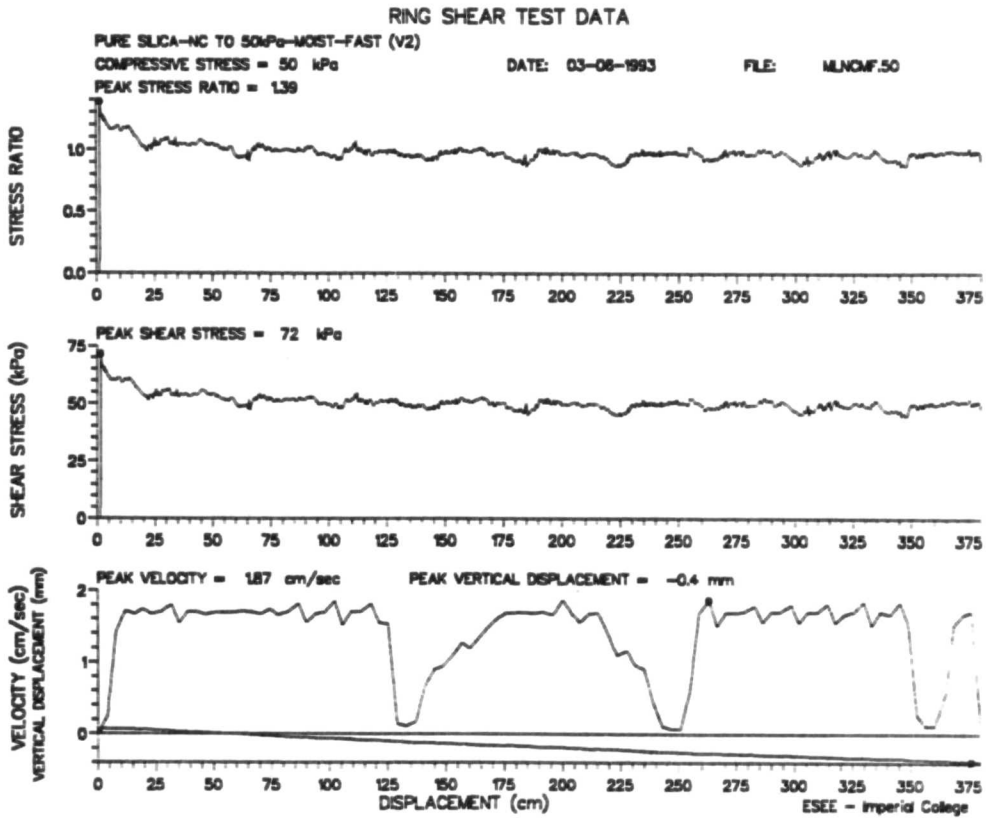


Fig. V.1.2 Fast shearing test on normally consolidated, non-submerged pure silica (50 kPa)

its slow residual value. The shape of the sample height curve suggested the existence of an irregular shear surface which was also reflected on the measured shear strength. After 82.5 cm of shearing the rate of displacement was increased to 1.68 cm/sec and it was kept at that level for the next 214.7 cm of shearing. The strength fluctuated between 33.8° and 43.5° probably because of the irregularities of the shear surface. After 300 cm of cumulative shearing the rate of displacement was decreased gradually, at 1.68 cm/sec the measured strength was 40.9°, at 0.5 cm/sec it was 38.4°, at 0.23 cm/sec it was 36.8° and at 0.096 cm/sec it was 35.9°. This fast shearing part of the test was terminated after 480 cm of cumulative shearing.

Test IV.22

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 31.1°

$\phi_{threshold}$= 34.7°

(Figures IV.22.1-2)

The measured strength reached its slow residual value of 31.1° after 101.7 mm of shearing at 0.09 mm/min at which point the sample height had decreased by 0.224 mm. The decrease in sample height during the early stages of this slow shearing part of the test suggested that the structural relaxation associated with the previous test must have been completed. This slow shearing part of the test was terminated after 158.7 mm of shearing at which point the sample height had decreased by a further 0.044 mm.

The rate of displacement was suddenly increased to 1.7 cm/sec and the measured strength reached immediately its threshold value of 34.7°. Further shearing at the same rate of displacement led to a decrease in measured strength while the sample appeared to be dilating. After 10.5 cm of shearing the sample height had increased by 0.26 mm, and the strength had decreased to 31.1°, while, after 14.2 cm of shearing the sample height had increased by a further 0.04 mm and the measured strength had decreased to 25.9°. However, as shearing continued, the sample appeared to be contracting and its measured strength increased. After 52 cm of shearing the measured strength had increased to 33.3°. Shearing at 1.7 cm/sec continued for 130 cm of cumulative displacement while the fast residual strength of the sample fluctuated between 35.5° and 37.2° and an undulating shear surface with

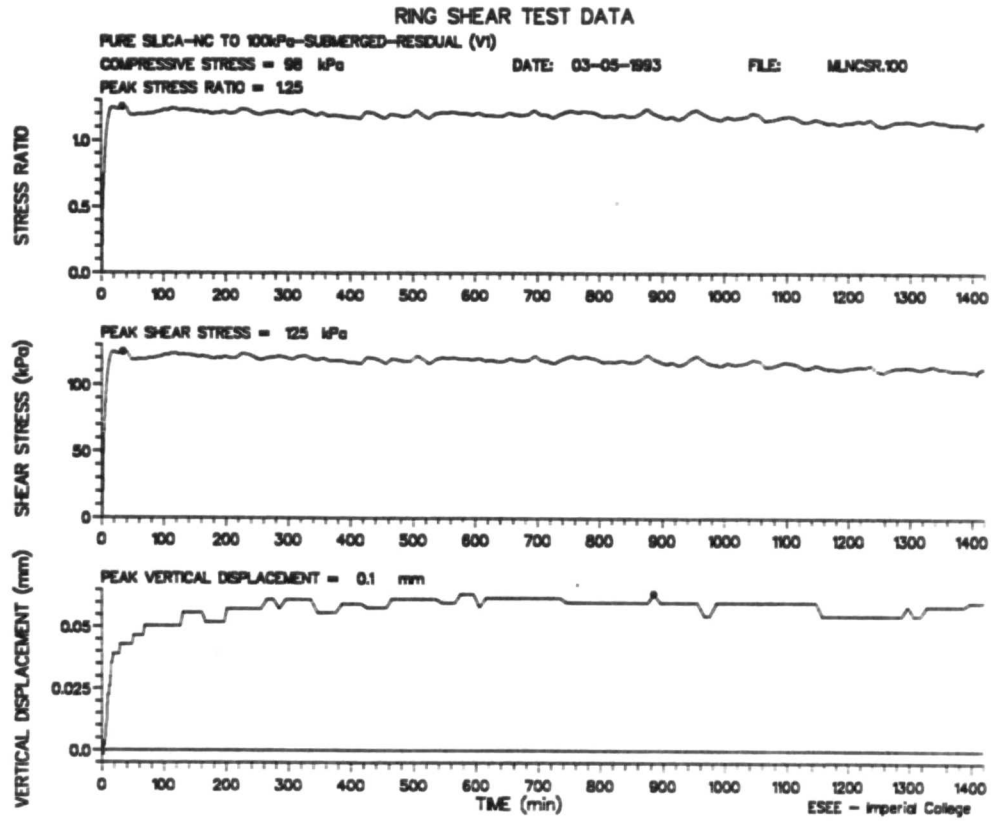


Fig. V.6.1 Slow shearing test on normally consolidated, submerged pure silica (100 kPa)

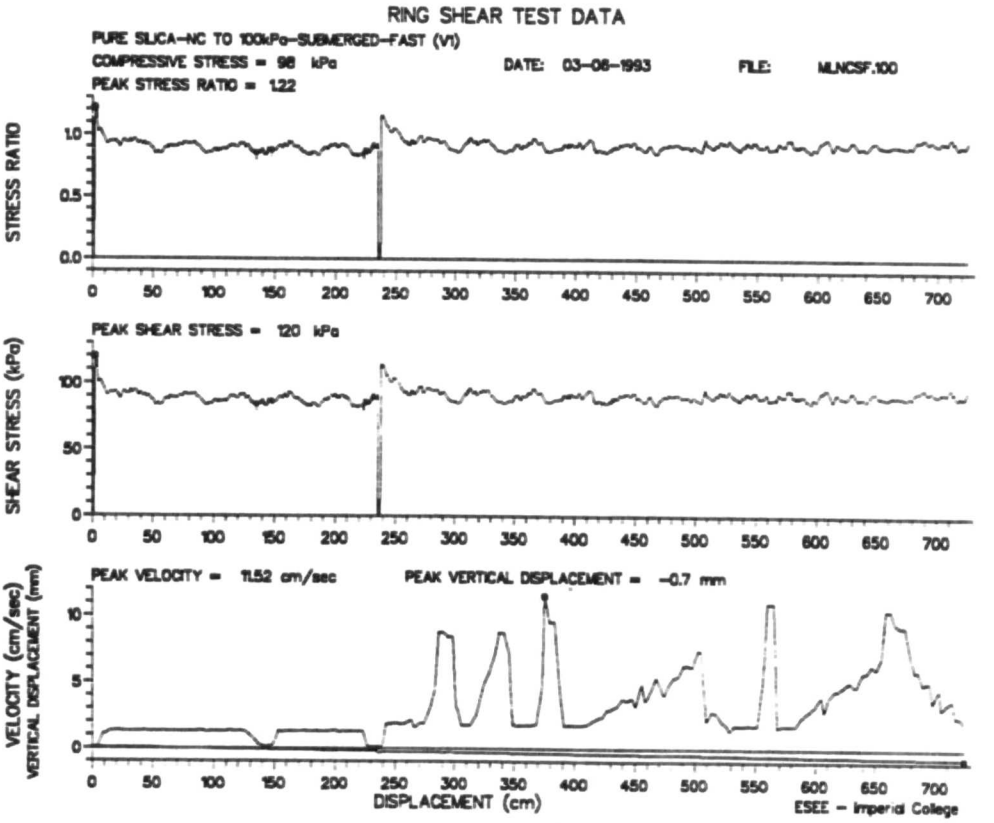


Fig. V.6.2 Fast shearing test on normally consolidated, submerged pure silica (100 kPa)

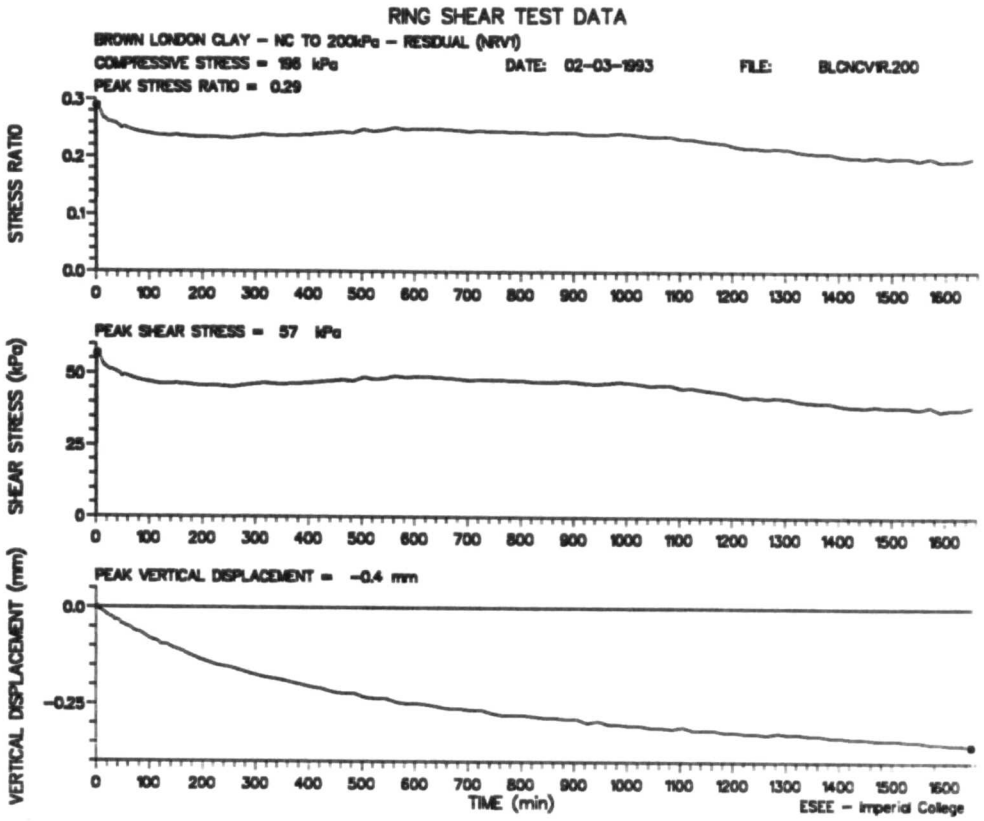


Fig. IV.3.1 Slow shearing test on normally consolidated, non-submerged brown London clay (200 kPa)

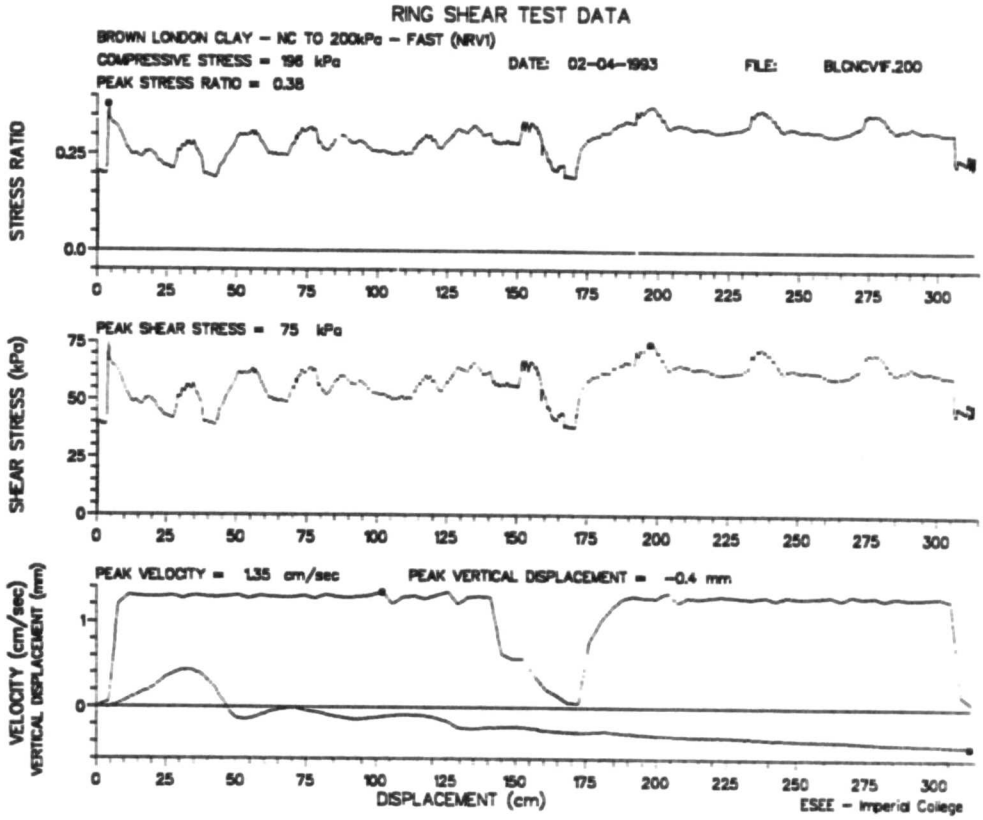


Fig. IV.3.2 Fast shearing test on normally consolidated, non-submerged brown London clay (200 kPa)

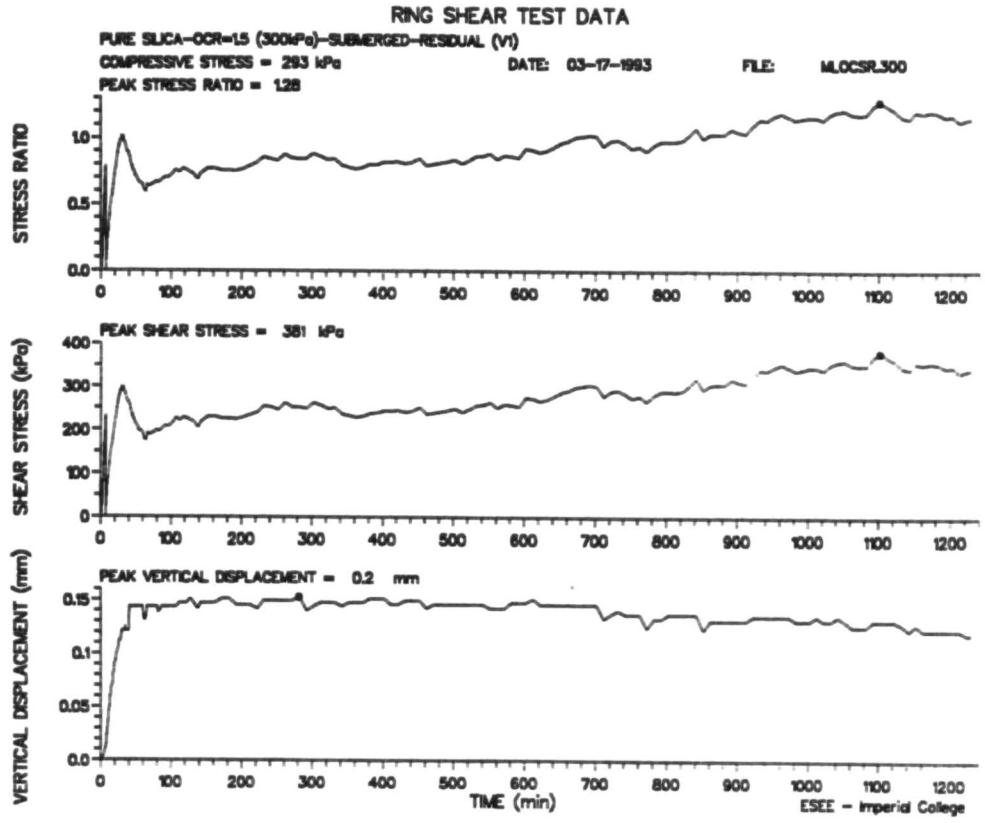


Fig. V.11.1 Slow shearing test on over-consolidated, submerged pure silica (300 kPa - OCR= 1.5)

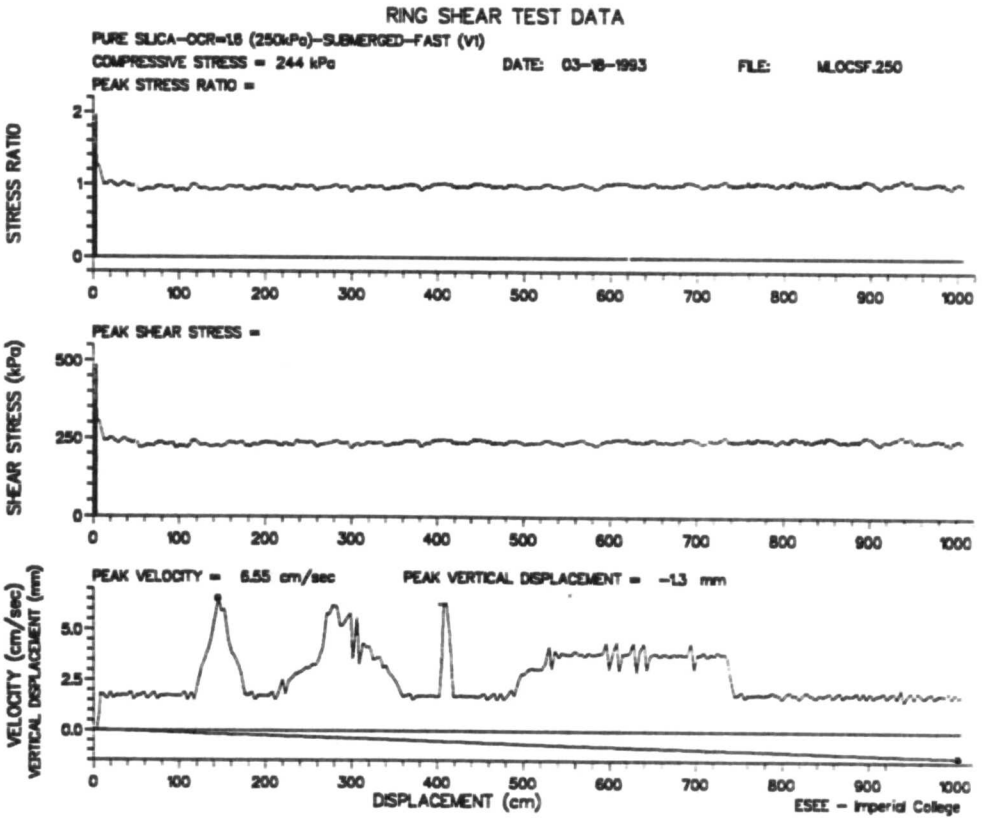


Fig. V.11.2 Fast shearing test on over-consolidated, submerged pure silica (300 kPa - OCR= 1.5)

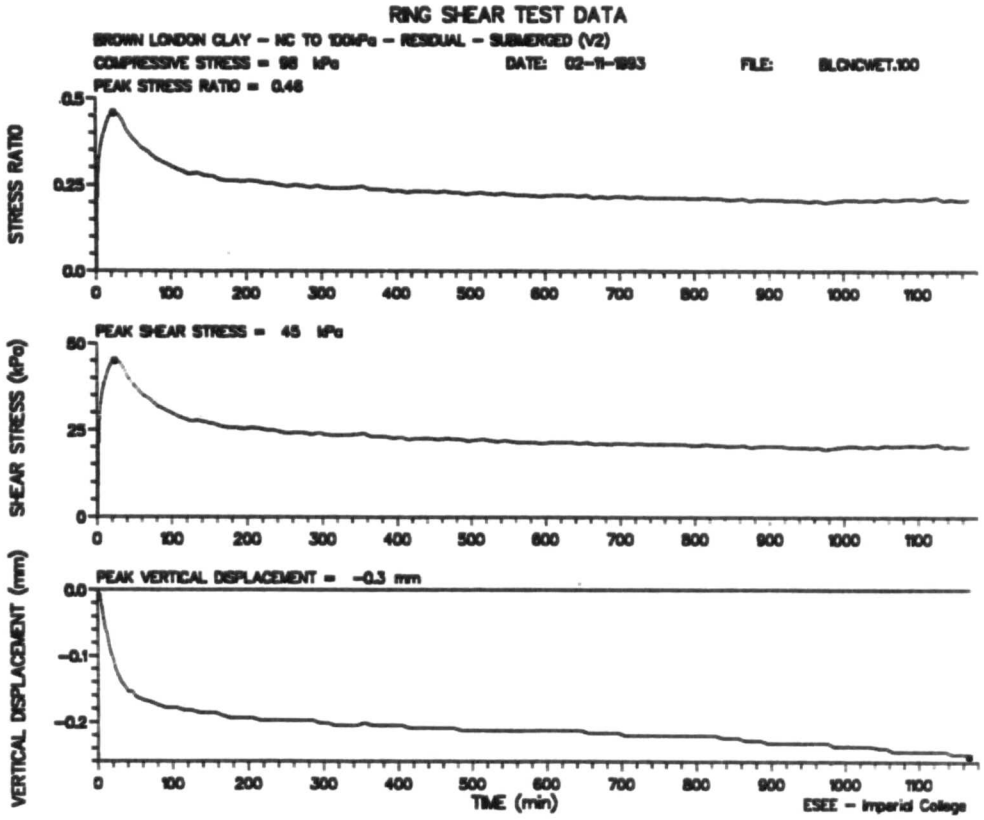


Fig. IV.8.1 Slow shearing test on normally consolidated, submerged brown London clay (100 kPa)

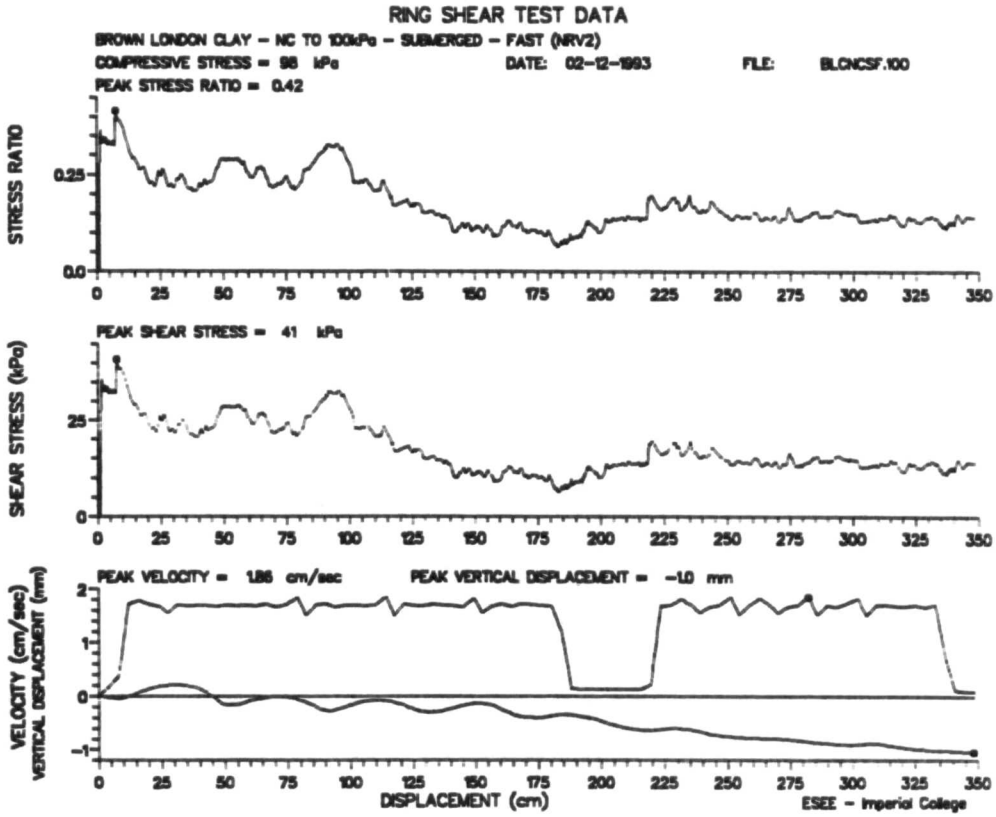


Fig. IV.8.2 Fast shearing test on normally consolidated, submerged brown London clay (100 kPa)

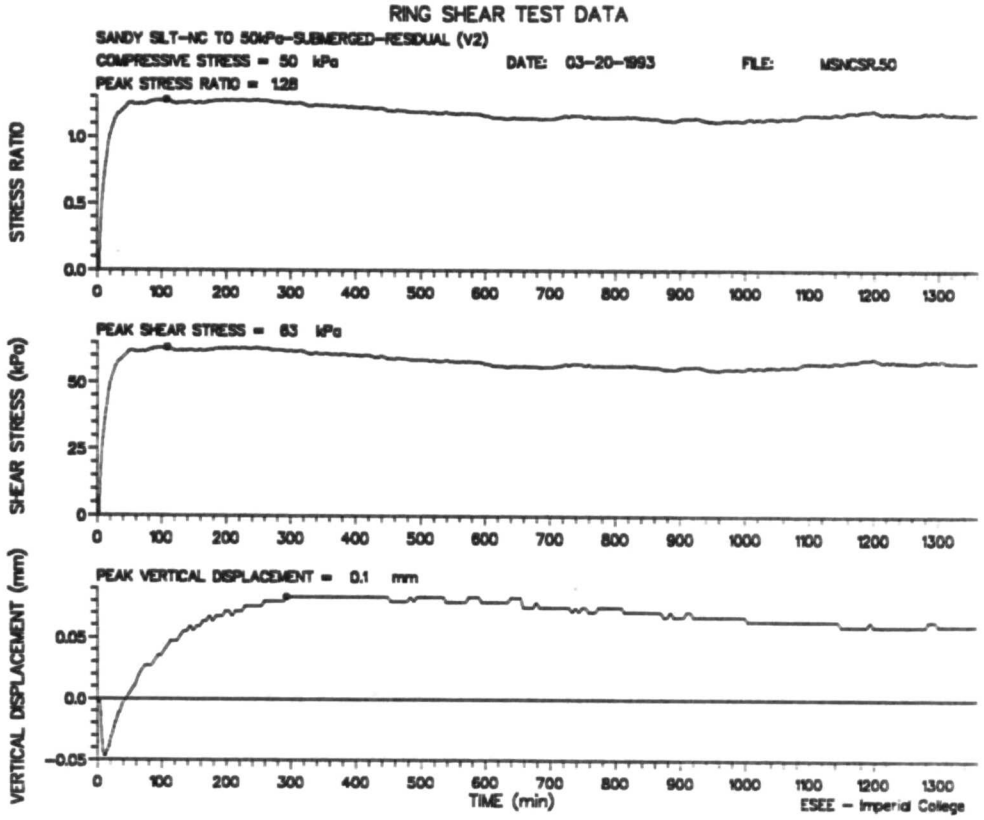


Fig. V.16.1 Slow shearing test on normally consolidated, submerged artificial sandy-silt (50 kPa)

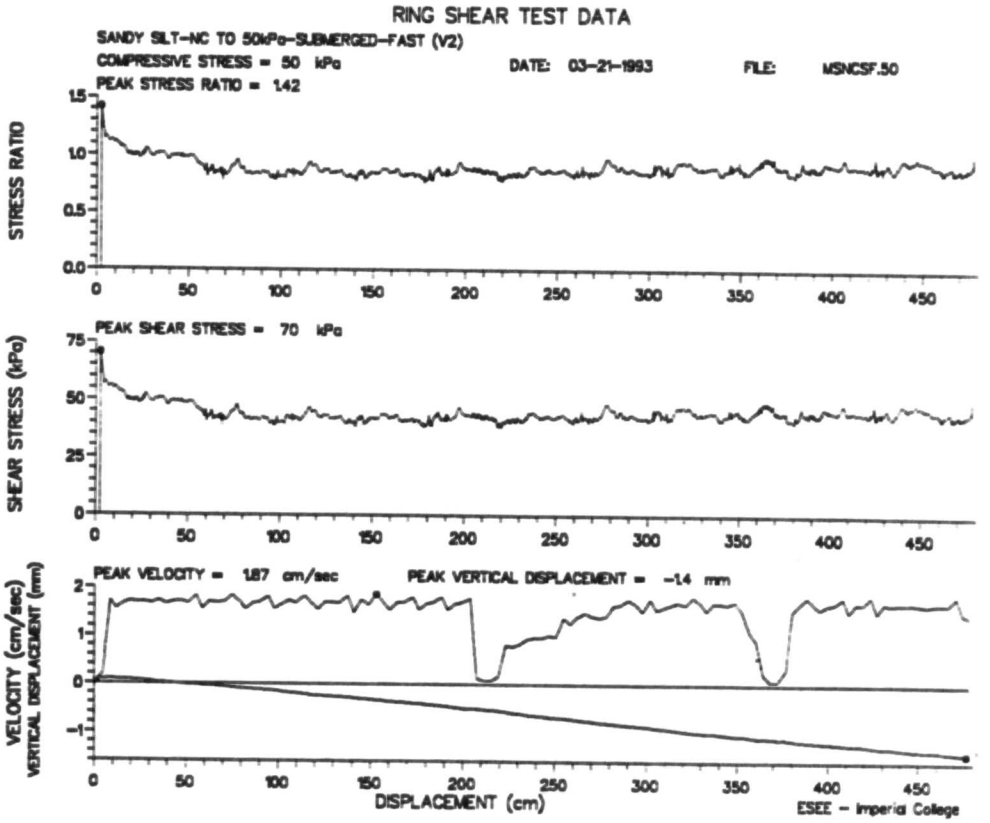


Fig. V.16.2 Fast shearing test on normally consolidated, submerged artificial sandy-silt (50 kPa)

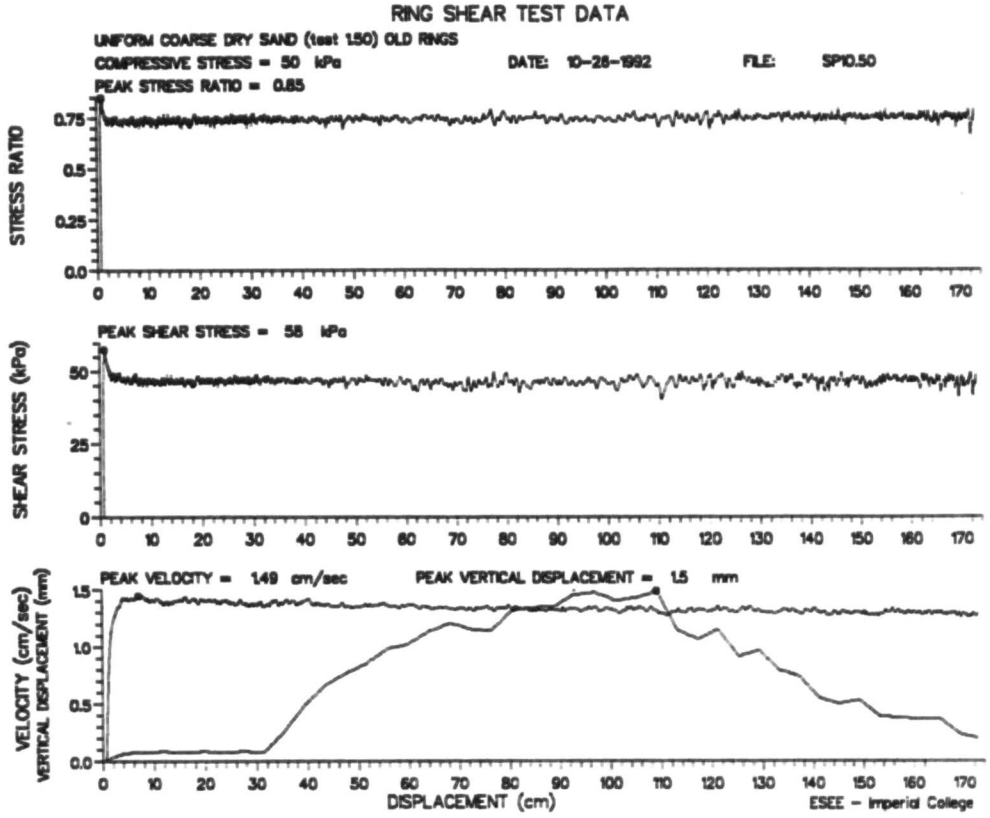


Fig. V.21.1 Fast shearing test on normally consolidated, dry medium-coarse sand (50 kPa)

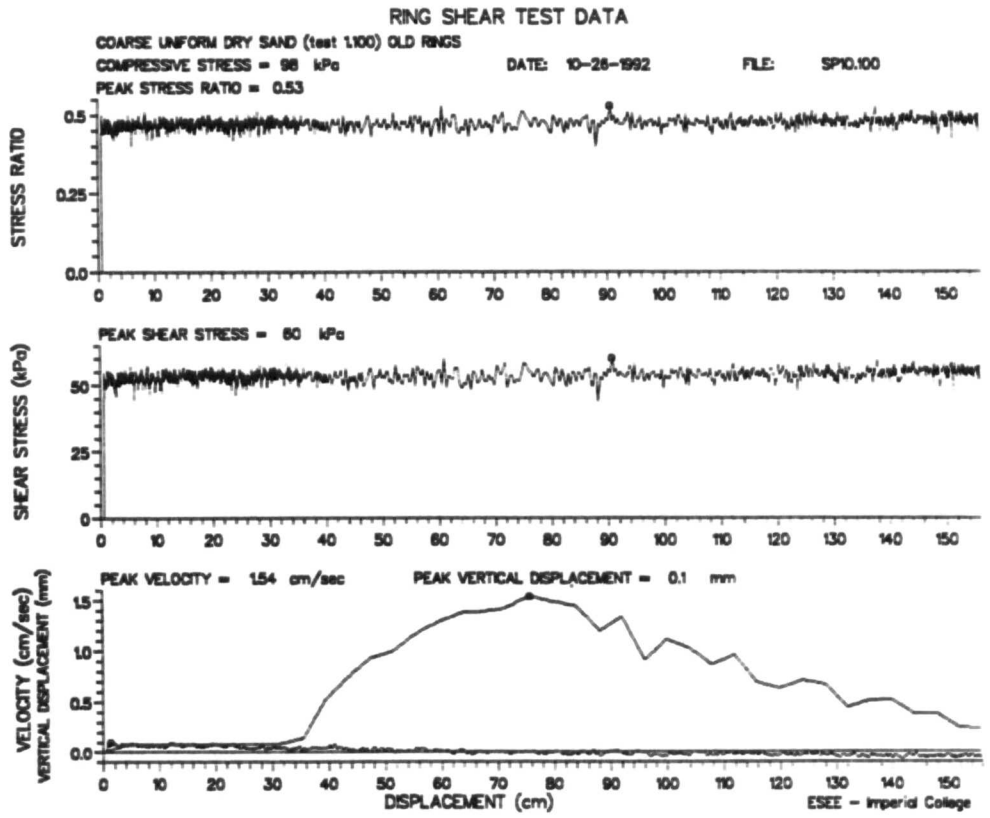


Fig. V.22.1 Fast shearing test on normally consolidated, dry medium-coarse sand (100 kPa)

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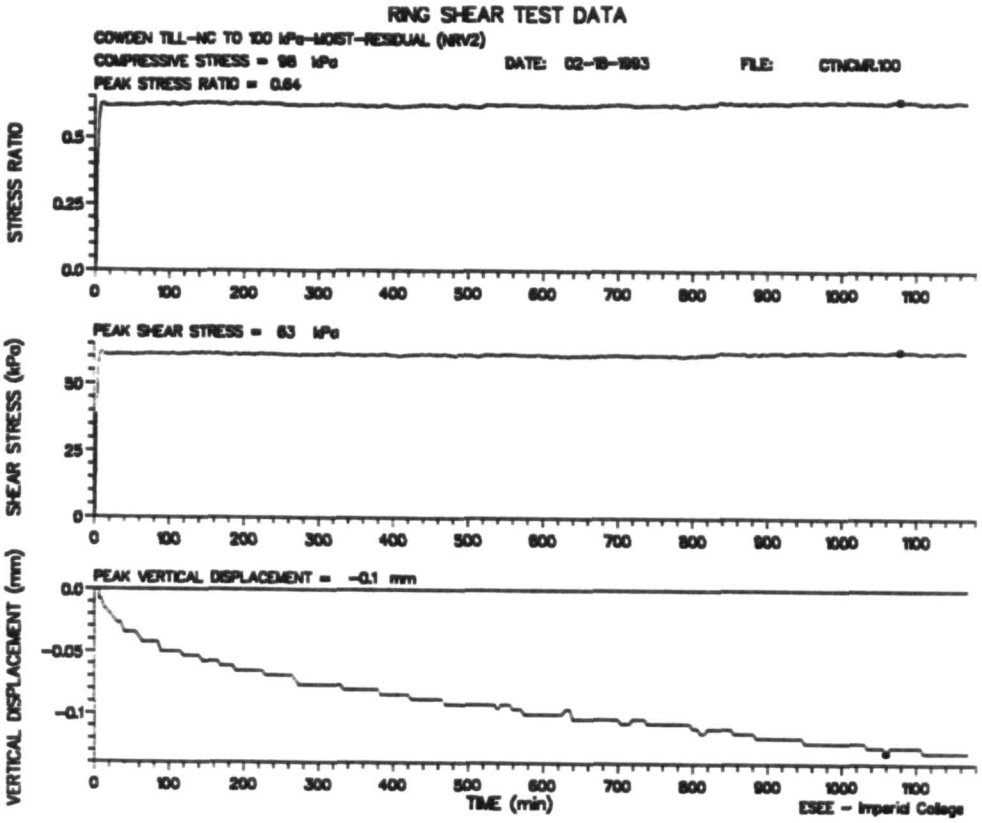


Fig. IV.14.1 Slow shearing test on normally consolidated, non-submerged Cowden till (100 kPa)

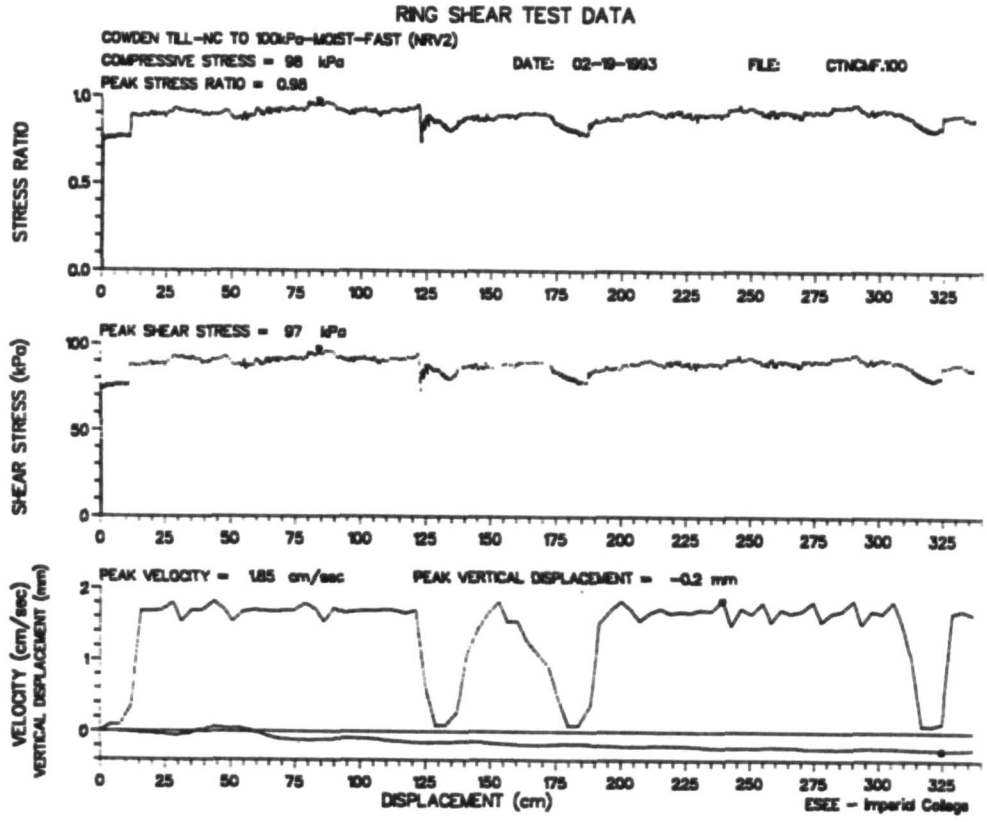


Fig. IV.14.2 Fast shearing test on normally consolidated, non-submerged Cowden till (100 kPa)

0.12 cm/sec -after the sample had been sheared for 188.4 cm- and the strength dropped to 12.5°. The rate of displacement was again increased to 1.25 cm/sec for the next 111.6 cm and the strength of the sample fluctuated between 15.6° and 21.9°. After the sample had been sheared for 300 cm the rate of displacement was decreased to 0.4 cm/sec and then to 0.12 cm/sec and its strength decreased to 13.5° and 12.7°, respectively. After 311.6 cm of cumulative shearing the rate of displacement was increased to 1.72 cm/sec while the strength of the sample fluctuated between 21.0° and 24.5°. Further gradual increase of the rate of displacement to 7.38 cm/sec had no effect on the strength of the sample. After the sample had been sheared for 387.4 cm the rate of displacement was decreased to 1.72 cm/sec and its strength dropped to 18.3°. The rate of displacement was increased again gradually to 8.28 cm/sec but no effect was observed -above 1.88 cm/sec- on the strength of the sample which had stabilised within a range between 21.3° and 23.4°. This fast shearing part of the test was terminated at a cumulative shear displacement of 670 cm. It should be noted at this point that throughout the duration of the test no evidence of an undulating shear surface were observed.

Subsequently, the sample was sheared at 0.09 mm/min and its measured strength decreased from its initial value of 16.7° until it reached its slow residual value after 20.8 mm of shearing. The strength stabilised at 10.9° until the test was terminated after 133 mm of shearing.

Test VI.2

σ_n= 200 kPa

OCR= 2

$\phi_{residual}$= 11.0°

$\phi_{threshold}$= 21.5°

(Figures VI.2.1-3)

The same sample was loaded to 200 kPa, and sheared at 0.09 mm/min. The measured strength stabilised at its residual value of 11.0° from the beginning of this slow shearing part of the test until the latter was terminated after a displacement of 133 mm. By then, the sample height had decreased by 0.2 mm due to sample loss.

The rate of displacement was increased suddenly to 0.31 cm/sec and the measured strength reached its immediately threshold value of 21.5°. While the rate of displacement continued to increase -it reached 1.04 cm/sec after 7.37 cm of shearing

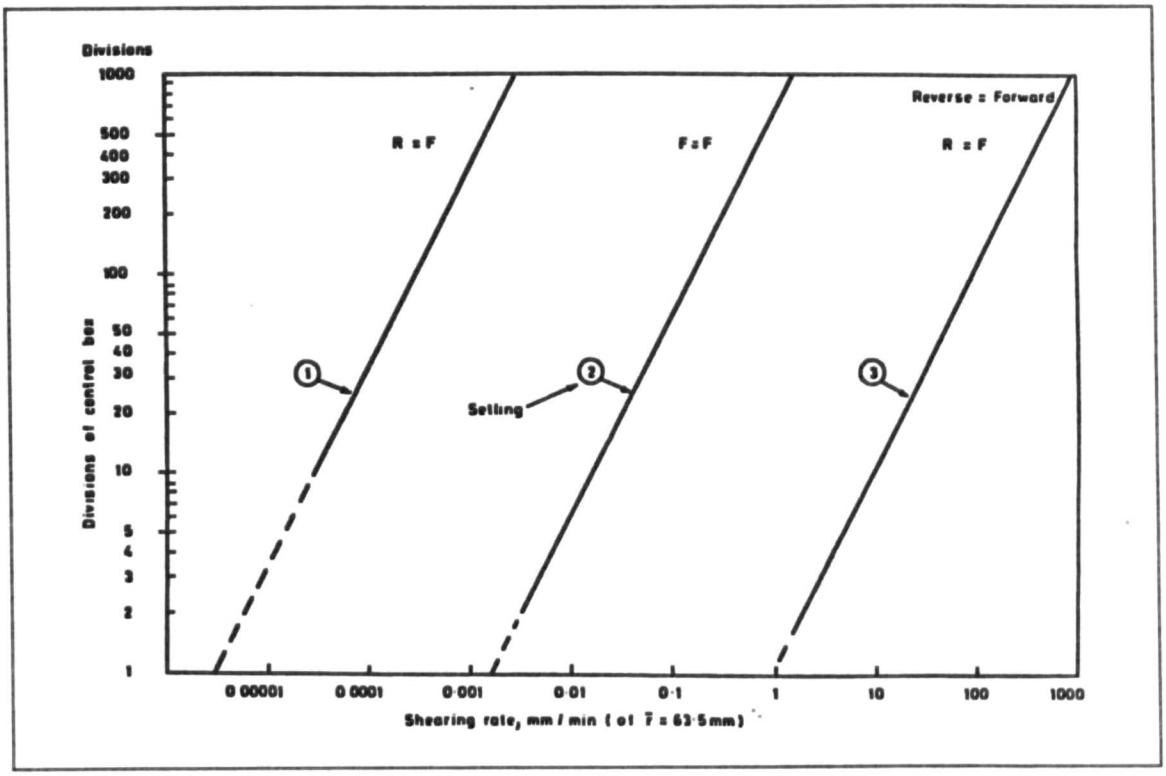


Fig. AI.3.2 Calibration of Motor M2-Ring Shear 2 (Tika, 1989)

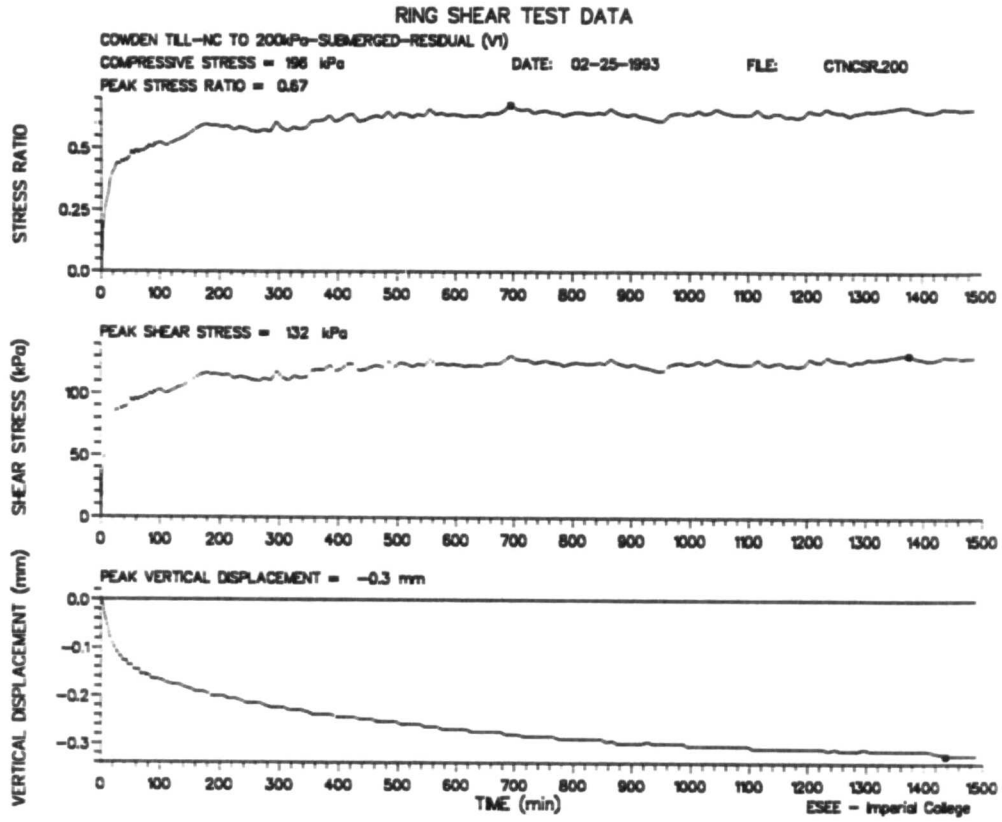


Fig. IV.19.1 Slow shearing test on normally consolidated, submerged Cowden till (200 kPa)

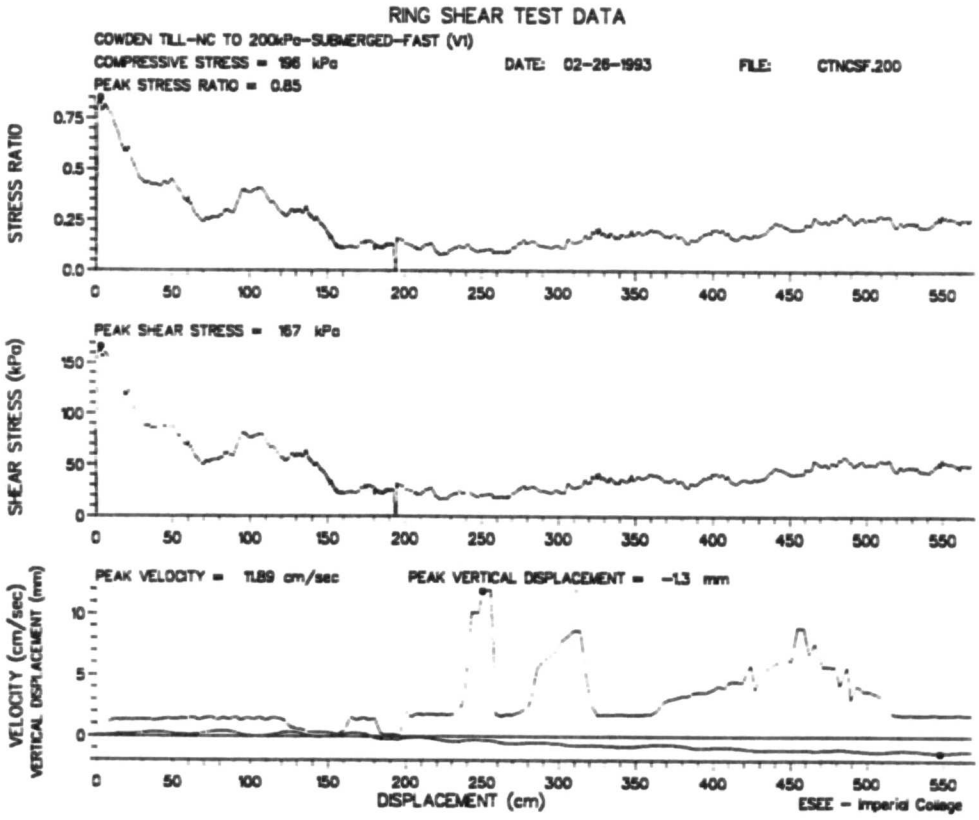


Fig. IV.19.2 Fast shearing test on normally consolidated, submerged Cowden till (200 kPa)

cm/sec, the fast residual strength of the sample appeared to increase slightly and varied between 36.5° and 39.7°. The subsequent decrease of the rate of displacement to 0.9 cm/sec again had no significant effect on the measured strength. After the sample had been sheared for 272.4 cm the rate of displacement was increased to 1.70 cm/sec. The rate of displacement was varied, suddenly or gradually, from 1.70 cm/sec, to 7.8 cm/sec and to 9.66 cm/sec before the test was terminated at 580 cm of cumulative displacement. However, no effect on the measured strength was observed.

Test VI.6

σ_n= 200 kPa

$\phi_{residual}$= 37.6°

$\phi_{threshold}$= 42.3°

(Figures VI.6.1-2)

The sample was sheared at 0.09 mm/min, and the measured strength reached its slow residual value of 37.6° after 34.2 mm. This slow shearing part of the test was terminated after the sample had been sheared for 131.1 mm.

The rate of displacement was increased suddenly to 1.27 cm/sec and the measured strength reached immediately its threshold value of 42.3°. The sample was sheared at the same rate of displacement for a cumulative 111.7 cm of shear displacement while the its strength stabilised within a range between 41.2° and 43.8°. After the sample had been sheared for 111.7 cm at 1.27 cm/sec the rate of displacement was decreased gradually to 0.1 cm/sec and then increased back to 1.27 cm/sec but no effect on the fast residual strength of the sample was observed. After 228 cm of cumulative shearing the rate of displacement was changed from 1.77 cm/sec, to 8.65 cm/sec, to 9.02 cm/sec and to 3.47 cm/sec until the test was terminated after 660 cm of shearing. However, the strength of the sample stabilised within a range between 41.2° and 43.8°, and no effect of the varying rate of displacement was observed.

Overconsolidated

Test VI.7

σ_n= 100 kPa

OCR= 4

Therefore, the error in the stress ratio which is introduced by the inaccuracies in shear stress measurement can be calculated by the following equation:

$$\varepsilon = \frac{(\tau/\sigma_n)_A - (\tau/\sigma_n)}{(\tau/\sigma_n)} = \frac{F_A\text{-}F}{F} \qquad \text{(AIII.4).}$$

This error was calculated for several loads which were applied on the load cells of Ring 2 [Fig. AIII.1-2] and it was observed that, while at very low loads the error may be as high as 10 %, it drops rapidly to about 1 % when the load increases above 200 N. It was concluded that this error would have an insignificant effect on the test results in the present investigation.

APPENDIX V

DESCRIPTION OF TESTS ON NON-PLASTIC SOILS

Tests On Pure Silica:

General:

The soil which was used in this series of tests was a commercially available pure silica which was available at the Soil Mechanics Laboratory of Imperial College.

Normally Consolidated, Non-Submerged

Test V.1

σ_n= 50 kPa

$\phi_{residual}$= 44.4°

$\phi_{threshold}$= 54.3°

(Figures V.1.1-2)

The sample was sheared at 0.09 mm/min and after 13.3 mm of shearing the strength had reached its slow peak value of 45.6°, and the sample height had decreased by 0.41 mm. Further shearing was associated with an additional 0.01 mm decrease in sample height as the measured sample strength reached its as residual value of 44.4° after 30.4 mm of displacement. This slow shearing part of the test was terminated after 136.8 mm of shearing while the sample height had decreased by an additional 0.02 mm indicating minimal soil loss.

The rate of displacement was suddenly increased to 1.7 cm/sec and the measured strength reached its threshold value of 54.3° at zero associated displacement. Further shearing at the same rate of displacement led to a decrease in strength as the sample appeared to dilate. After 20.2 cm of shearing the sample height had increased by 0.06 mm, and the measured sample strength decreased to 45.03°. The fast residual strength of the sample stabilised around 46.1° between 20.2 cm and 58.5 cm of cumulative displacement while the sample height appeared to be decreasing due to soil loss. The rate of displacement was suddenly decreased to 0.1 cm/sec after 124.1 cm of shearing but this had no effect on the strength which varied

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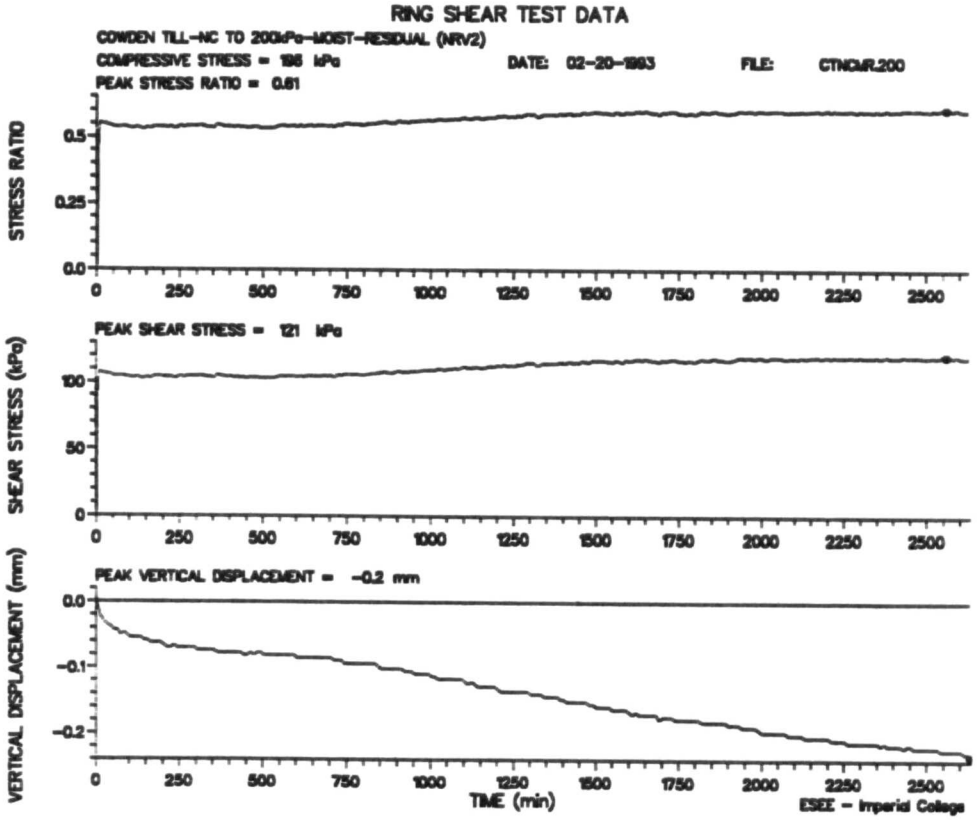


Fig. IV.15.1 Slow shearing test on normally consolidated, non-submerged Cowden till (200 kPa)

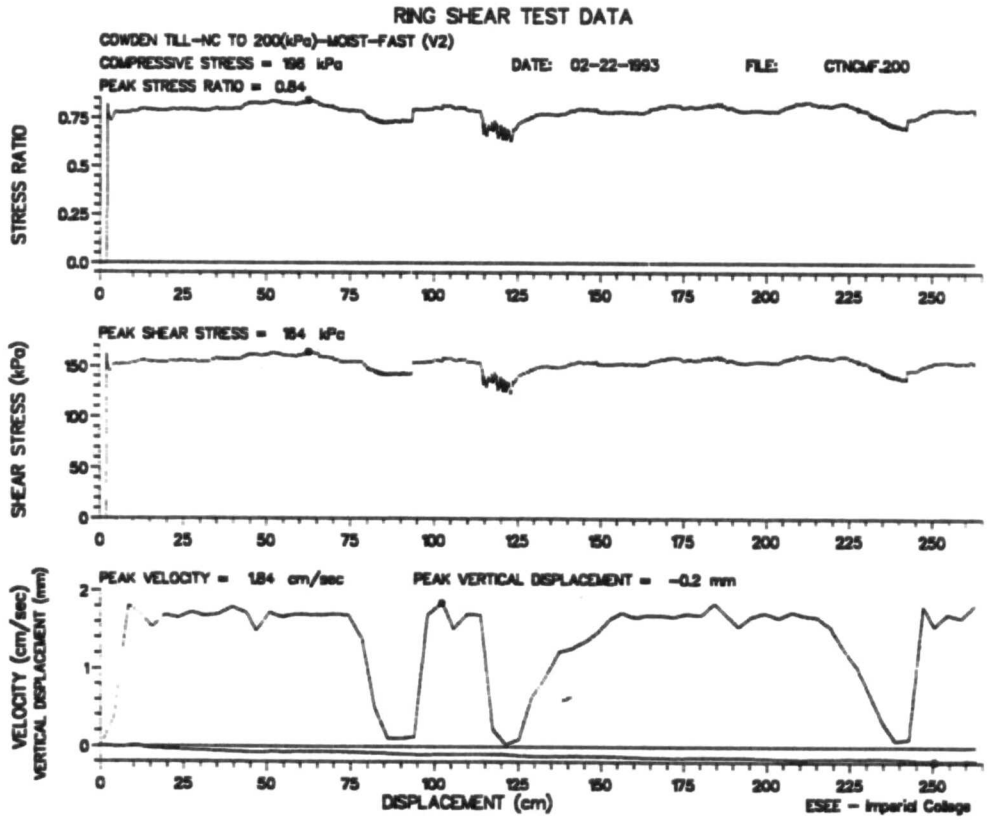


Fig. IV.15.2 Fast shearing test on normally consolidated, non-submerged Cowden till (200 kPa)

APPENDIX II

COMPUTER PROGRAM FOR DATA ACQUISITION, CALCULATION, AND PRESENTATION OF RESULTS

AII.1 Introduction

The computer program called **"Ringtest"** enables the operator to calibrate all the transducers of the system, conduct consolidation and/or shearing procedures simultaneously in both ring shear devices, collect and store the readings from each transducer, and calculate and display the results from each test on the PC screen while the tests are in progress. The program is written in Quick Basic and it comprises the following six sub-routines:

1) "Opencommunication"
2) "Calibration"
3) "Consolidation"
4) "Shearing"
5) "Graphdisplay"
6) "Printing"

In the remainder of this Appendix the basic functions of these sub-routines will be described and brief guidelines for their operation will be presented.

A.II.2 "Opencommunication"

This is the first sub-routine which is activated and establishes the connection between the PC and the transducers of both ring shear devices through the data measurement processor which has been described in Chapter 3. This sub-routine opens all communication channels automatically and it does not require any interaction with the operator.

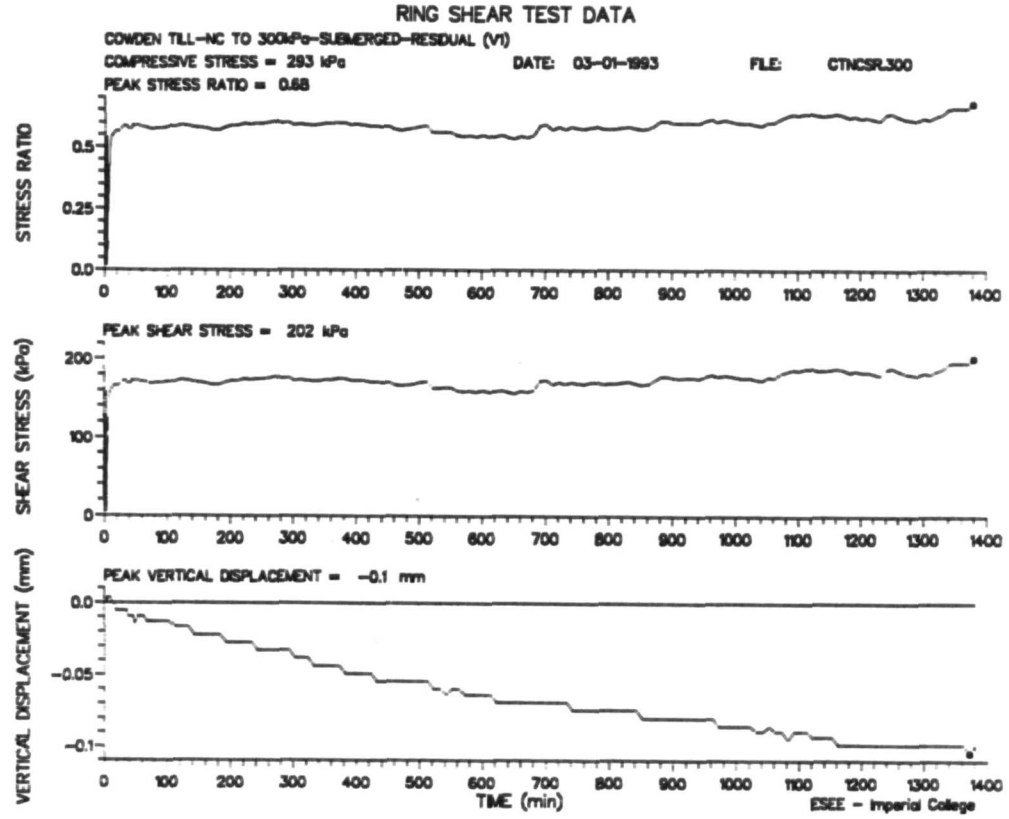


Fig. IV.20.1 Slow shearing test on normally consolidated, submerged Cowden till (300 kPa)

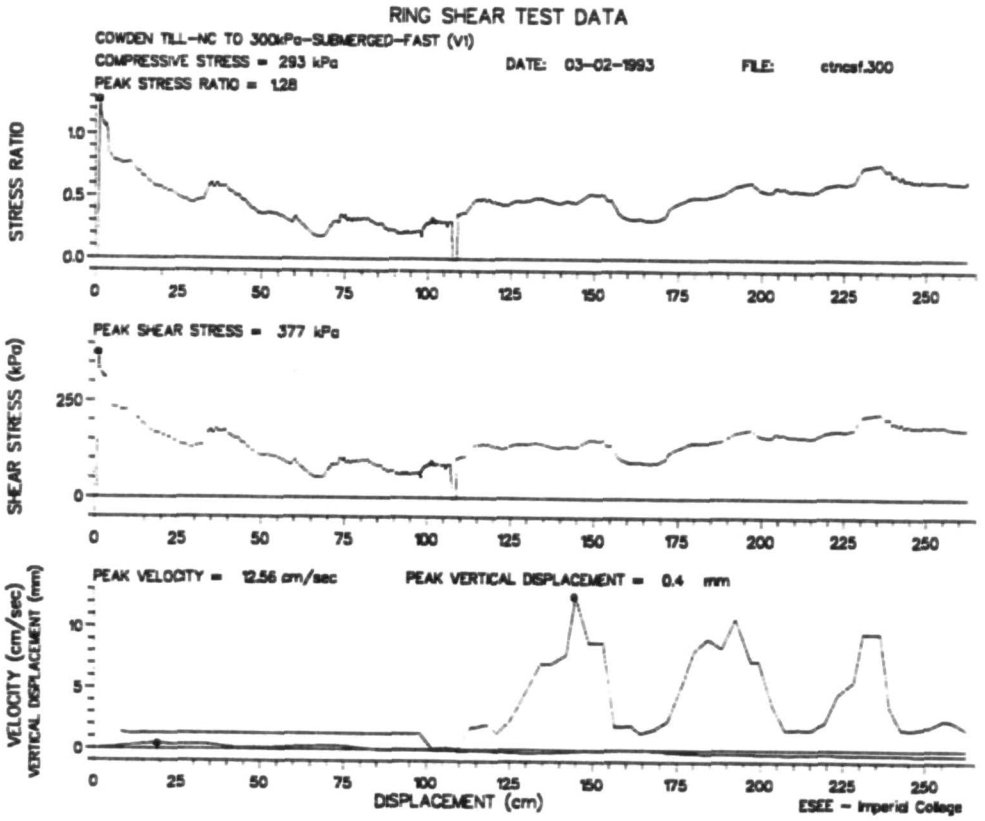


Fig. IV.20.2 Fast shearing test on normally consolidated, submerged Cowden till (300 kPa)

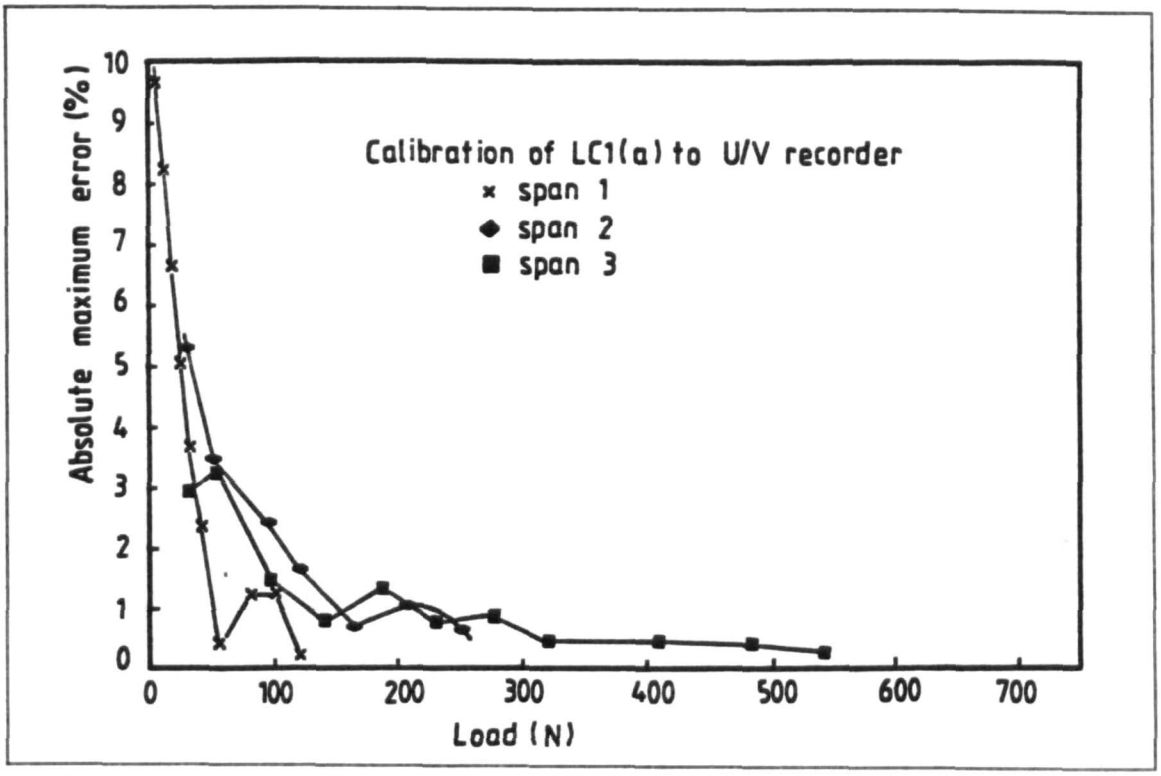


Fig. AIII.1 Error due to Shear Stress Measurement by Load Cell 1 (Ring 2), Tika (1989)

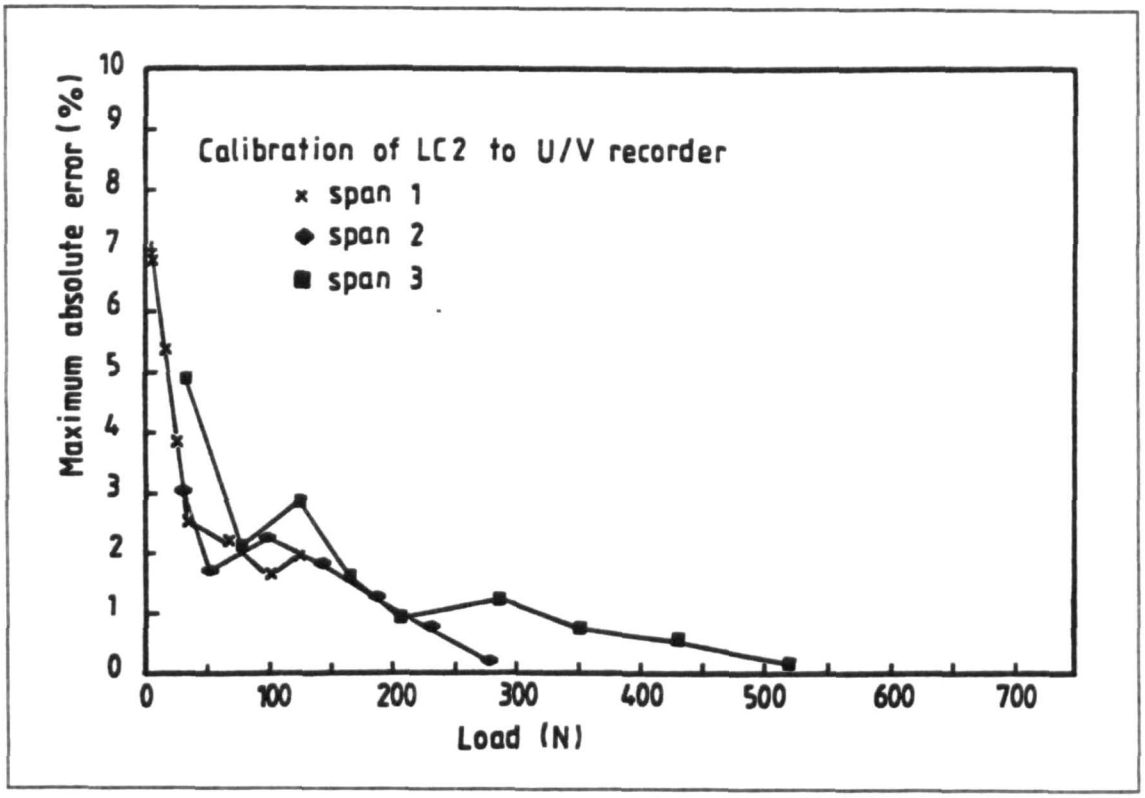


Fig. AIII.2 Error due to Shear Stress Measurement by Load Cell 2 (Ring 2), Tika (1989)

between 43.1° and 45.5°. Before this test was terminated at 380 cm of cumulative shearing, the rate of displacement was varied between 0.1 cm/sec and 1.7 cm/sec, suddenly and gradually, but no effect of those changes on the strength of the sample was observed.

Test V.2

σ_n= 100 kPa

$\phi_{residual}$= 44.4°

$\phi_{threshold}$= 51.3°

(Figures V.2.1-2)

The sample was sheared at 0.09 mm/min and its strength reached its slow peak value of 49.0° after a displacement of 1.9 mm. Further shearing led to a decrease in strength which reached its slow residual value of 44.4° after 74.1 mm of displacement. This slow shearing part of the test was terminated after 119.7 mm of displacement. It should be mentioned that the sample height curve should not be taken into consideration since it was probably affected by a malfunction of the transducer.

The rate of displacement was suddenly increased to 1.69 cm/sec and the strength reached its threshold value of 51.3° at zero associated displacement. Further shearing at the same rate of displacement led to a decrease in strength as the sample appeared to dilate. After 25 cm of shearing the sample height had increased by 0.064 mm, and the strength had decreased to 44.4°. Shearing at the same rate of displacement was continued until a cumulative displacement of 142.5 cm while the strength of the sample fluctuated between 41.7° and 43.1°. The rate of displacement was then decreased suddenly to 0.11 cm/sec but this had no effect on the strength of the sample. Further changes of the rate of displacement, gradual or sudden, did not appear to affect the strength until the test was terminated after 440 cm of cumulative shearing.

Test V.3

σ_n= 200 kPa

$\phi_{residual}$= 45.4°

$\phi_{threshold}$= 52.0°

(Figures V.3.1-2)

El-Ruwayih, A.A. (1975). Design manufacture and performance of a lateral strain device. Geotechnique, Vol. 26, No. 1, 215-216.

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Hungr, O. and Morgenstern, N.R. (1984). High velocity ring shear tests on sand. Geotechnique, Vol. 34, No. 3, 415-421.

Hutchinson, J.N. and Bhandari, R.I. (1971). Undrained loading: a fundamental mechanism of mudflows and other mass movements. Geotechnique, Vol. 21, No. 4, 353-358.

Hutchinson, J.N. (1986). A sliding-consolidation model for flow slides. Can. Geot. Journal, Vol. 23, No. 2, 115-126.

Hvorslev, M.J. (1936). A ring shearing apparatus for the determination of the shearing resistance and plastic flow of soils. Proc. 1st I.C.S.M.F.E., Harvard University, Cambridge, Massachusetts, Vol. 2, 125-129.

Idriss, I.M. (1981). Evaluating seismic risk in engineering practice. Proc. 11th I.C.S.M.F.E., San Francisco, Vol. 1, 255-320.

Kenney, T.C. (1967). The influence of mineral composition on the residual strength of natural soils. Proc. Geotech. Conf., Oslo, Vol. 1, 123-129.

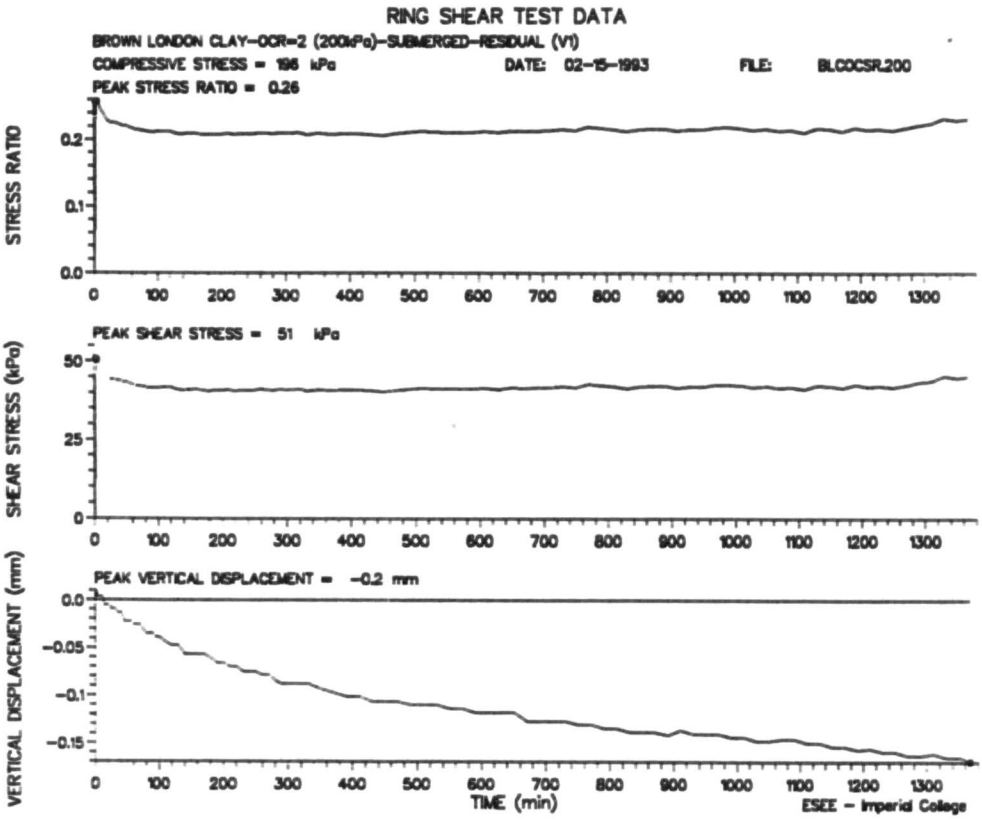


Fig. IV.11.1 Slow shearing test on over-consolidated, submerged brown London clay (200 kPa - OCR= 2)

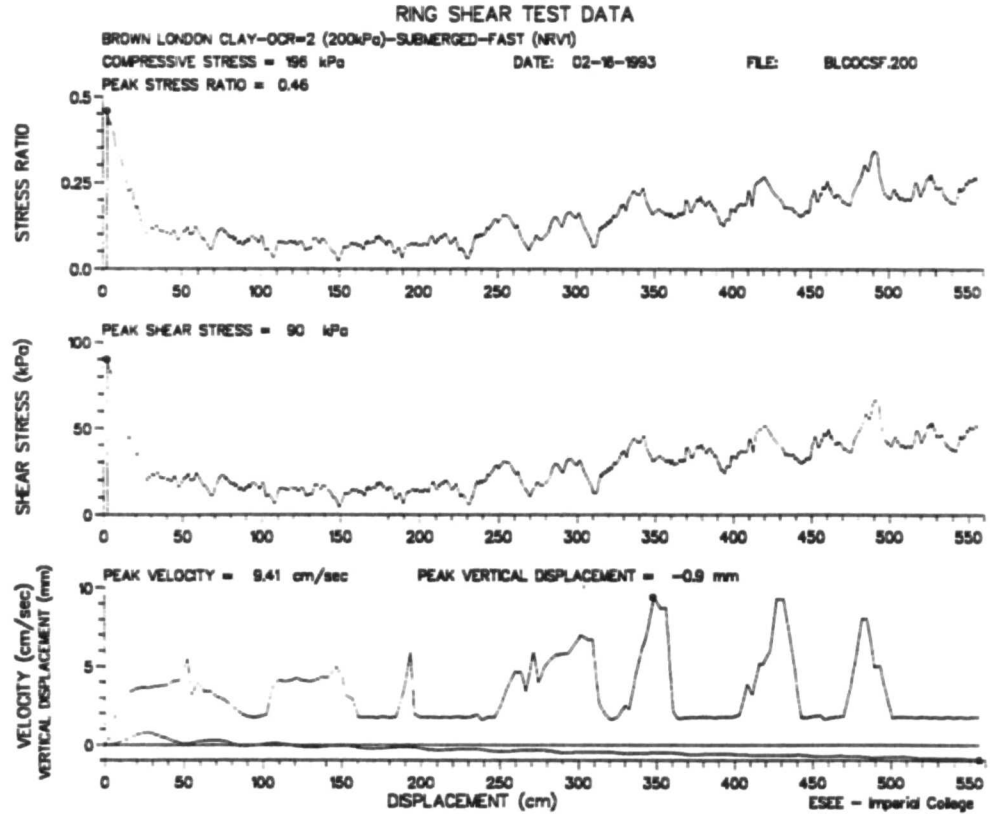


Fig. IV.11.2 Fast shearing test on over-consolidated, submerged brown London clay (200 kPa - OCR=2)

APPENDIX I

SYSTEM CALIBRATION

AI.1 Calibration of the Instrumentation

The load cells, proving rings, and displacement transducers of the ring shear devices were calibrated using the procedure which has been adopted as standard in laboratory research. The physical parameters measured by each transducer were varied systematically and the output readings were recorded for the determination of the calibration constants. The calibration procedure involved 30 increase/decrease cycles of each physical parameter in order to ensure the steady response of each transducer. Subsequently, the calibration constants were calculated from the relationships between the physical parameters and the output readings. The calibration procedure was repeated every three months during the period of the laboratory research.

The load cells were calibrated in compression by means of the *Budenberg* dead-weight calibrator and in tension by directly applying tensile load. Since the minimum load which can be applied through the Budenberg calibrator is 108.5 N, smaller compressive loads were also applied directly to the load cells. After the end of each test each load cell was zeroed in order to observe any substantial drift in the output which would have indicated malfunction of the measuring device.

The proving rings were calibrated by applying directly on them a sequence of compressive and tensile loads.

The vertical displacement transducers were calibrated by means of a *vernier* calibrator with 0.01 mm/div resolution.

The horizontal displacement transducer, for reasons which have been explained in Chapter 3, was not calibrated.

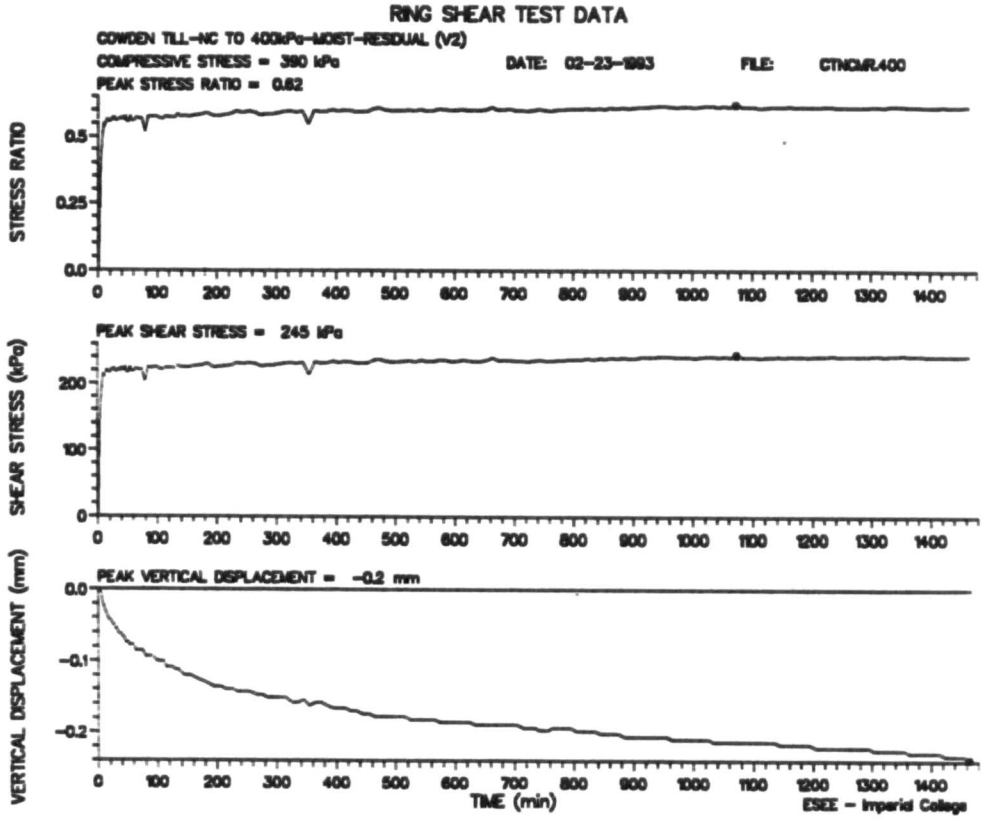


Fig. IV.16.1 Slow shearing test on normally consolidated, non-submerged Cowden till (400 kPa)

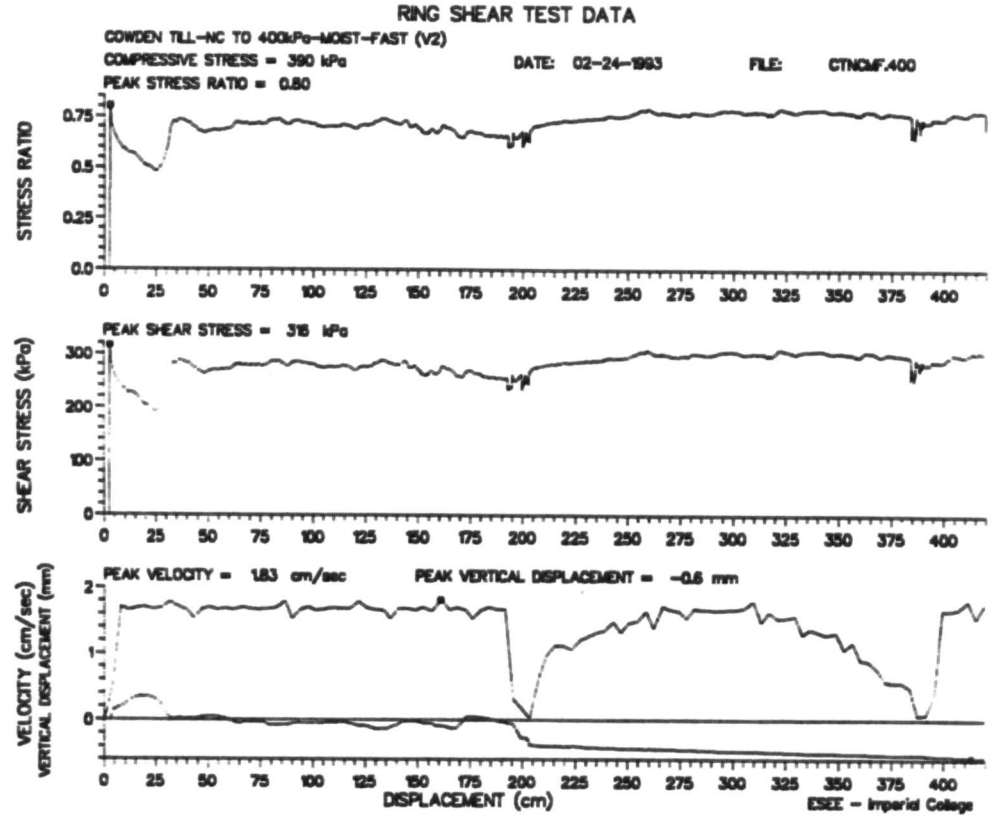


Fig. IV.16.2 Fast shearing test on normally consolidated, non-submerged Cowden till (400 kPa)

A.II.3 "Calibration"

This sub-routine enables the operator to conduct the calibration of all channels which are connected to transducers. The operator has to input the number of channels which will be calibrated and the number of different readings from each channel which will be taken for the calculation of the calibration constants. After all channels have been calibrated, the constants are saved in a file and are recalled each time the data measurement processor is activated. Subsequently, each time the program is initiated, the operator is prompted to choose which calibration file will be used and whether any channel has to be re-calibrated or its calibration checked. Therefore, before a new test is conducted, the zero drift or possible malfunctions of each transducer may be checked and corrected.

A.II.4 "Consolidation" & "Shearing"

These sub-routines form the core of the program since they are associated with the testing procedures. The operator has to indicate which procedure and what type of test will be conducted in each apparatus, what the approximate duration of the test will be (mins), whether the sample will be sheared "slow" or "fast", what is the normal load (kN) which is applied on the hanger, whether there will be pore pressure and temperature measurements, and at what frequency will readings from the transducers will be recorded. Furthermore, the operator is able to interrupt the scanning of the transducers at any point during a test and alter the frequency when it is required. After all the preliminary information is provided, the program prompts the operator to specify a name for the file in which the test results will be saved, takes the initial readings from all transducers involved in the previously specified testing procedure and records the initial time and date along with one line of information serving as the title of the file. The operator may interrupt and terminate the test conducted in one of the ring shear devices without affecting the procedure which involves the other device. Furthermore, the program enables the operator to view on the PC screen the results from a previously conducted test while another test is still in progress in one of the ring shear devices.

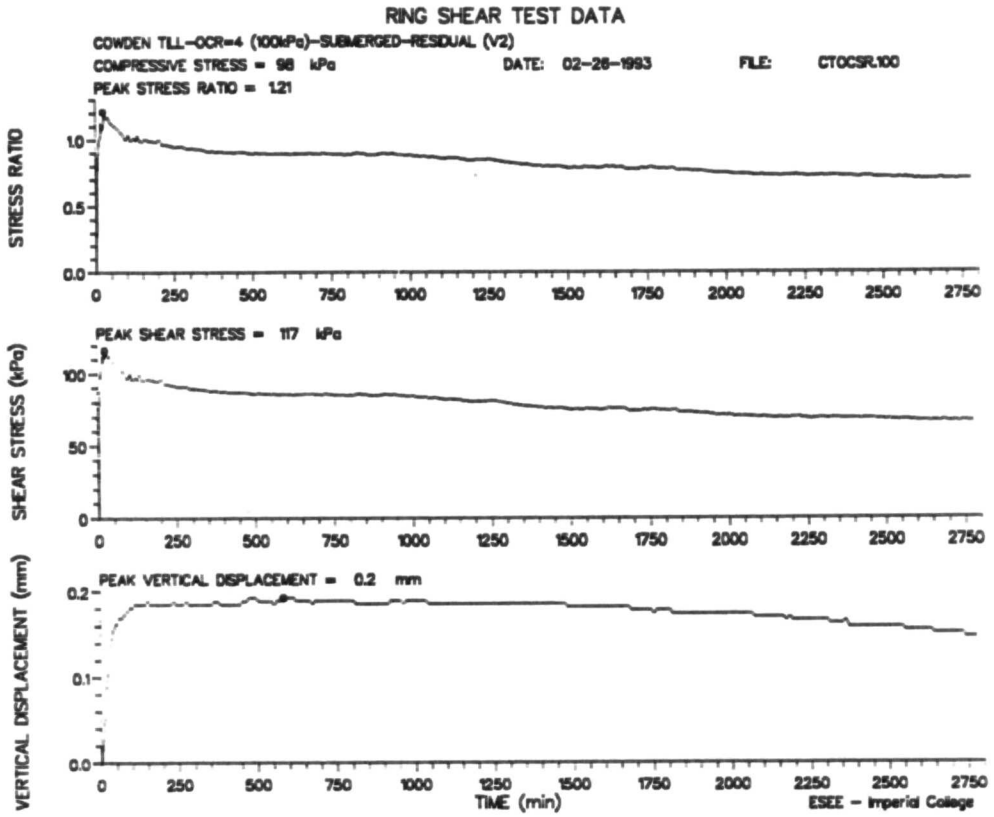


Fig. IV.21.1 Slow shearing test on over-consolidated, submerged Cowden till (100 kPa - OCR= 4)

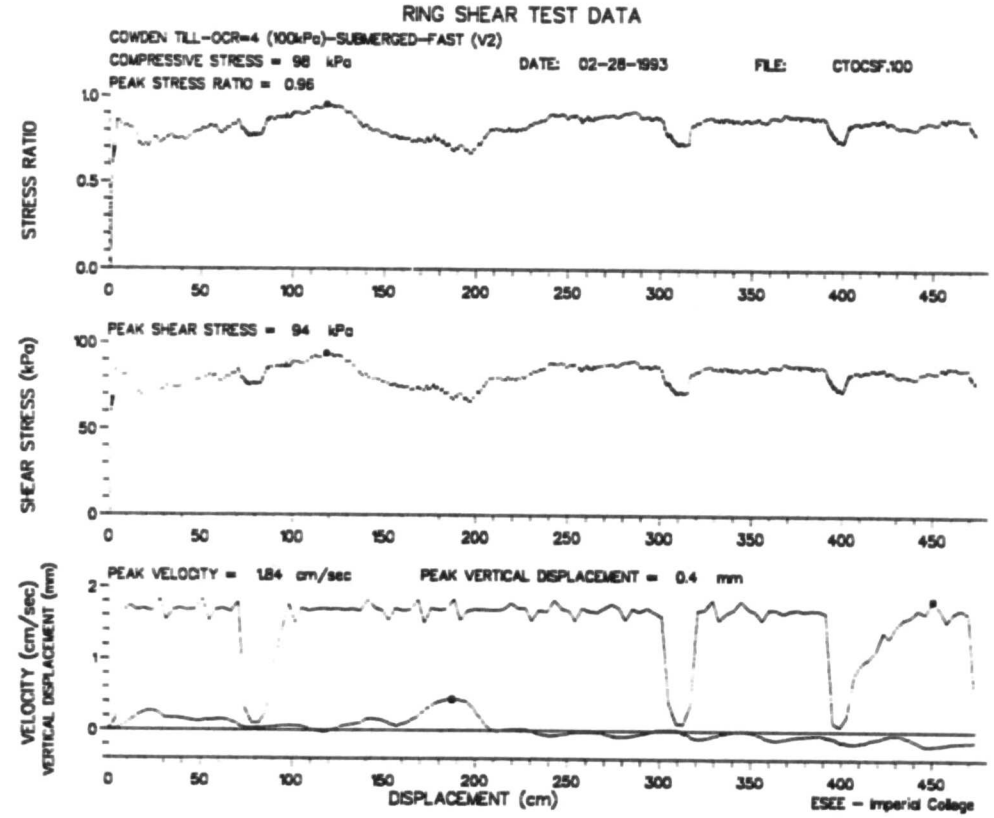


Fig. IV.21.2 Fast shearing test on over-consolidated, submerged Cowden till (100 kPa - OCR= 4)

APPENDIX IV

DESCRIPTION OF TESTS ON PLASTIC SOILS

Tests on brown London clay:

Normally Consolidated, Non-Submerged

The soil was left to hydrate overnight after it had been remoulded, it was placed in the confining rings at a water content around its plastic limit and it was not submerged in water. After the apparatus had been assembled, the sample was loaded to the lowest normal stress level of 50 kPa. The multistage testing technique was followed unless otherwise stated.

TEST IV.1

σ_n = 50 kPa

$\phi_{residual}$ = 15.6°

$\phi_{threshold}$ = 25.8°

(Figures IV1.1-2)

The sample was sheared until it had reached its residual state at the constant rate of displacement of 0.09 mm/min for a total time of shearing of 4300 mins and a shear displacement of 38.7 cm. It was observed that after the first 4.5 cm of displacement the shear strength of the sample had already stabilised at its residual value of 15.6°, however, the shearing at the slow rate of displacement was continued in order to verify that the sample had reached its residual state and also to check the behaviour of the new rings in slow shearing for a full rotation. After a shear displacement of 1.64 mm the sample had reached its slow peak strength of 28.4° and at 29.3 mm of shear displacement the sample had reached its residual state. The first 0.14 mm of decrease in sample height must be attributed to particle reorientation and sample loss, where as the additional 0.082 mm must be attributed solely to sample lost through the gap.

The rate of displacement was increased suddenly to 0.72 cm/sec and the strength of the sample reached its threshold value of 25.8° at zero shear displacement.

The sample was sheared at 0.09 mm/min and the strength reached its slow peak value of 48.5° after a displacement of 1.9 mm. Further shearing led to a gradual decrease in measured strength which reached its slow residual value of 45.4° after 89.3 mm of shearing. The slow shearing part of this test was terminated after 143.6 mm of displacement. It should be noted at this point that the sample height curve of this part of the test should not be taken into consideration because it has probably been affected by a malfunction of the vertical displacement transducer.

The rate of displacement was suddenly increased to 1.69 cm/sec and the measured strength reached immediately its threshold value of 52.0°. As shearing continued at the same rate of displacement, the sample dilated, and the strength decreased. After 7 cm of shearing the sample height had increased by 0.08 mm and the strength had decreased to its slow residual level. Further shearing at 1.69 cm/sec led to further decrease in strength which fluctuated between 41.9° and 44.4°. After 114 cm of shearing the rate of displacement was decreased suddenly to 0.082 cm/sec, gradually increased to 1.69 cm/sec, and decreased again in similar manner. However, neither these nor any other changes of the rate of displacement had any effect on the strength of this sample. This testing procedure was terminated after 335 cm of cumulative shearing, by which point the sample height had decreased by 0.7 mm mainly because of soil lost through the gap of the rings.

Test V.4

σ_n= 300 kPa

$\phi_{residual}$= 45.2°

$\phi_{threshold}$= 52.0°

(Figure V.4.1-2)

The sample was consolidated under 300 kPa normal stress and sheared at 0.09 mm/min. The measured sample strength reached its slow peak value of 50.7° after 5.5 mm of shearing and its residual value of 45.2° after 115.9 mm. The dilation of the sample indicated by the sample height curve may have been due to malfunction of the vertical displacement transducers or may, indeed, have been due to the nature of the sample. This slow shearing part of the test was terminated after 135 mm of displacement.

The rate of displacement was increased suddenly to 1.63 cm/sec and the strength reached immediately its threshold value of 52.0°. The strength decreased

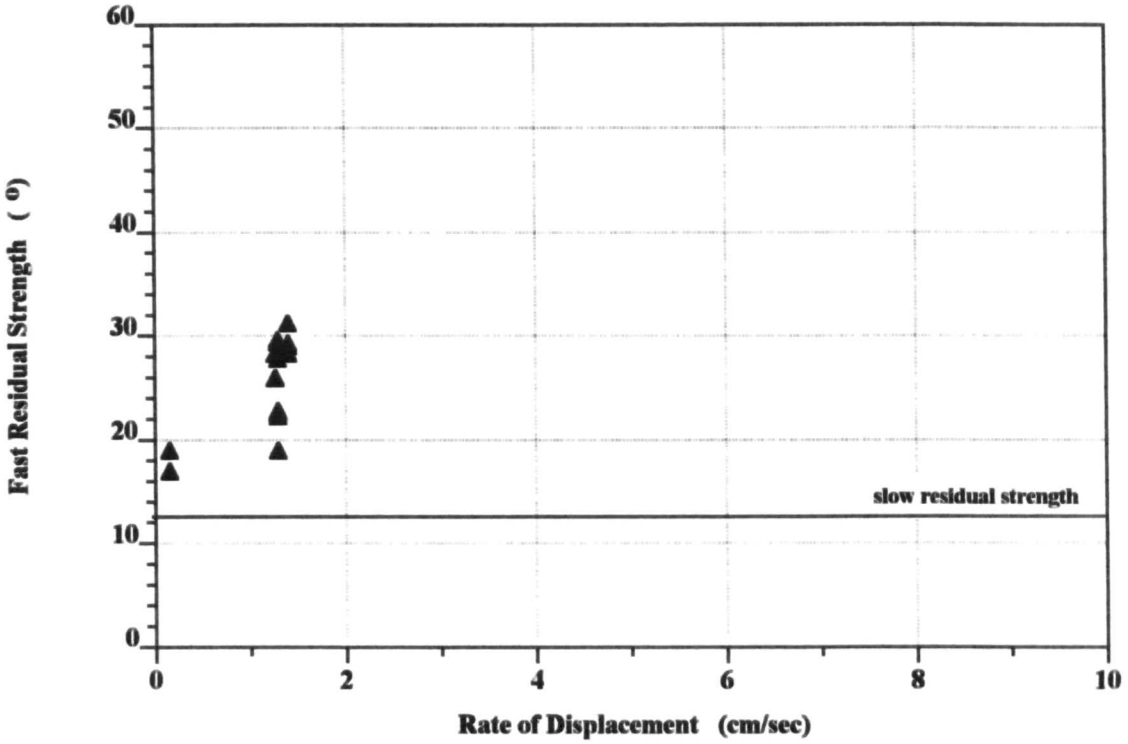


Fig. 6.3.47 Rate dependent characteristics of over-consolidated, non-presheared, brown London clay (100 kPa)

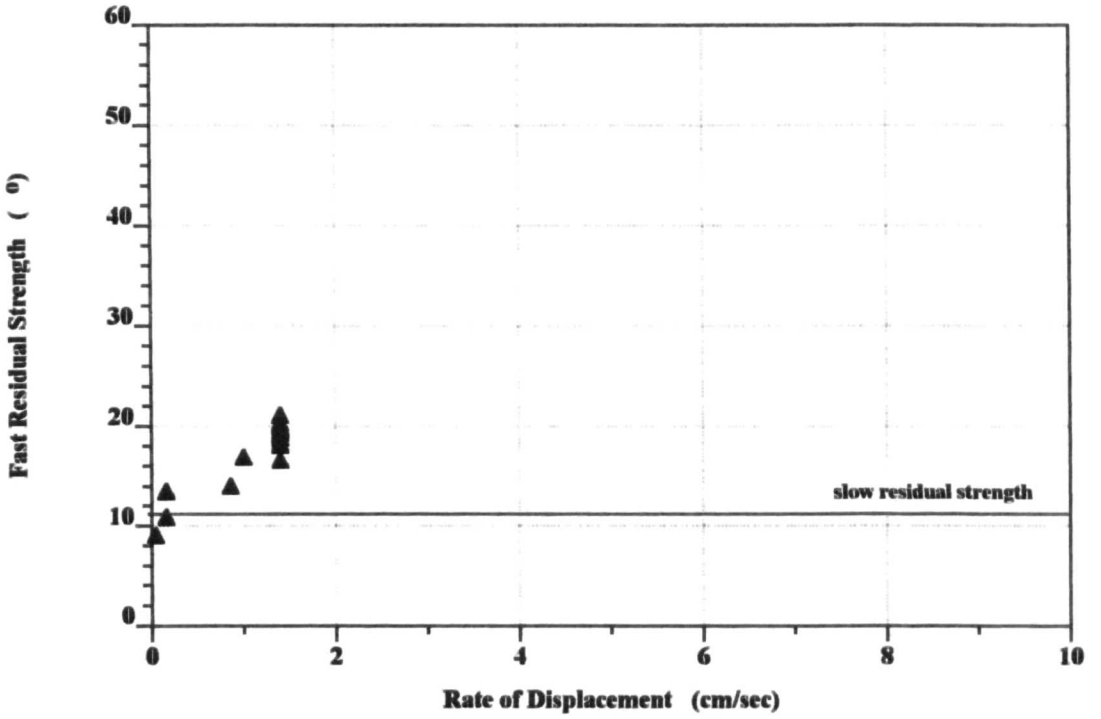


Fig. 6.3.48 Rate dependent characteristics of over-consolidated, non-presheared, brown London clay (200 kPa)

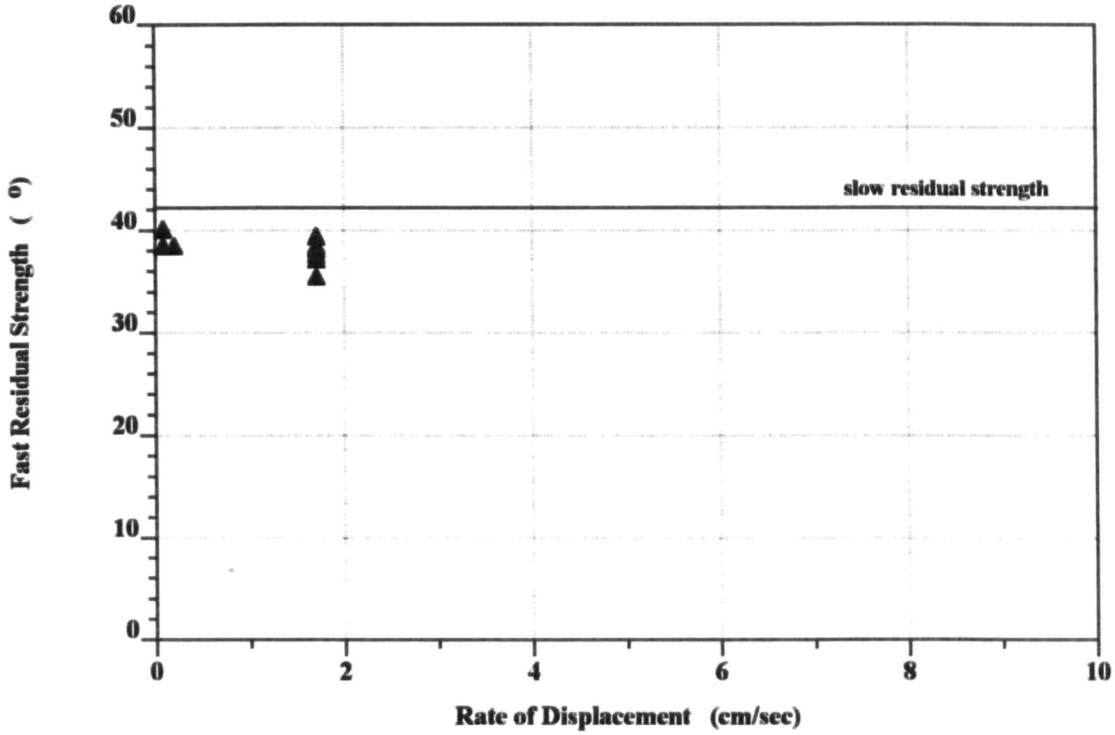


Fig. 6.3.57 Rate dependent characteristics of over-consolidated, completely dry, Cowden till (100 kPa)

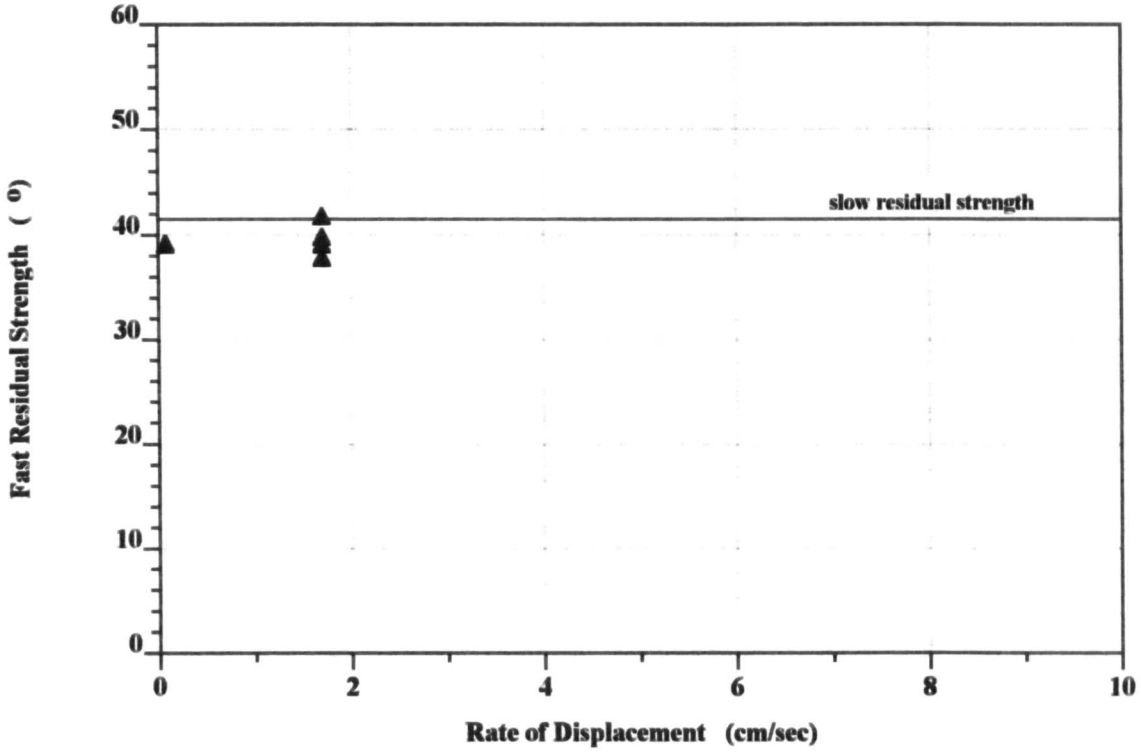


Fig. 6.3.58 Rate dependent characteristics of over-consolidated, completely dry, Cowden till (200 kPa)

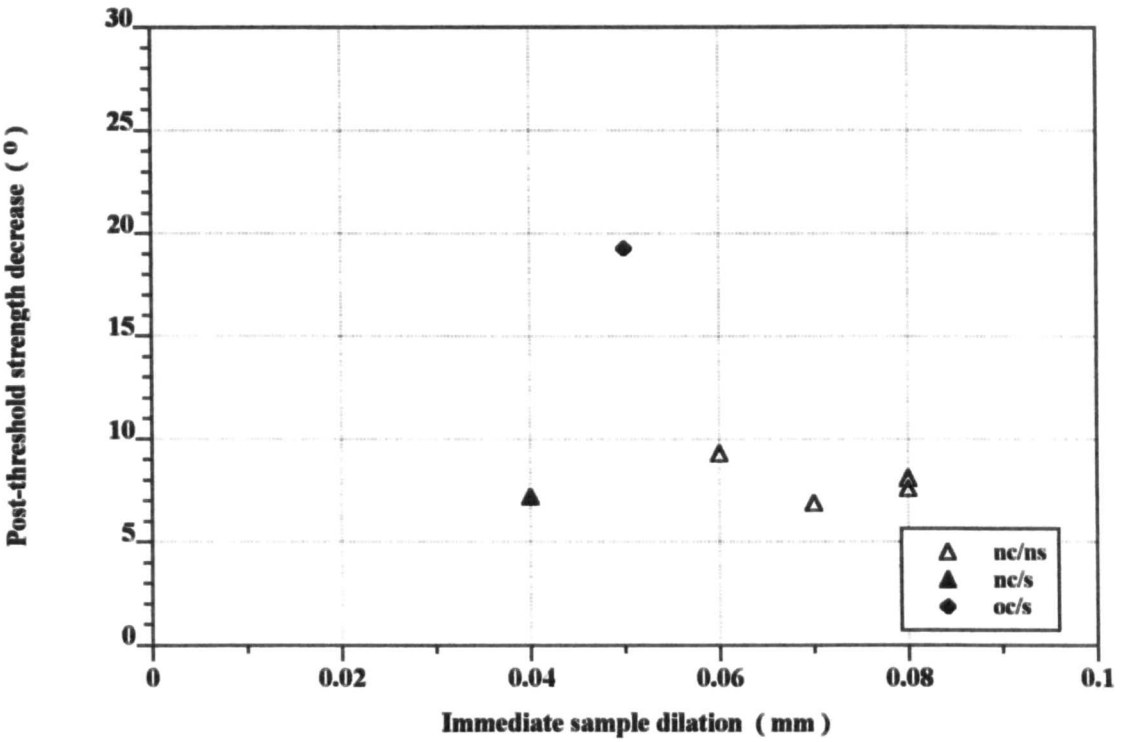


Fig. 6.4.9 Immediate sample dilation versous post-threshold strength decrease for pure silica samples

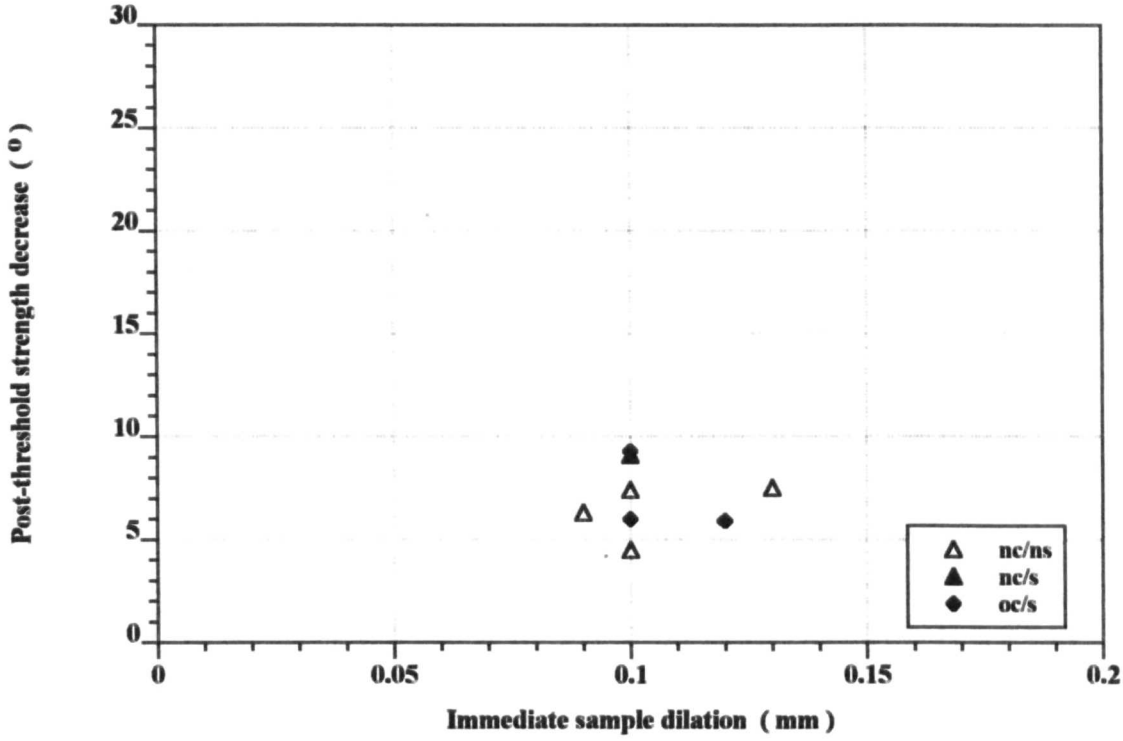


Fig. 6.4.10 Immediate sample dilation versous post-threshold strength decrease for sandy-silt samples

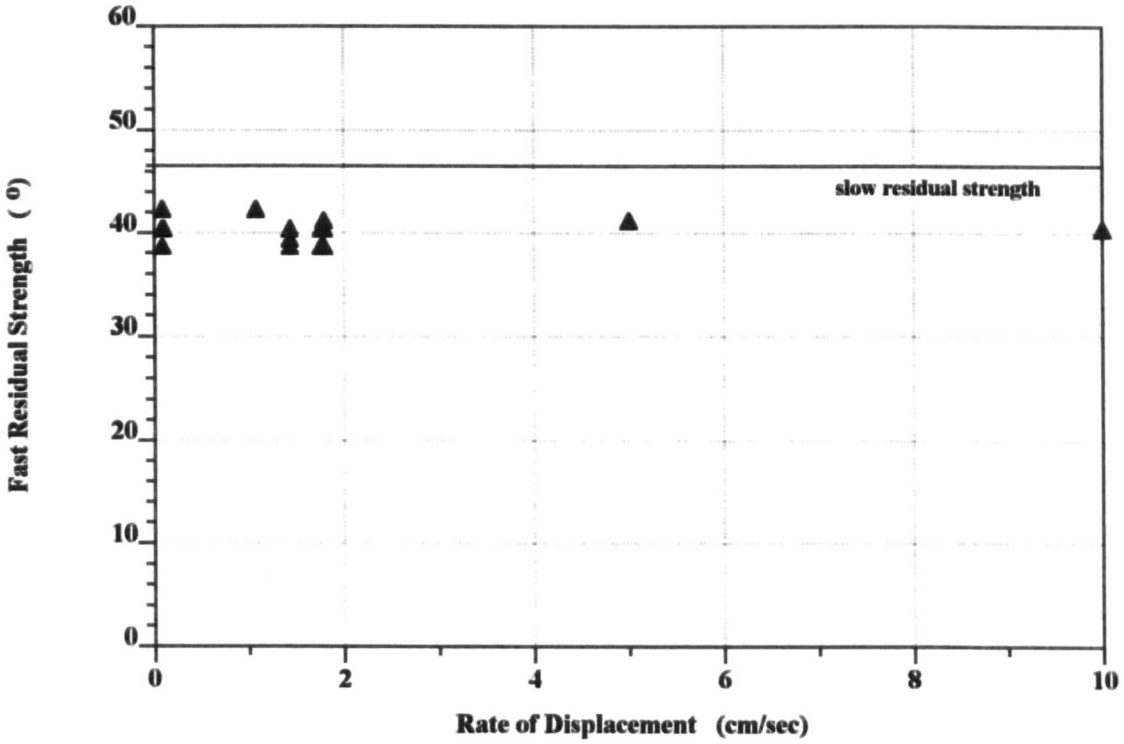


Fig. 6.4.19 Rate dependent characteristics of over-consolidated, submerged pure silica sample (100 kPa)

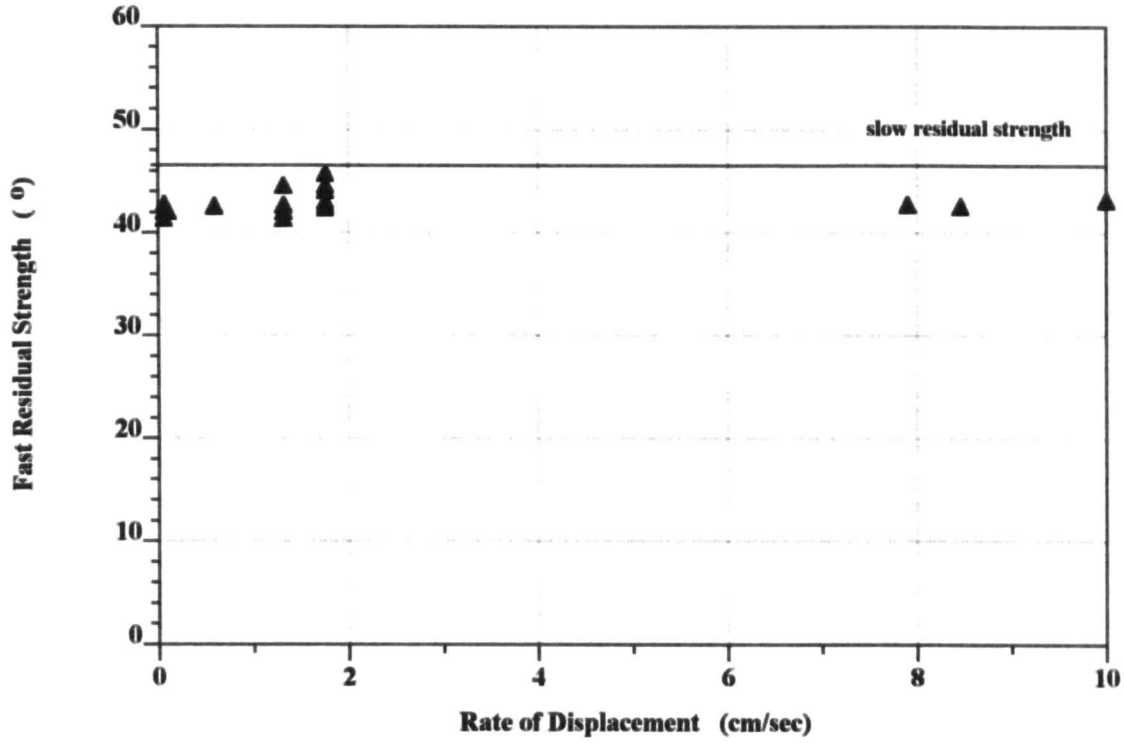


Fig. 6.4.20 Rate dependent characteristics of over-consolidated, submerged pure silica sample (200 kPa)

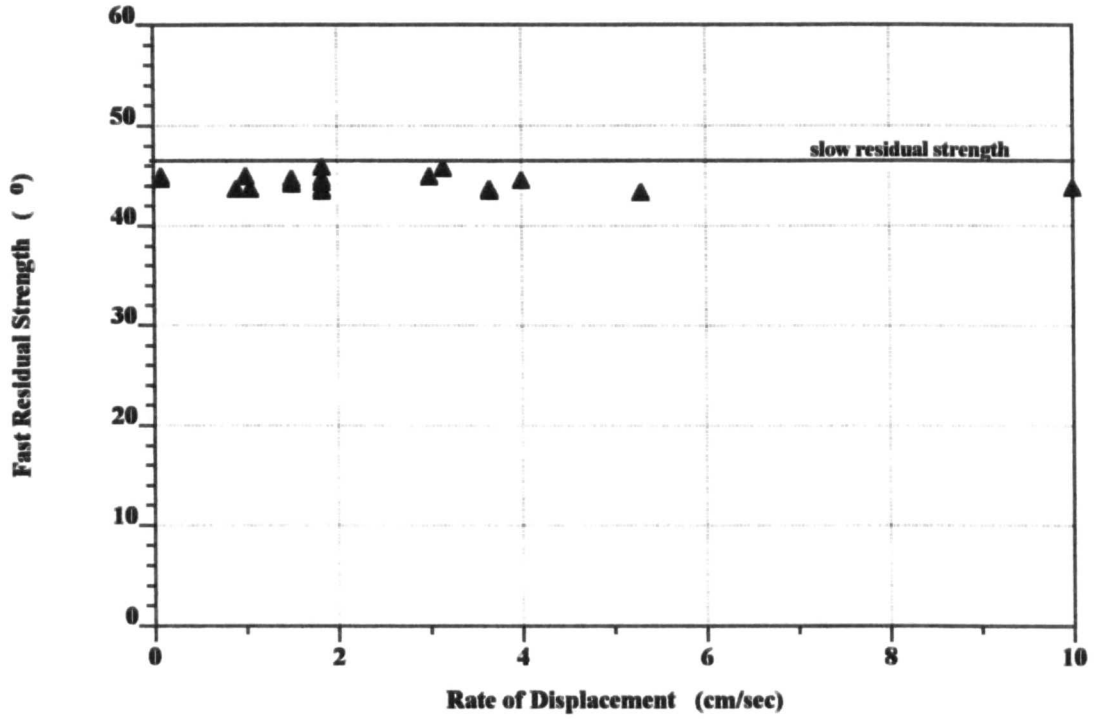


Fig. 6.4.29 Rate dependent characteristics of over-consolidated, submerged, sandy-silt sample (200 kPa)

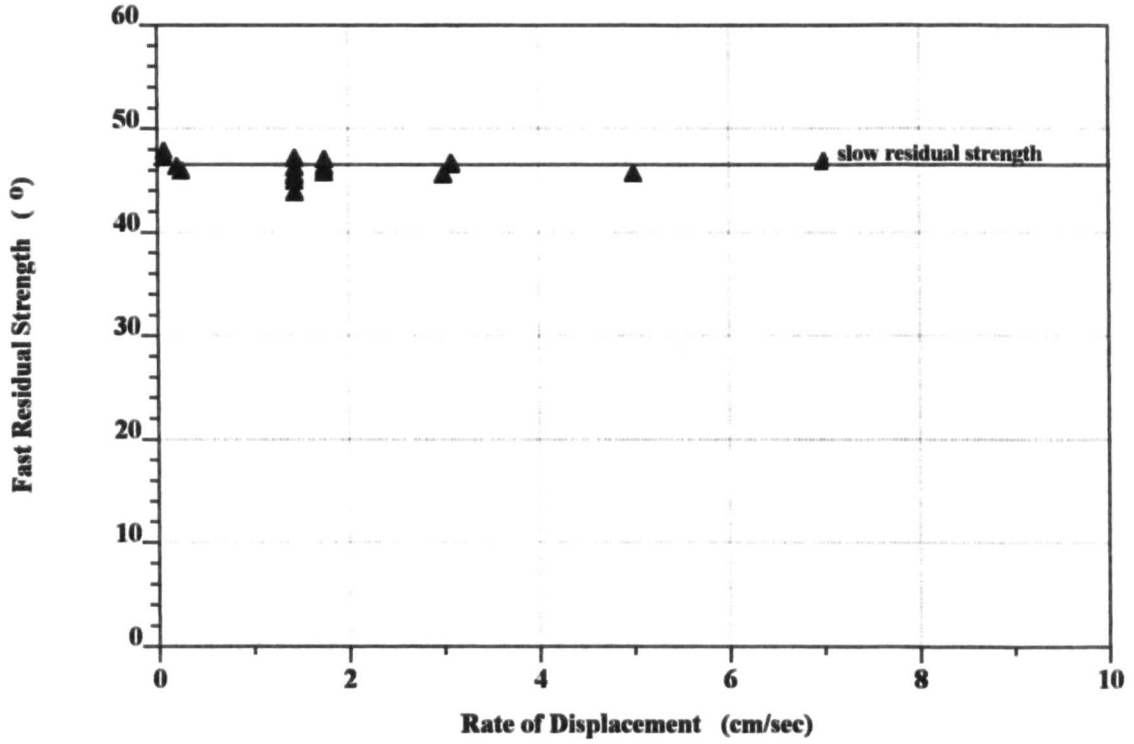


Fig. 6.4.30 Rate dependent characteristics of over-consolidated, submerged, sandy-silt sample (250 kPa)

influence the fast residual strength of samples which exhibit positive rate effects. Moreover, the fast residual strength of plastic soil samples which exhibit a positive rate effect decreases with increasing normal stress [Figs. 6.3.27-6.3.30]. Finally, when the rate of displacement increases from 0.2 to 2.0 cm/sec, the increase in fast residual strength becomes less significant at higher normal stresses, *i.e.* soils become less rate dependent at higher normal stresses. No relation between the magnitude of the increase in fast residual strength and the plasticity index of soils is observed from the present results.

The negative rate effect [*Type II* behaviour] occurs with all non-plastic soils used in this investigation (PI=0 %, 5 %<CF<9 %), except the sands, and with plastic soil samples which are submerged in water and which develop an undulating or non-planar shear surface.

In the case of the non-plastic soils the negative rate effect is attributed to an increase in the porosity of the shearing matrix resulting from the interparticle movements upon the application of fast rates of displacement. This is in agreement with the conclusions of previous researchers (Lemos, 1986; Tika, 1989). The results from this investigation indicate that the fast residual strength of non-plastic soils may be 4 to 12 % lower than their slow residual strength, is not rate dependent, and does not appear to be influenced by the increasing normal stress.

In the case of the plastic soils, the negative rate effect is consistent with the development of pore pressures induced by a mechanism of shearing involving an undulating shear surface. It has been shown that, if the rate of displacement is decreased to a rate which would allow for the dissipation of pore pressures, the shear strength of the sample gradually increases and returns to its slow residual level. Visual inspection of the material within the shear zone of plastic soil samples which exhibit a negative rate effect shows a soft texture, while the increased water content within this zone provides evidence of a liquefied structure. This suggests that the undulating shape of the shear surface induces a "pumping action" which, combined with the presence of water around the sample, causes the pore pressures within the shearing matrix to increase. If the amplitude of the undulating shear surface decreases during shearing, the "pumping action" diminishes, the pore pressures begin to dissipate, and the shear strength gradually increases.

Kenney, T.C. (1977). Residual strengths of mineral mixtures. Proc. 9th I.C.S.F.M.E., Tokyo, Vol. 1, 155-160.

Kerr, P.F. and Drew, I.M. (1968). Quick-clay slides in the U.S.A. Eng. Geol., Vol. 2, No. 4, 215-238.

La Gatta, D.P. (1970). Residual strength of clays and clay shales by rotation shear tests. Harvard Soil Mechanics series, No. 86, Cambridge, Massachussets.

La Gatta, D.P. (1971). The effect of rate of displacement on measuring the residual strength of clay. U.S. Army Waterways Experiment Station Contract Report 5-71-5, Aug. 1971.

Lemos, L.J.L., Skempton, A.W. and Vaughan, P.R. (1985). Earthquake loading of shear surfaces in slopes. Proc. 11th I.C.S.F.E., San Francisco, Vol. 4, 1955-8.

Lemos, L.J.L. (1986). The effect of rate of shear on residual strength soil. PhD. Thesis, University of London.

Lemos, L.J.L. (1991). Shear strength of shear surfaces under rapid loading. Proc. 10th E.C.S.M.F.E., Firenze, Italy, Vol. 1, 137-141.

Lupini, J.F. (1981). The residual strength of soils. PhD. Thesis, University of London.

Lupini, J.F., Skinner, A.E. and Vaughan, P.R. (1981). The drained residual strength of cohesive soils. Geotechnique, Vol. 31, No. 2, 181-213.

Maksimovic, M. (1989). On the residual shearing strength of clays. Geotechnique, Vol. 39, No. 2, 347-351.

Mandl, G., De Jone, L.N.J. and Maltha, A. (1977). Shear zones in granular material. Rock Mech., Vol. 9, 95-144.

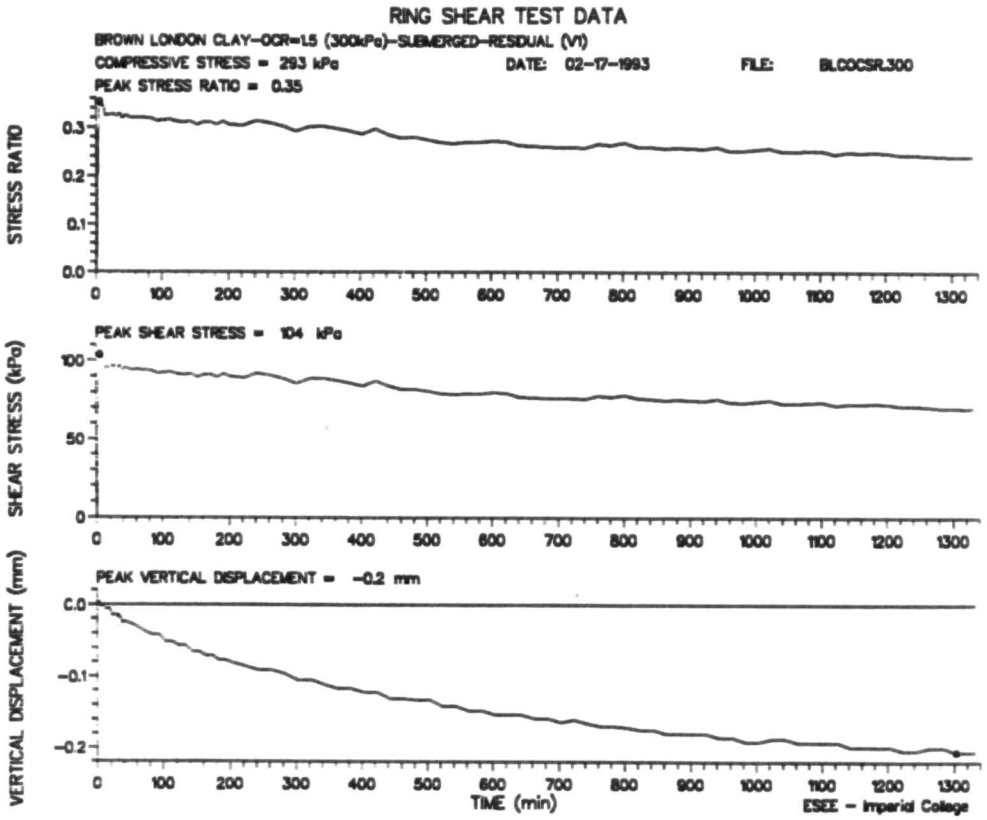


Fig. IV.12.1 Slow shearing test on over-consolidated, submerged brown London clay (300 kPa - OCR= 1.5)

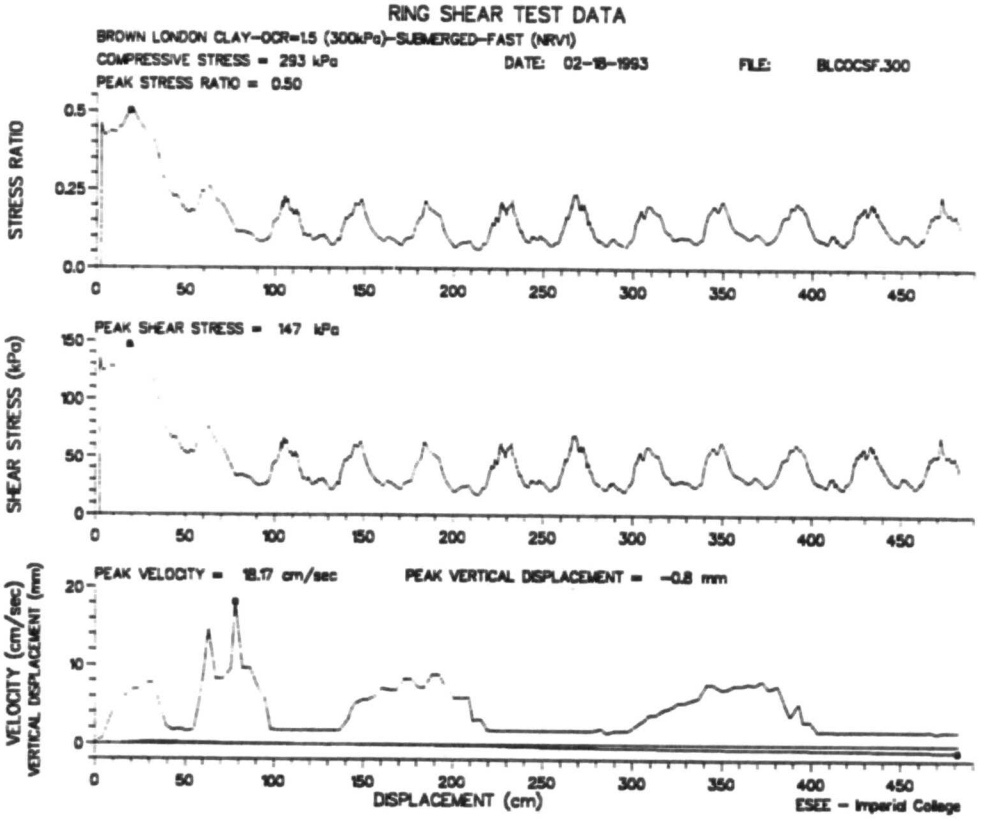


Fig. IV.12.2 Fast shearing test on over-consolidated, submerged brown London clay (300 kPa - OCR=1.5)

AI.2 Vertical Deformation of the Ring Shear Apparatus

It has been reported by previous researchers (Lupini, 1981, and Tika, 1989) that the ring shear apparatus deforms on the application of vertical load. Although this was thought to have insignificant effects on the quantities measured during a test, the results from measurements which were conducted by Garga (1970), Lupini (1981), and Tika (1989) are presented for reference purposes [Fig. AI.2.1].

AI.3 Calibration of the Motor Drive Units

The constant speed AC motors were calibrated by measuring, by means of the angular vernier scale, the displacement which the rotating part of the apparatus travels in one time unit. This procedure was repeated at several settings of the semi-automatic controlled motors and a set of curves was produced for each motor which enable the estimation of the rate of displacement [Fig. AI.3.1].

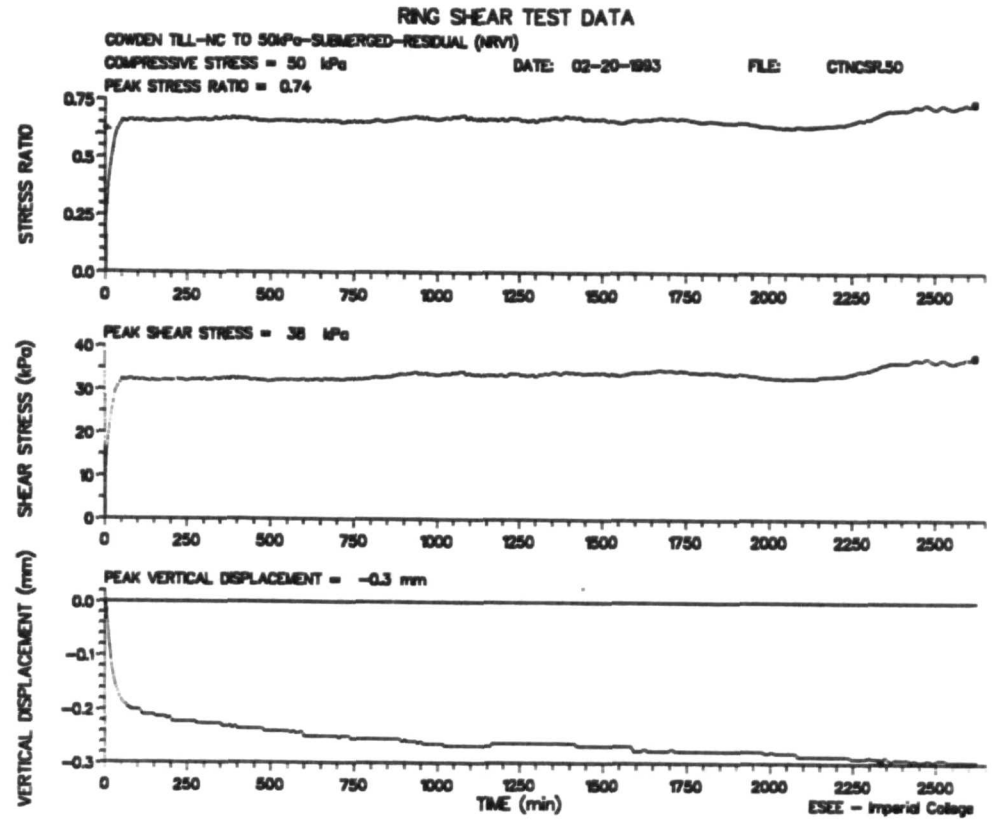


Fig. IV.17.1 Slow shearing test on normally consolidated, submerged Cowden till (50 kPa)

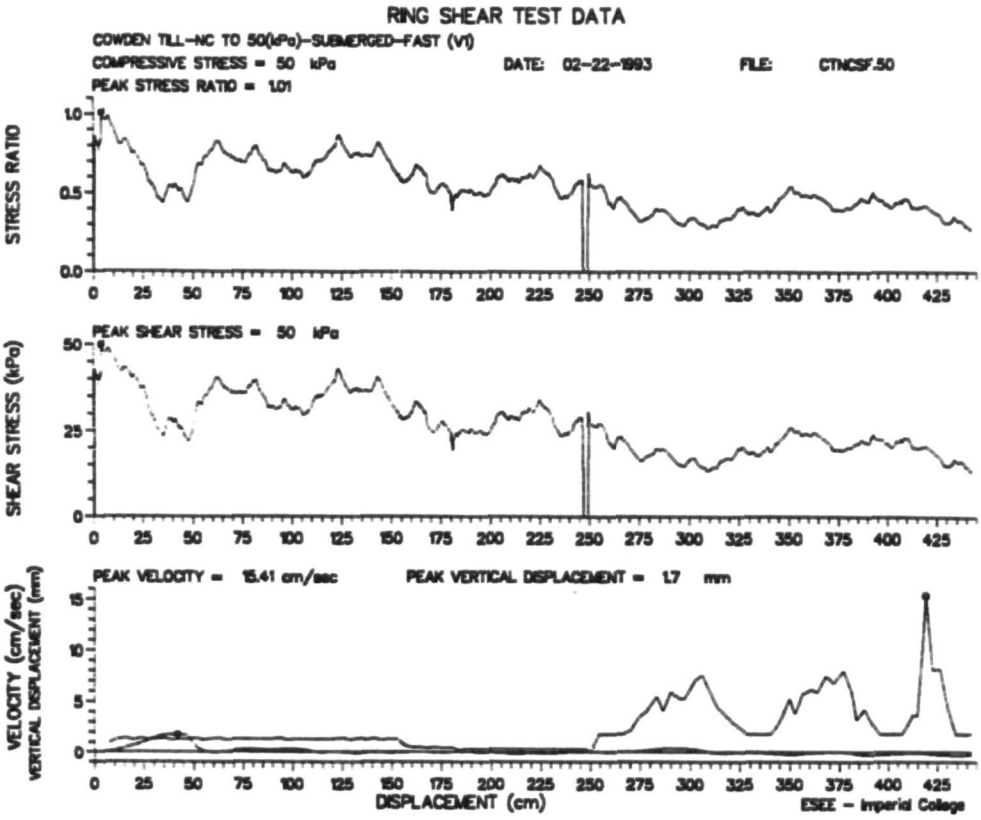


Fig. IV.17.2 Fast shearing test on normally consolidated, submerged Cowden till (50 kPa)

AII.5 "Graphdisplay" & "Printing"

These sub-routines are associated with the calculation and presentation of the results while the testing procedure is in progress. The program collects the signals transmitted by the transducers, converts them to physical parameters with the use of the calibration constants, and calculates the horizontal and vertical displacements, the rate of displacement, the shear and normal stresses, and the stress ratio using equations presented in Chapter 3. All results are saved into an ASCII file corresponding to each particular test. Simultaneously, the results are plotted on the PC screen so that the operator can have a better control of the rate of displacement and develop an immediate view regarding the behaviour of the sample which is being tested.

AII.6 Complementary Programs

Three more computer programs were written in order to provide a better management of the files containing the results from each test and apply the side and gap friction corrections. The file which contains the results from each test is retrieved, the operator is prompted to choose whether a correction should be applied, and if the answer is "yes" which correction file should be used. Subsequently the test results, corrected or not as the case may be, are transferred into a floppy disk and may be plotted. The program **"Ringdata"** deals with the "slow" tests and provides graphs in which the results are plotted versus *"Time of testing"*, whilst the program **"Ringfile"** deals with the "fast" tests and provides graphs in which the results are plotted versus the *"Shear displacement"*. Finally, the program **"Ringplot"** deals with all tests and enables the operator to view on the PC screen the test results, corrected or not, plotted in various ways *i.e.* stress ratio vs. velocity vs. displacement (three dimensional graph), stress ratio vs. velocity, stress ratio vs. displacement, or velocity vs. displacement.

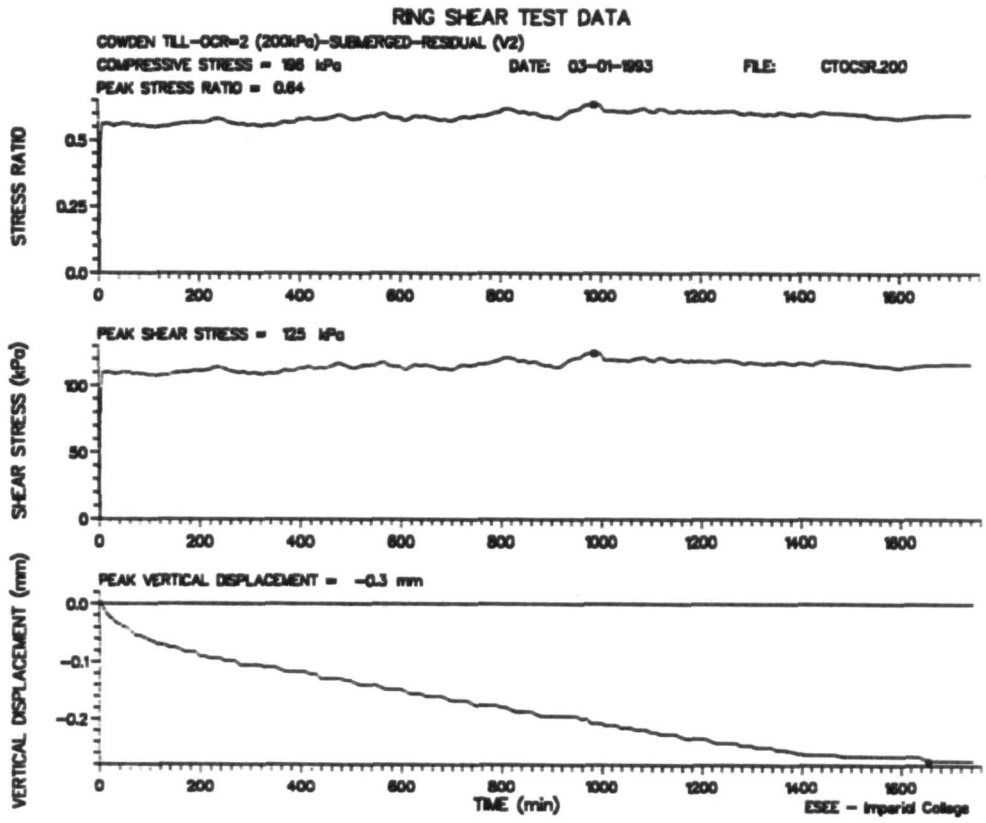


Fig. IV.22.1 Slow shearing test on over-consolidated, submerged Cowden till (200 kPa - OCR= 2)

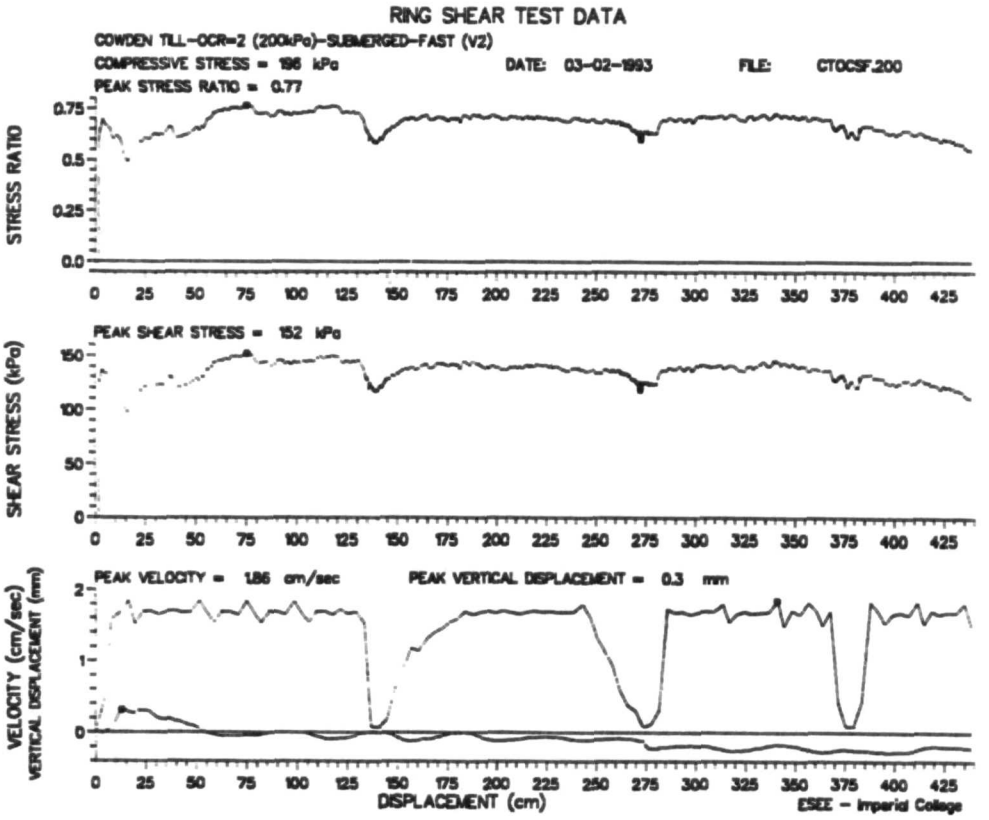


Fig. IV.22.2 Fast shearing test on over-consolidated, submerged Cowden till (200 kPa - OCR= 2)

On the first application of fast shearing, the sample contracted to 0.093 mm below its original height causing a decrease in shear strength to 17.9°. As shearing continued, the sample dilated to 0.71 mm above its original height and the strength dropped to 10.1° after a shear displacement of 40 cm (full sample rotation). The rate of displacement was consequently increased to 1.7 cm/sec and the undulating shear surface which developed had an amplitude of 0.62 mm after 70 cm of shear displacement, which decreased to 0.13 mm after 580 cm of shear displacement, and a wave-length of 40 cm. After the shear strength dropped below its residual value at the beginning of the fast shearing stage, it increased above that value and oscillated around 19.3° for the remainder of the test. No further effect of any change in the rate of displacement was observed after the first 45 cm of shear displacement. It should be noted that the periodic sharp increases of the measured strength observed in the stress ratio/displacement curve may be partly due to contamination of the rings which became more apparent at the lower normal stress levels.

TEST IV. 2

σ_n= 100 kPa

$\phi_{residual}$= 11.7°

$\phi_{threshold}$= 14°

(Figures IV.2.1-2)

The sample was sheared at 0.09 mm/min for 1360 mins to a displacement of 12.3 cm. After 0.2 mm of shearing it had reached its slow peak strength of 20.8° while continued shearing was associated with a decrease in the measured sample strength which reached its slow residual value of 11.4° after 3.6 cm of displacement. The 0.16 mm decrease in sample height associated with the first 3.6 cm of shearing should be attributed to a combination of particle reorientation and sample loss, where as the further 0.14 mm decrease in sample height before this slow shearing part of the test was terminated should be attributed solely on sample loss.

The rate of displacement was increased gradually in order to observe the effect which the rate of displacement gradient has on the response of the sample under fast shearing conditions. The rate of displacement was increased from 0.09 mm/min to 0.1 cm/sec during the first 7.5 cm of shearing and the strength of the sample remained constant at 14.0° without exhibiting a pronounced threshold strength. As the rate of displacement was increased gradually to 1.4 cm/sec during

with continued shearing while the sample appeared to be dilating. After 13.6 cm of shearing the sample height had increased by 0.094 mm, and the strength had decreased to its slow residual value. Further shearing at the same rate of displacement led to a decrease of the sample height due to soil loss. The measured strength had stabilised between 42.2° and 45.2° when the rate of displacement was decreased suddenly to 0.11 cm/sec after 152.2 cm of shearing. The sample was sheared at 0.11 cm/sec for a further 20 cm and then the rate of displacement was increased gradually to 1.63 cm/sec. However, any change in the rate of displacement, sudden or gradual, had no effect on the strength of the sample which varied between 42.2° and 45.2° until the test was terminated after 415 cm of shearing.

Normally Consolidated, Submerged

Test V.5

σ_n= 50 kPa

$\phi_{residual}$= 49.2°

$\phi_{threshold}$= 49.2°

(Figures V.5.1-2)

The sample was sheared at 0.09 mm/min and its strength reached its slow peak value of 50.2° after 9.9 mm of shearing. The sample height curve indicates that the sample initially contracted by 0.1, and then dilated by the same amount. The measured strength reached its slow residual value of 49.2° after 22.8 mm of shearing and this part of the test was terminated after a cumulative displacement of 165.3 mm.

The rate of displacement was increased suddenly to 1.2 cm/sec and the strength reached its threshold value of 49.2° at zero associated displacement. Further shearing at the same rate of displacement led to a continuous decrease in strength which appeared to stabilise between 41.7° and 46.3° while the sample appeared to be dilating by 0.1 mm. The rate of displacement was decreased to 0.14 cm/sec after 76.5 cm of shearing and the measured strength dropped to 40.4°. Further changes in the rate of displacement had no profound effect on the strength which remained fluctuating between 41.7° and 46.3°. However, after 161.6 cm of shearing, the rate of displacement was suddenly increased to 1.67 cm/sec and the measured strength increased immediately to 51.3°. Continuous shearing led to a decrease in strength which again stabilised between 40.4° and 46.3° and any further change of the rate of

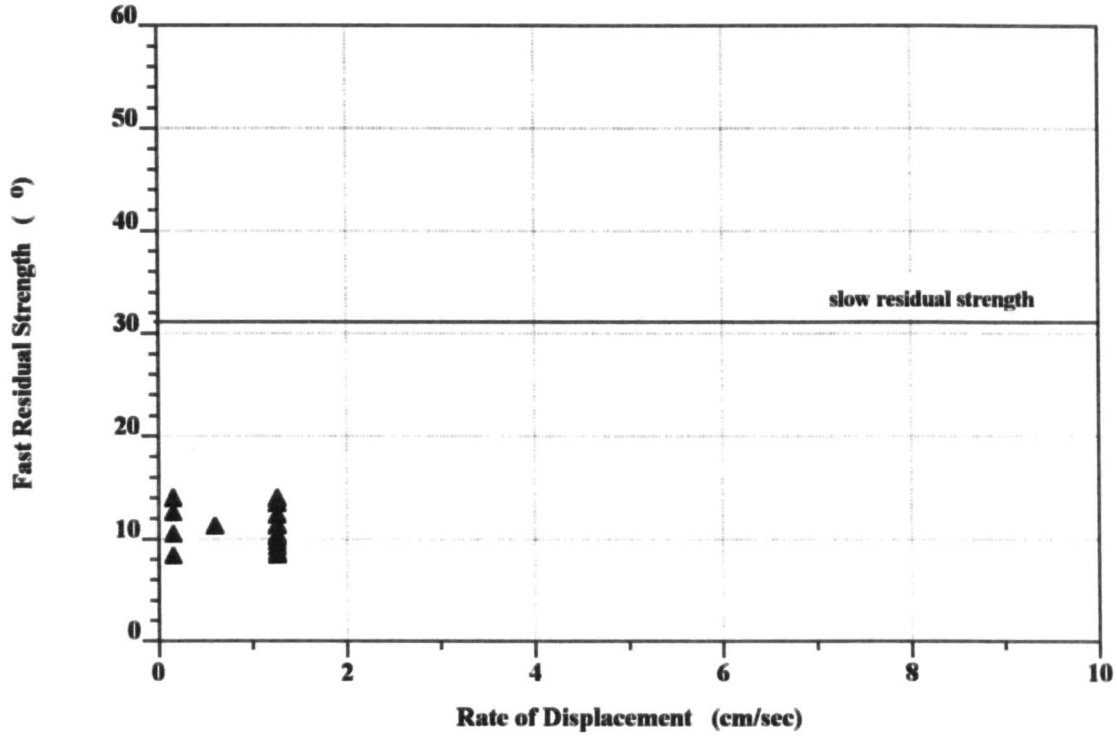


Fig. 6.3.49 Rate dependent characteristics of normally consolidated, non presheared, Cowden till (100 kPa)

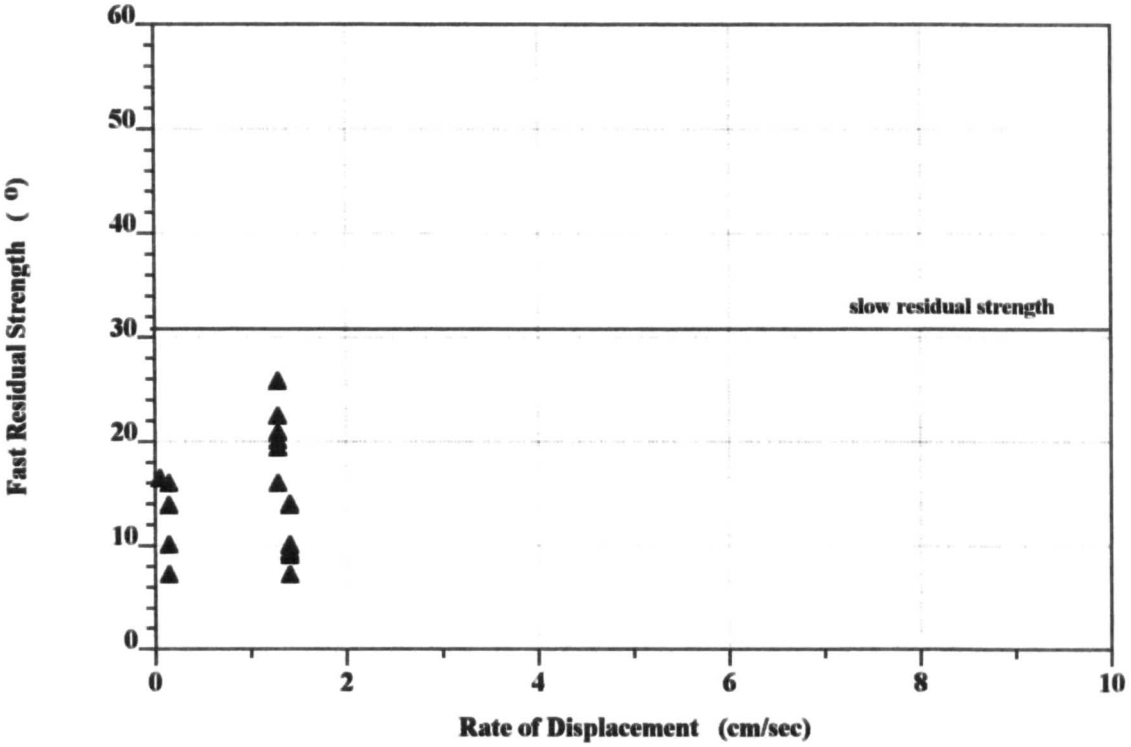


Fig. 6.3.50 Rate dependent characteristics of normally consolidated, non presheared, Cowden till (200 kPa)

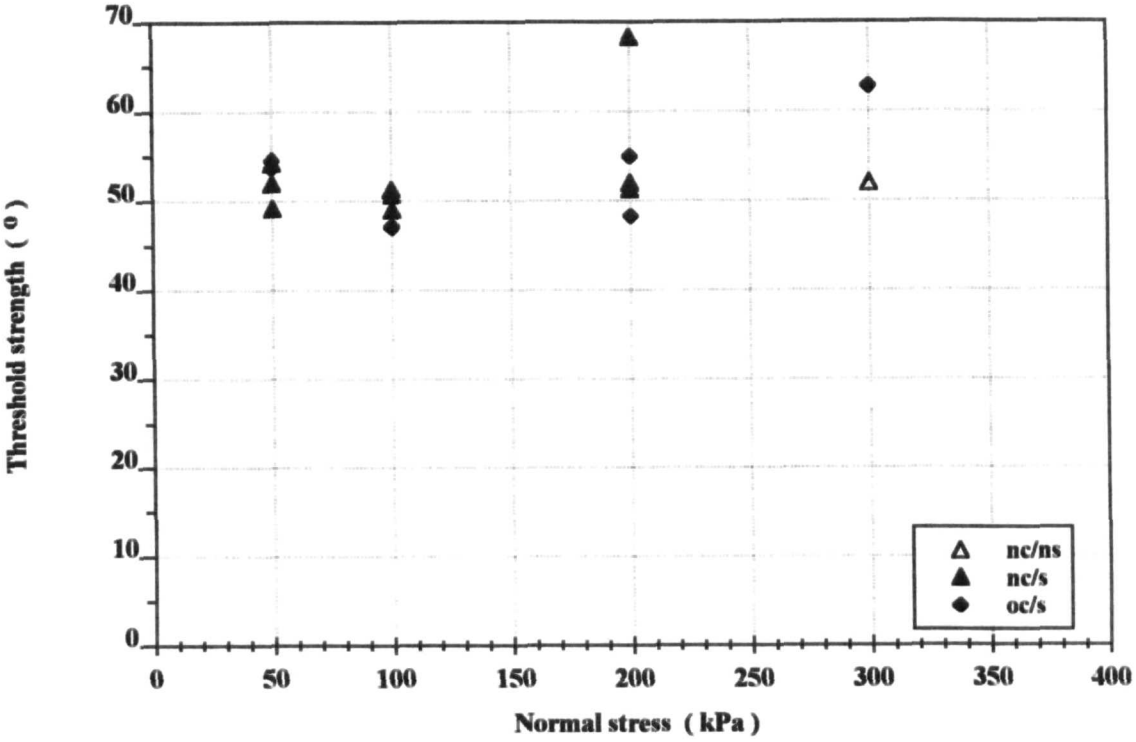


Fig. 6.4.1 Normal stress effect on threshold strength of pure silica samples

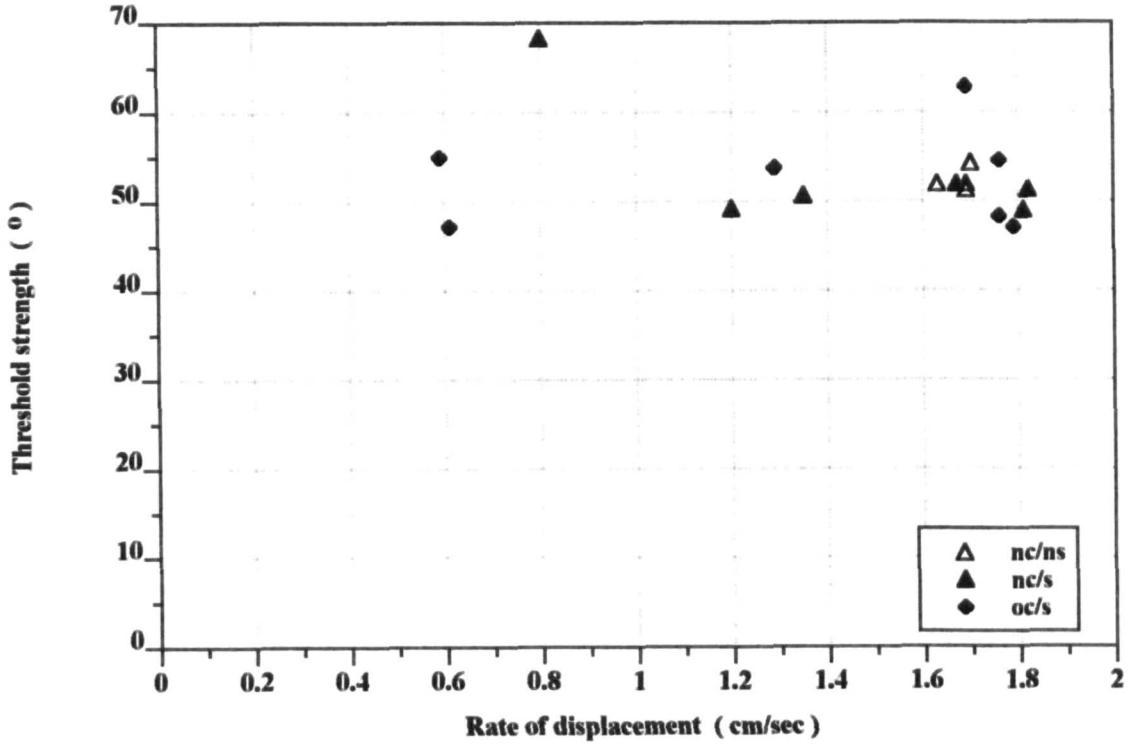


Fig. 6.4.2 Rate of displacement effect on threshold strength of pure silica samples

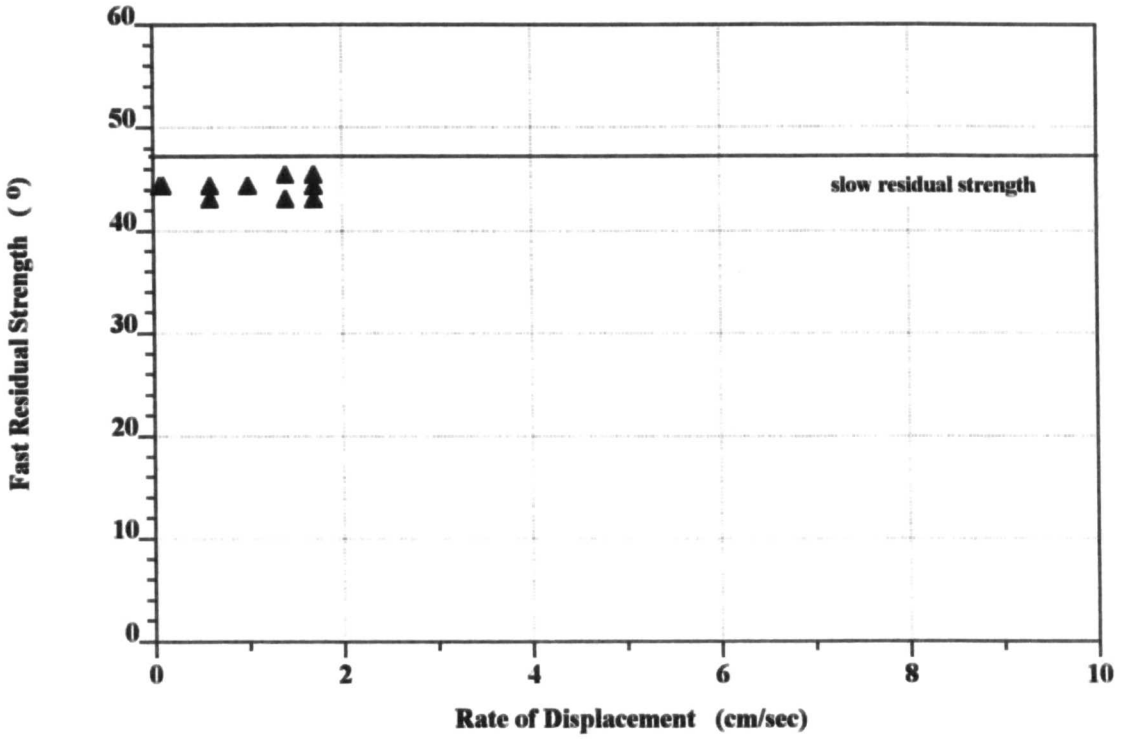


Fig. 6.4.11 Rate dependent characteristics of normally consolidated, non-submerged, pure silica sample(50 kPa)

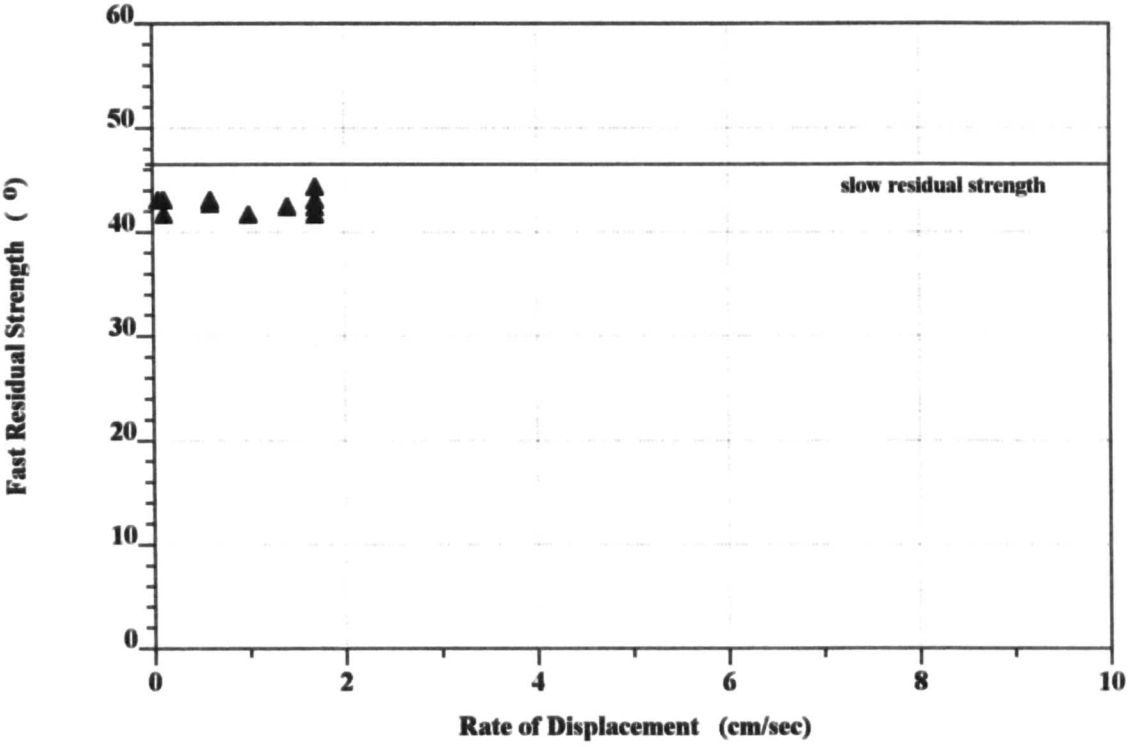


Fig. 6.4.12 Rate dependent characteristics of normally consolidated, non-submerged, pure silica sample(100 kPa)

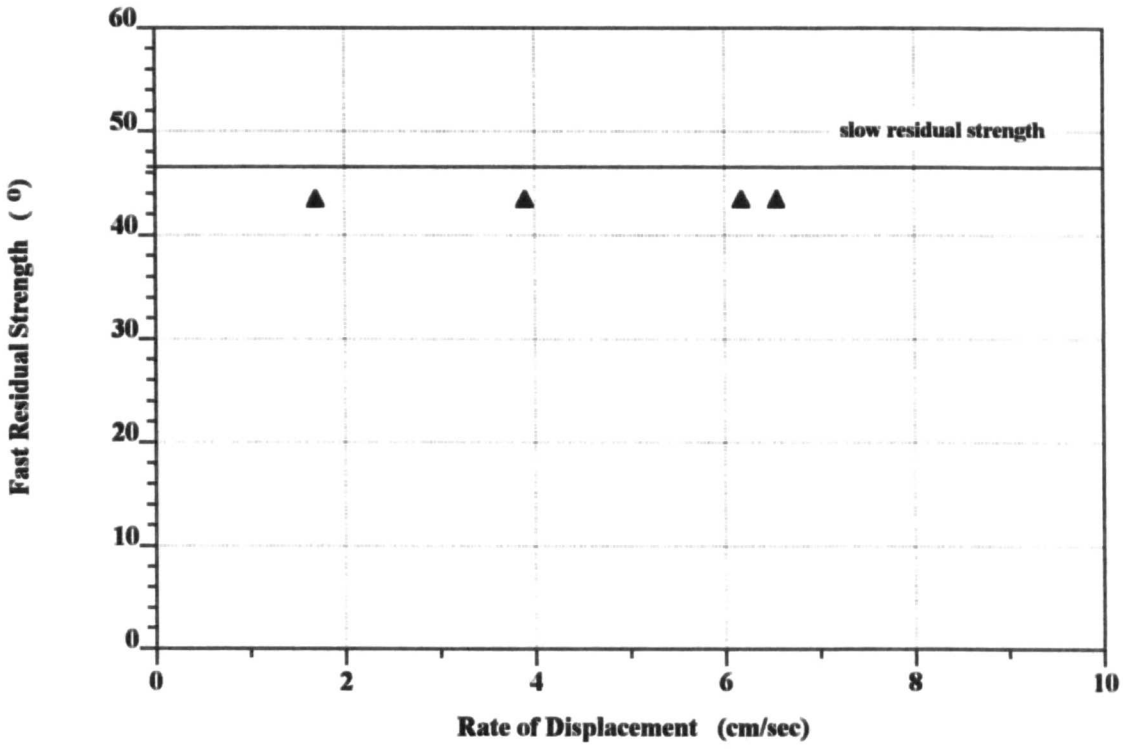


Fig. 6.4.21 Rate dependent characteristics of over-consolidated, submerged pure silica sample (250 kPa)

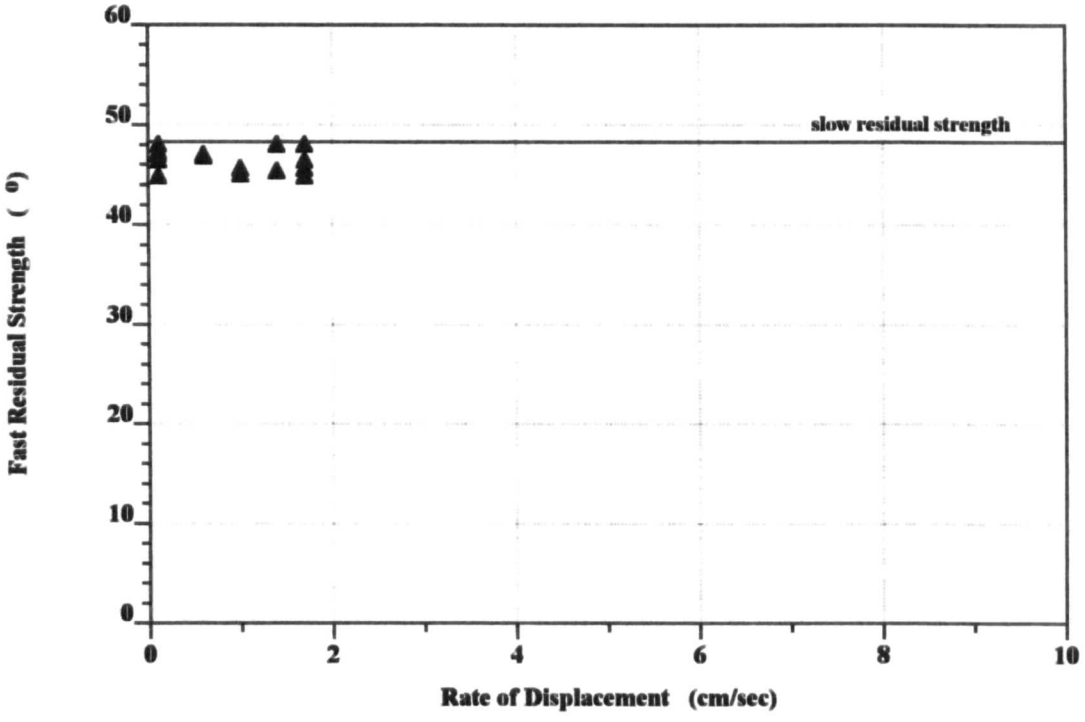


Fig. 6.4.22 Rate dependent characteristics of normally consolidated, non-submerged, sandy-silt sample (50 kPa)

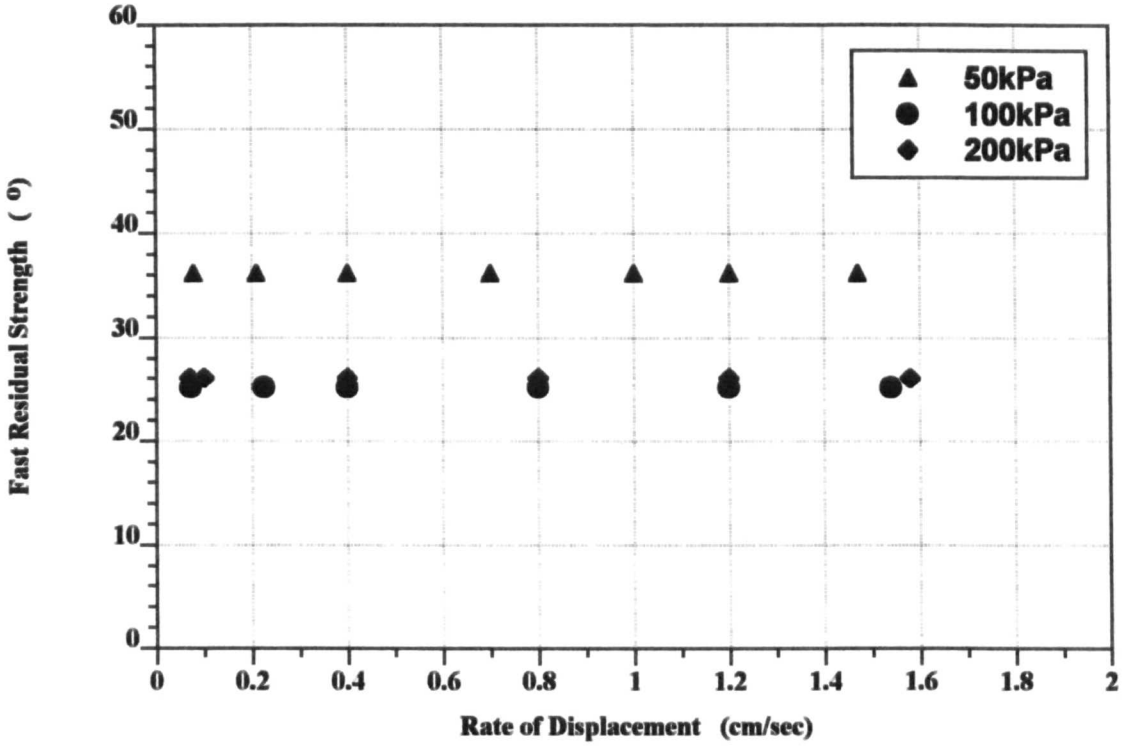


Fig. 6.4.31 Rate dependent characteristics of normally consolidated, dry, clean-coarse sand sample (50-200kPa)

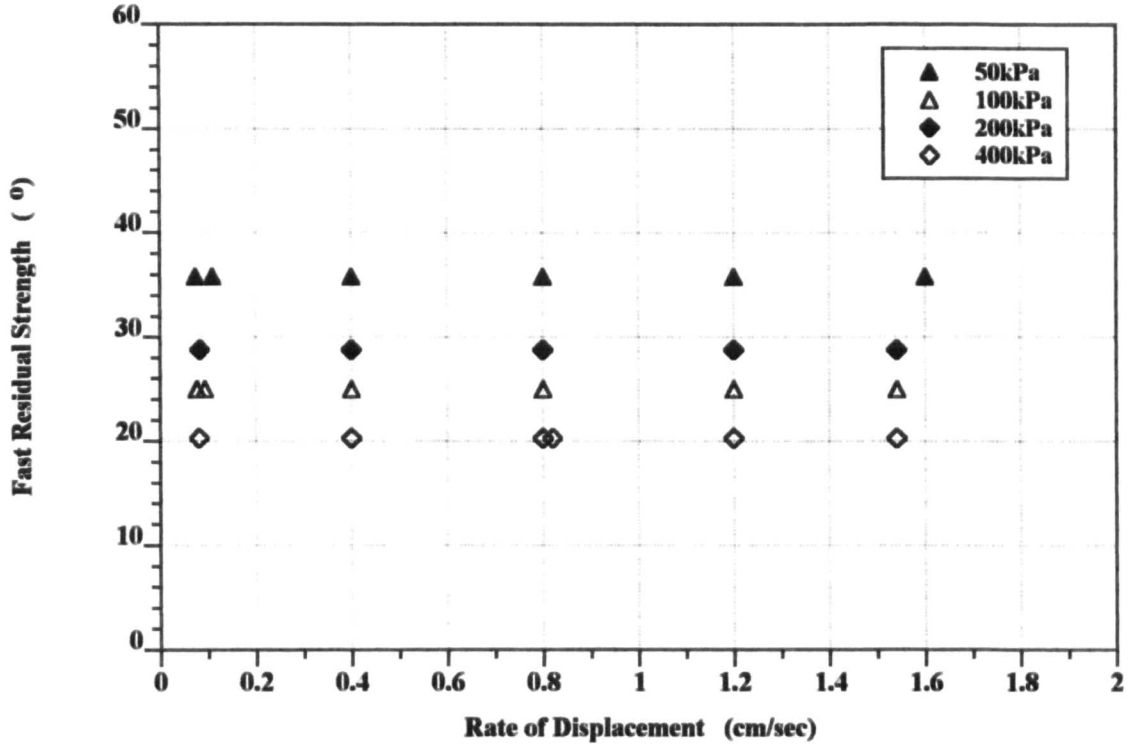


Fig. 6.4.32 Rate dependent characteristics of normally consolidated, moist, clean-coarse sand sample (50-400kPa)

In the case of plastic soil samples which exhibit a negative rate effect the fast residual strength is not rate dependent, is not affected by the increase in normal stress, and may be as little as 78 % lower than the slow residual strength. Furthermore, the drop in strength from slow to fast residual levels appears to be directly related to the amplitude of the undulating shear surface. It is postulated that in cases where the negative rate effect is associated with increased pore pressures it should be viewed as a result of the shearing mechanism involved and not as a intrinsic soil property.

Two hybrid types of behaviour are also observed. *Type Ia* behaviour [Fig. 6.5.1.b] occurs with plastic soils which exhibit a positive rate effect and their post threshold strength falls below slow residual. This behaviour is associated with the magnitude of the immediate dilation of the sample observed upon the first application of fast rates of displacement. *Type IIb* behaviour [Fig. 6.5.1.b] occurs with submerged plastic soils, is a combination of behaviour types Ia and II, and is observed when the amplitude of the undulating shear surface increases as a fast test progresses. Both hybrid types of behaviour are considered to depend on the circumstances involved in ring shear testing.

Finally, it is demonstrated that the reversing of the shearing direction does not appear to have a significant effect on the behaviour of plastic soils nor on their fast residual strength.

It is concluded that, in the absence of an undulating shear surface, the fast residual strength of all remoulded plastic soils may be as much as 250 % higher than their slow residual strength, and the fast residual strength of all non-plastic soils may be as little as 10 % lower than their slow residual strength. In the presence of an undulating shear surface, the fast residual strength of all submerged plastic soils may be as little as 78 % lower than their slow residual strength. Due to the fact that the behaviour of plastic soils during fast shearing is related to the shape of the shear surface, it is difficult to speculate, using the existing results, on the response of a landslide upon earthquake loading. Although further research on this subject is required, it is believed that the results from this investigation provide a better understanding of the factors which control the behaviour of soils during fast shearing and a more comprehensive picture of that behaviour.

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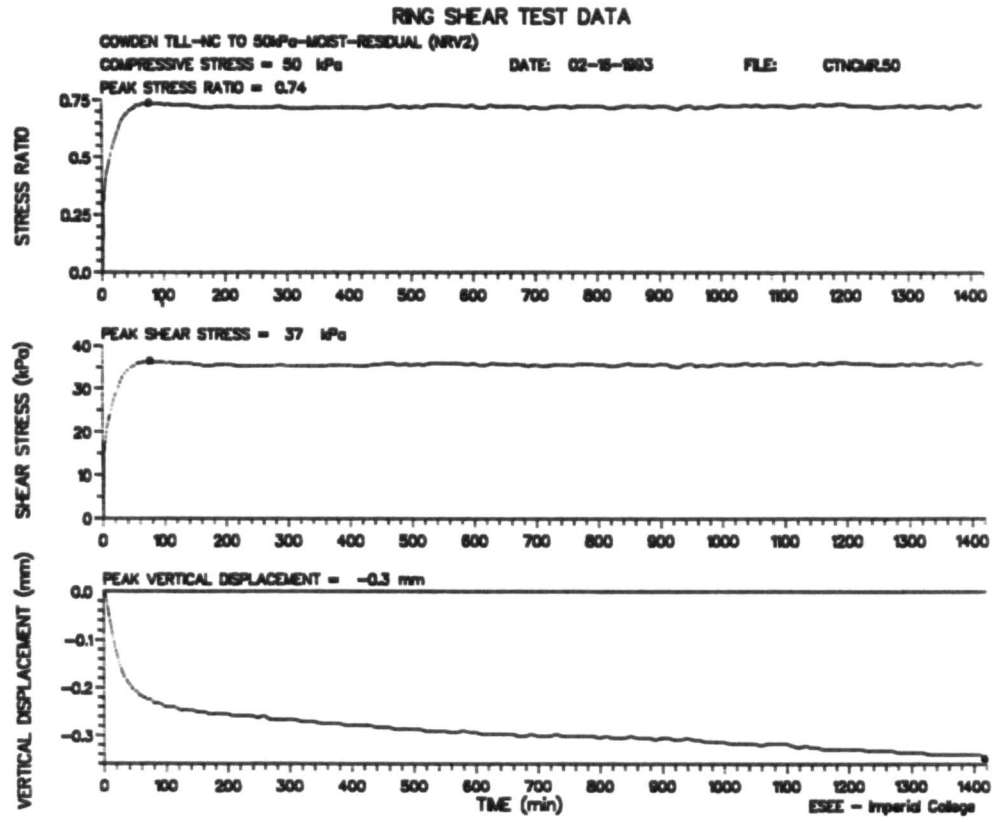


Fig. IV.13.1 Slow shearing test on normally consolidated, non-submerged Cowden till (50 kPa)

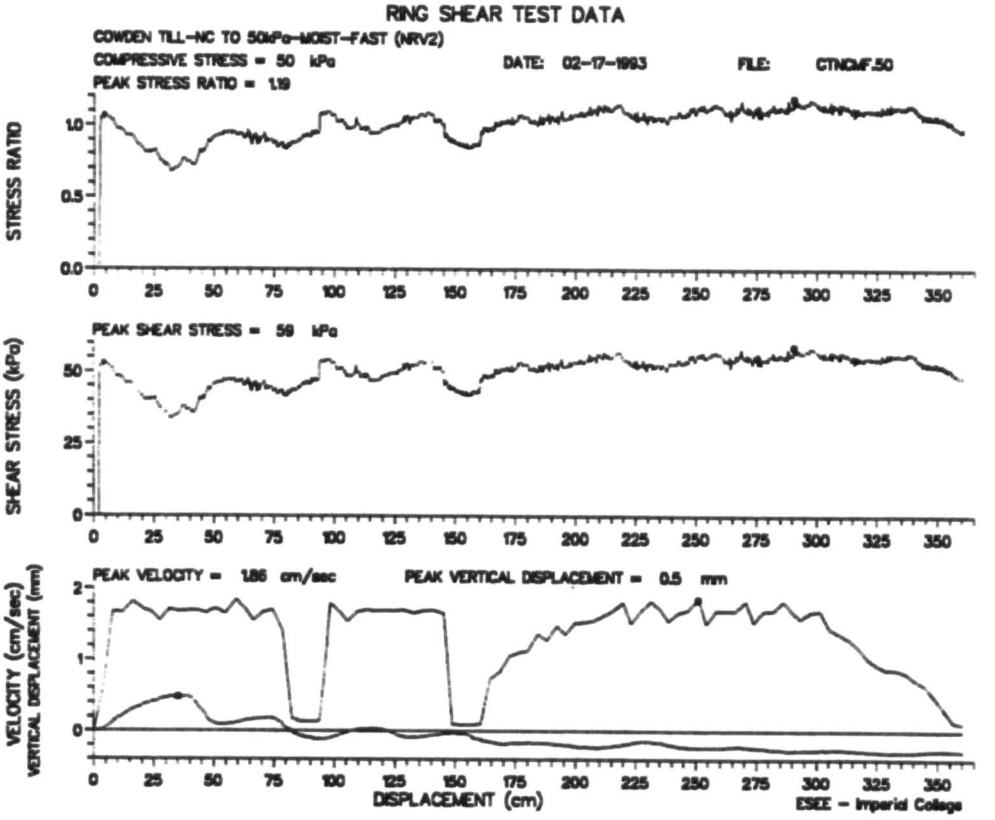


Fig. IV.13.2 Fast shearing test on normally consolidated, non-submerged Cowden till (50 kPa)

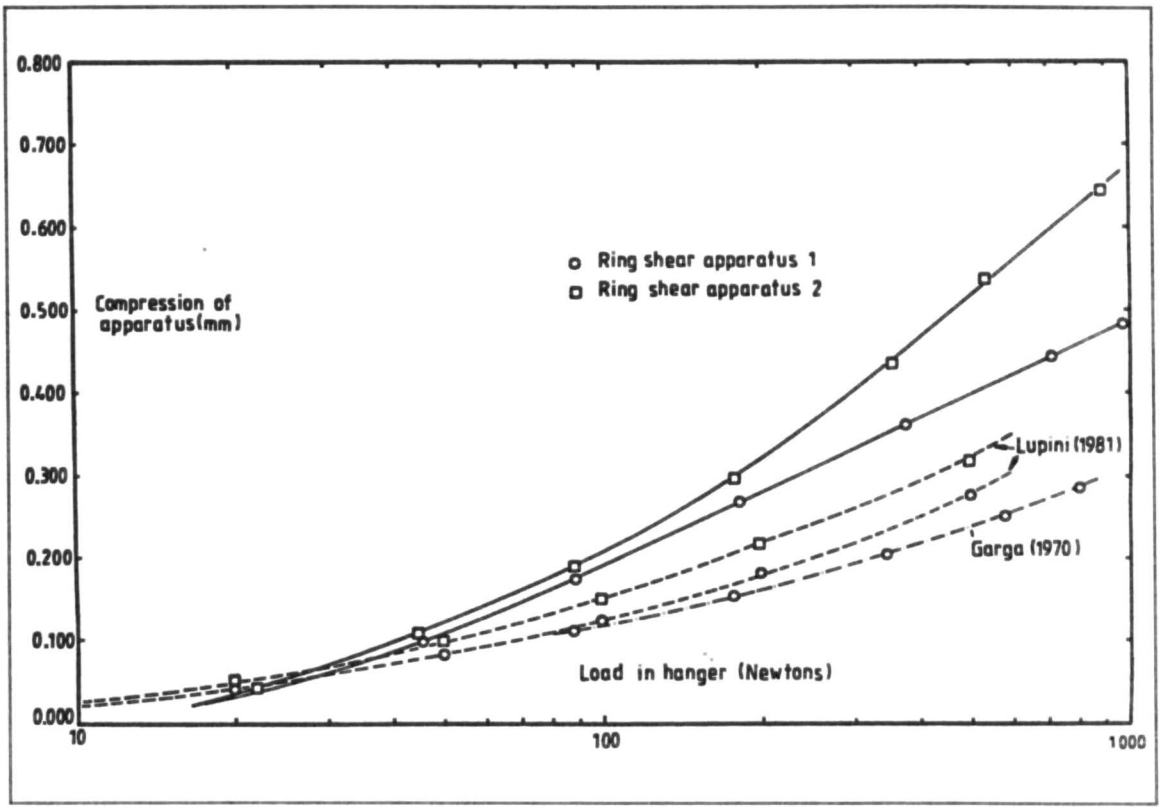


Fig. AI.2.1 Vertical Deformation of the Ring Shear Apparatus (Tika, 1989)

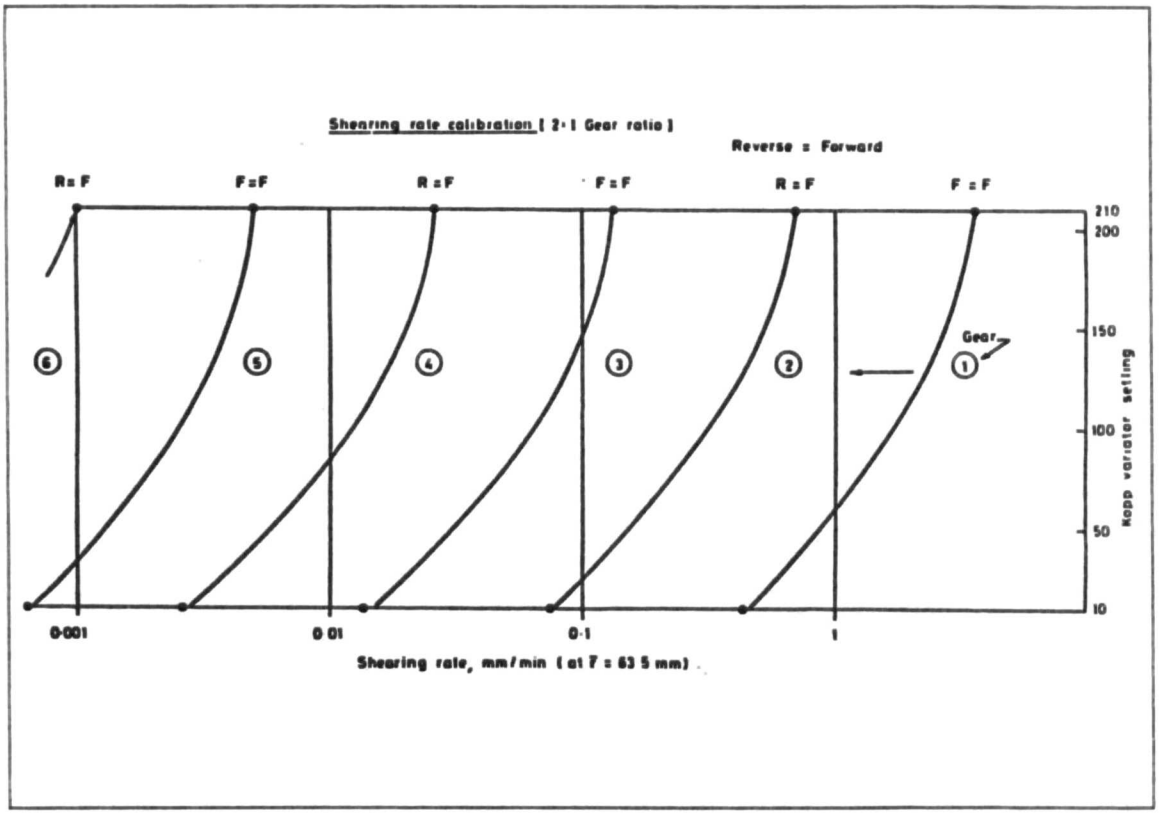


Fig. AI.3.1 Calibration of Motor M1-Ring Shear 1

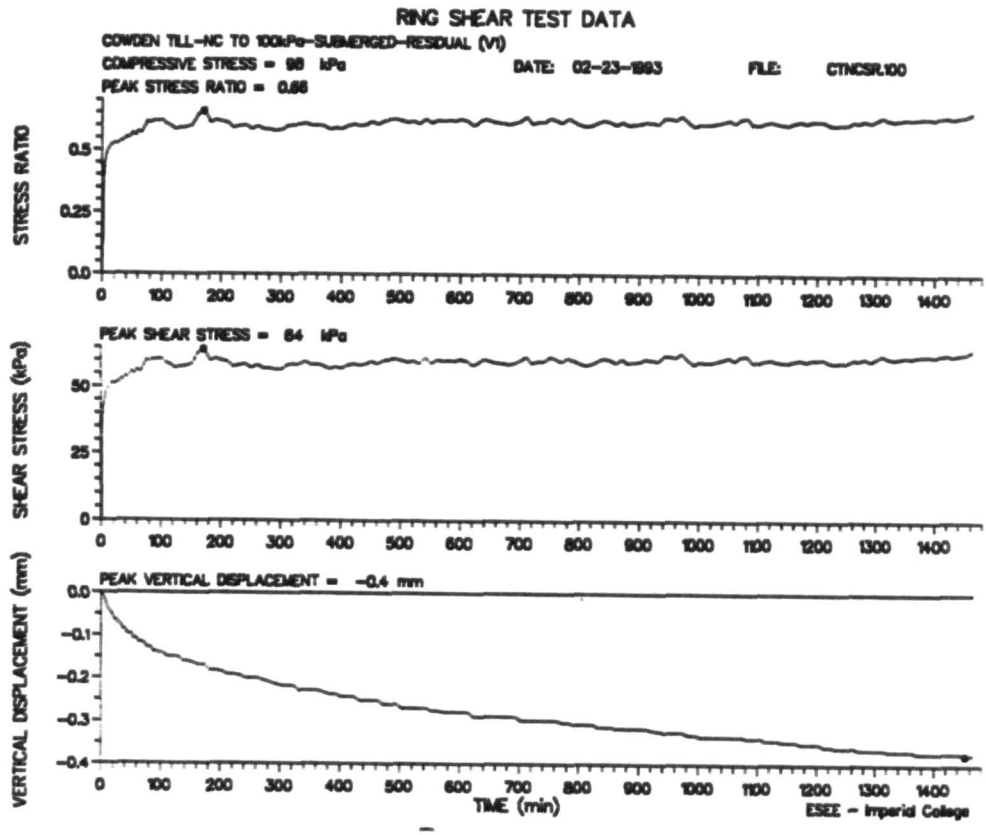


Fig. IV.18.1 Slow shearing test on normally consolidated, submerged Cowden till (100 kPa)

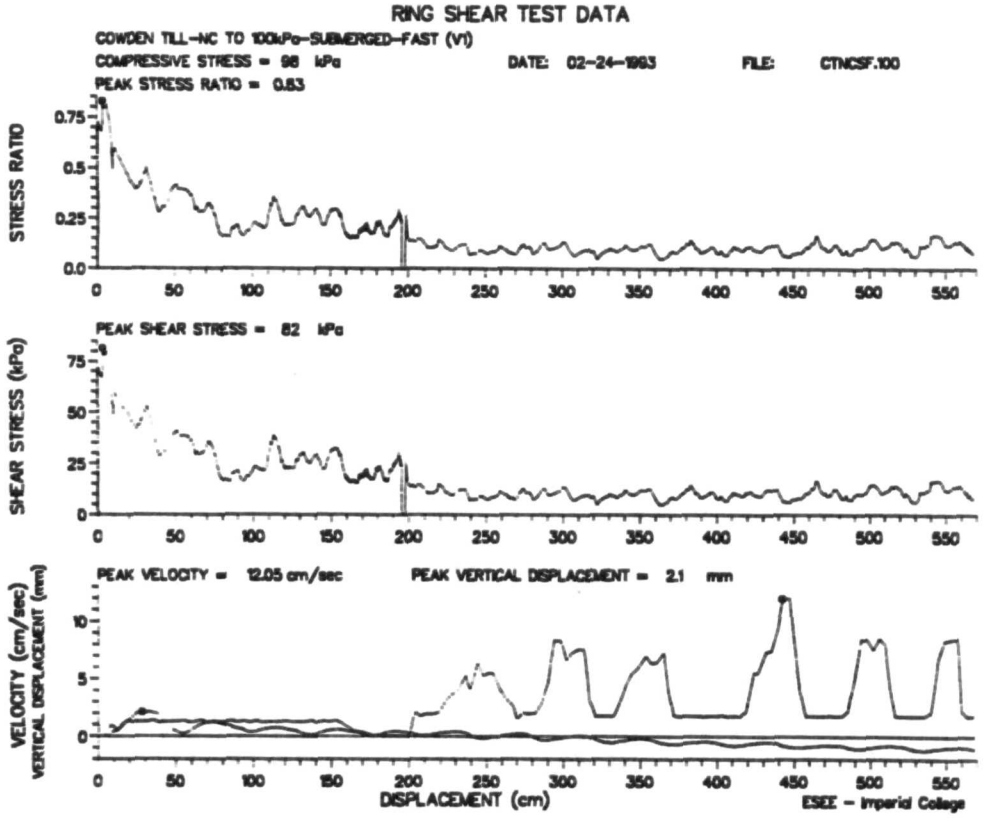


Fig. IV.18.2 Fast shearing test on normally consolidated, submerged Cowden till (100 kPa)

APPENDIX III

CALCULATIONS AND MEASUREMENT ERRORS

AIII.1 Introduction

The equations which have been employed for the calculation of the results of ring shear tests have been presented in Chapter 3. Furthermore, the side and gap friction corrections which may be applied to these results, when the new rings are used, have also been addressed in Chapter 4. Tika (1989) presented a thorough investigation on the errors which may be associated with the results of ring shear tests and indicated that, apart from the two previously mentioned sources of error (gap and side friction), inaccuracies of the measurements should also be considered. The following arguments are presented for reference purposes.

AIII.2 Shear Stress Measurement Accuracy

Tika (1989) suggested that the difference between the apparent shear stress measured by the load cells (F_A) and the true shear stress applied on them (F) may introduce an error (ε) in the calculations of the stress ratio which has to be considered. It was suggested in Chapter 3 that the stress ratio in ring shear tests can be calculated using the following equation:

$$(\tau/\sigma_n) = 3.7298 \times \frac{F_1 + F_2}{W_n} \qquad \text{(AIII.1).}$$

Furthermore, if one assumes that the shear stresses which are measured by the two load cells are, for all practical purposes, equal, the previous equation may be re-written as follows:

$$(\tau/\sigma_n)_A = 7.4596 \times \frac{F_A}{W_n}, \text{ for the apparent stress ratio} \qquad \text{(AIII.2)}$$

and

$$\tau/\sigma_n = 7.4596 \times \frac{F}{W_n}, \text{ for the true stress ratio} \qquad \text{(AIII.3).}$$

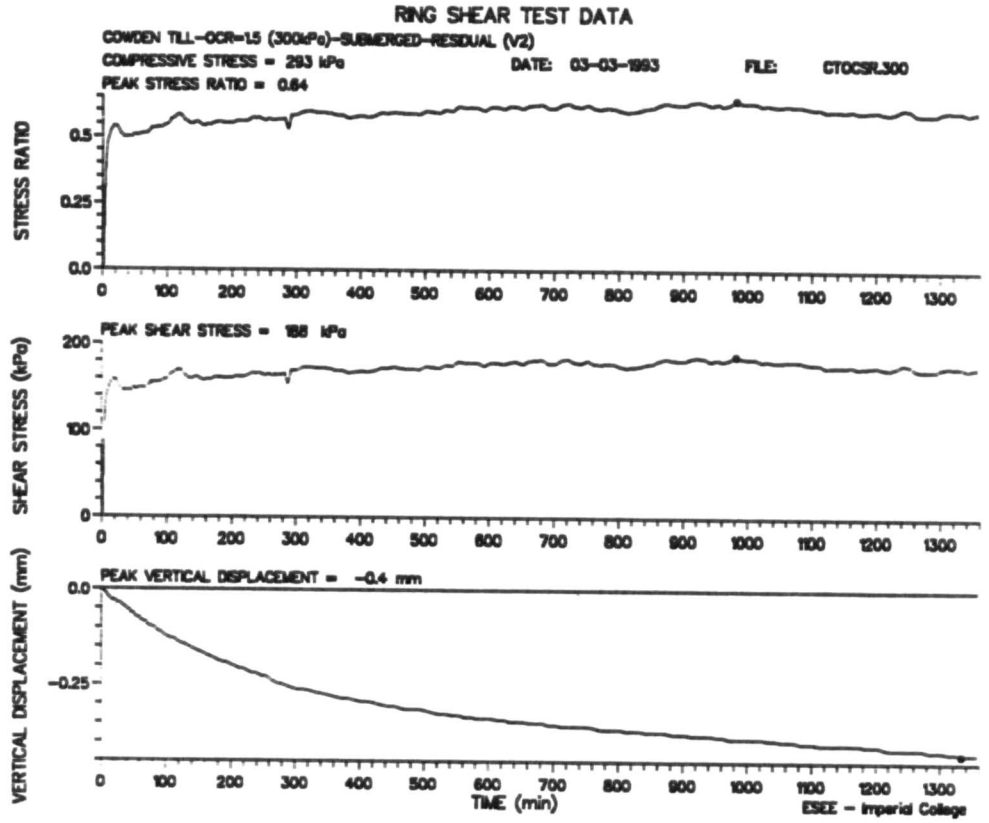


Fig. IV.23.1 Slow shearing test on over-consolidated, submerged Cowden till (300 kPa - OCR= 1.5)

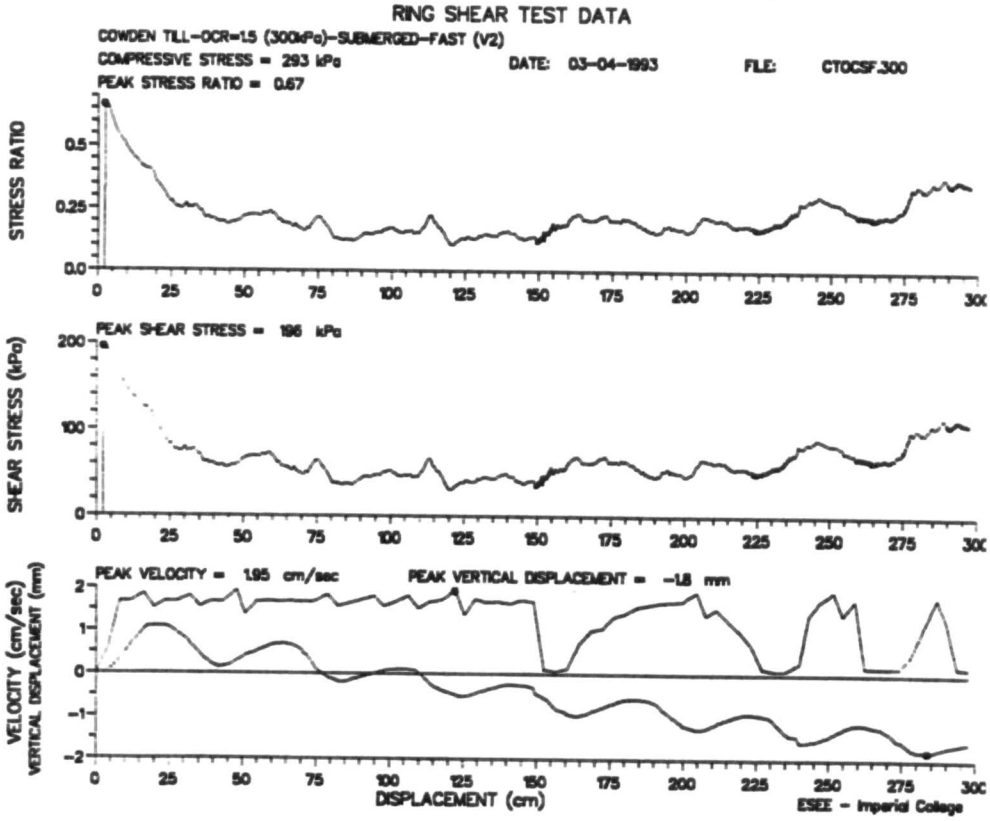


Fig. IV.23.2 Fast shearing test on over-consolidated, submerged Cowden till (300 kPa - OCR= 1.5)

the next 10 cm of shearing, the strength increased to 22.9°. While the rate of displacement remained at 1.4 cm/sec, the fast residual strength of the sample stabilised above its slow residual value of 22.5°. When the rate of displacement was decreased to 0.12 cm/sec, the fast residual strength of the sample dropped to 16.1°. The sample exhibited positive rate characteristics when the rate of displacement was increased or decreased gradually. When the rate of displacement was increased suddenly, the fast residual strength of the sample also increased suddenly. Also, it should be noted that from the shape of the sample height curve and the stress ratio/displacement curve it could be concluded that the shear surface was planar and that the contamination of the rings due to soil lost through the gap did not influence the measured strength of the sample.

TEST IV.3

σ_n= 200 kPa

$\phi_{residual}$= 11.3°

$\phi_{threshold}$= 20.8°

(Figures IV.3.1-2)

The sample was sheared at 0.09 mm/min for 1660 mins and reached its slow residual strength after a displacement of 13.3 cm. After 0.45 mm of slow shearing the sample had reached its slow peak value of 16.2° and the observed 0.4 mm decrease in sample height was attributed to combination of particle reorientation and soil loss.

After the residual state had been established, the rate of displacement was increased to 0.07 cm/sec for the first 3.2 cm of fast shearing and the strength of the sample increased to 11.2°, just above its slow residual value. Subsequently, the rate of displacement was increased suddenly to 1.29 cm/sec, and the shear strength of the sample reached its threshold value of 20.8° at zero associated shear displacement. Continued shearing at 1.3 cm/sec lead to a decrease in measured strength to 11.5°, as the sample dilated 0.42 mm above its height at the end of the slow shearing stage of this test. After 40 cm of shearing, while the rate of displacement was 1.3 cm/sec, the fast residual strength of the sample stabilised at 15.3°. The shape of the sample height curve suggested the existence of an irregular rather than an undulating shear surface which flattened-out after 158 cm of shear displacement. Furthermore, the shape of the stress ratio/displacement curve suggested some contamination of the rings which did not, however, seem to influence the measured strength of the sample. Positive

displacement between 1.67 cm/sec and 10 cm/sec, gradual or sudden, had no effect on it. The test was terminated after 590 cm of shearing.

Test V.6

σ_n= 100 kPa

$\phi_{residual}$= 47.2°

$\phi_{threshold}$= 50.7°

(Figure V.6.1-2)

The sample appeared to be dilating when sheared at 0.09 mm/min. After 3.4 cm of shearing the strength had reached its slow peak value of 51.3° and the sample height had increased by 0.043 mm. Further shearing led to a continuous decrease in measured sample strength until it reached its slow residual value of 47.2° after 119.7 mm. At that point the sample height had increased by a further 0.012 mm. This slow shearing part of the test was terminated after a displacement of 134.9 mm.

The rate of displacement was suddenly increased to 1.35 cm/sec and the measured strength immediately reached its threshold value of 50.7°. The sample appeared to be dilating while shearing continued at the same rate of displacement and the measured strength decreased to 45.8° after 6.1 cm. Due to soil loss the sample height decreased while the strength of the sample fluctuated between 40.4° and 34.5°. The rate of displacement was decreased to 0.135 cm/sec after 126 cm of shearing and then gradually increased to 1.35 cm/sec but those changes had no effect on the strength of the sample. After 235.9 cm of shearing the rate of displacement was increased suddenly to 1.81 cm/sec and the measured sample strength reached the value of 49.0° at zero associated displacement before it stabilised again within a range between 40.4° and 34.5°. The rate of displacement was varied between 1.81 cm/sec and 8.73 cm/sec, 10.0 cm/sec and 7.43 cm/sec, gradually and suddenly, before the test was terminated, but none of these changes had any effect on the strength of the sample. The test was terminated after 730 cm of shearing.

Test V.7

σ_n= 200 kPa

Kaolin:

Normally Consolidated, Submerged

Test VI.13

σ_n= 200 kPa

$\phi_{residual}$= 15.7°

$\phi_{threshold}$= 25.5°

(Figures VI.13.1-2)

The sample was sheared at 0.09 mm/min, and the measured strength reached its slow peak value of 23.7° after 3.8 mm of shearing while the sample height had decreased by 0.3 mm. Further shearing was associated with a decrease in measured strength which reached its slow residual value of 15.7° after 28.5 mm of shearing and while the sample height had decreased by an additional 0.12 mm. This slow shearing part of the test was terminated after the sample had been sheared for 140.6 mm and the increase in measured strength which was observed towards the end of this part should be attributed to the development of an undulating shear surface.

The rate of displacement was increased suddenly to 1.28 cm/sec and the measured strength reached immediately its threshold value of 25.5°. Further shearing at the same rate of displacement was associated with an immediate drop in strength to 14.6°. While shearing at 1.28 cm/sec continued, the shape of the sample height curve indicated that an undulating shear surface was developing with an initial amplitude of 0.48 mm. After the sample had been sheared for 40.7 cm its measured strength had reached the level of 24.3° but this was be partly attributed to internal ring friction. Nevertheless, the strength of the sample continued its increasing trend until the sample had been sheared for 234 cm, when the rate of displacement was decreased from 1.28 cm/sec to 0.174 cm/sec, and then to 0.087 cm/sec. At that point the measured strength had increased to 23.4° while the shape of the sample height curve indicated that the undulating shear surface had been flattened with shearing. The sample exhibited positive rate characteristics as the rate of displacement was decreased and the strength dropped from 23.4° at 1.28 cm/sec, to 18.6° at 0.174 cm/sec, and to 17.1° at 0.087 cm/sec. After the sample had been sheared for 250.5 cm the rate of displacement was increased to 1.28 cm/sec and the strength of the sample initially increased to 25.2°, and then stabilised at 21.6°. After 451.7 cm of shearing the rate of displacement was increased suddenly to 1.74 cm/sec and the