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RESIDUAL STRENGTH OF CLAY AT LOW NORMAL STRESSES

by

JANAN TOMA ANAYI, BSc, H.Diploma, MSc

A Doctoral Thesis submitted in partial fulfilment of the requirements for the Award of the Degree of Doctor of Philosophy of Loughborough University of Technology

June 1990

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DEDICATION

To the memory of my father Toma Anayi.
My wife Eitzaz.
My daughters Dalia and Maria
and
My mother Mary Anayi on her 70th birthday

* * * * *
SYNOPSIS

The residual strength of three different natural soils, lower Lias clay, London clay and a clay from Baghdad, Iraq were investigated using the shear box and a modified Bromhead ring shear apparatus.

The use of fast forward and backward shearing to generate a failure surface followed by slow shearing for an appropriate displacement, was found to be the best technique of measuring the residual strength of reconstituted shear box samples. This is due to time saving and the reduced possibility of producing an undulating shear surface, which may lead to higher residual strength values.

The Bromhead ring shear apparatus was modified by adding vanes to the top and bottom platens in order to produce shearing on flat surfaces at the mid-height of the sample with no significant side friction. The modified apparatus proved to be a very suitable tool for measuring the residual strength of clay samples and composite samples of clay and concrete as it is fast, accurate and simple.

The shape of the residual failure envelopes obtained are curved. The curvature is most pronounced below a critical normal stress of 150 to 250 kPa, depending on the type of clay and the loading sequence applied during testing whether it is loading or unloading. Test results were found to be highly affected by experimental errors at normal stresses below the critical normal stress.
The shape and degree of curvature of the residual envelope depends entirely on the shape of the dominant clay mineral existing in the soil under examination, while the residual strength of clay is governed by the strength of the dominant clay mineral.

Scanning electron microscope photographs showed that for clays exhibiting sliding mode of shearing, perfect horizontal particle orientation is attained at normal stresses in excess of 200 to 250 kPa.
ACKNOWLEDGEMENTS

The research described in this thesis was carried out in the field of Soil Mechanics in the Civil Engineering Department headed by Professor R McCaffer. The research has been carried out under the supervision of Dr J Boyce and C Rogers and directed by Dr R Allwood.

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The Ministry of Higher Education, Iraq for sponsoring this research.

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Deep thanks are extended also to my wife's family, especially my father-in-law Aziz Majeed for his generous help and encouragement.

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Finally there are no words to express my gratitude to my wife Eitzaz for her continuous support and encouragement and to my daughters Dalia and Maria, whom, I hope, will be much better Soil Engineers than their father! Dalia has made a good starting point as she has learned to write (Soil Mechanics) although she is not yet six years old.
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NOTATION

C   = cohesion intercept
C'  = effective cohesion intercept
Cr' = effective residual cohesion intercept
C_r = compression cycle of the reversible shear box
CF  = clay fraction
d  = particle diameter
D   = disturbed sample
e  = void ratio
F   = fast rate of shearing
G_s = specific gravity
I_b = brittleness index
Ip  = PI = plasticity index
LI  = liquidity index
P_cr = critical normal stress beyond which residual strength envelope becomes a straight line
R   = residual factor
R_l = local residual factor
So4% = percentage of soluble sulphate content
t_90 = time of 90% consolidation
t_100 = time of 100% consolidation
T   = tension cycle of the reversible shear box
tan <P = effective coefficient of friction
tan <P_r = effective residual coefficient of friction
tan <P_u = interparticle coefficient of friction
S   = slow rate of shearing
u   = pore water pressure
U_n = undisturbed sample
W   = water content
W_L = LL = liquid limit
W_p = PL = plastic limit
ΔV/V = volume change
\( \tau \) = \( \tau_f \) = shear stress
\( \tau_p \) = peak shear stress
\( \tau_r \) = residual shear stress
\( \sigma_n \) = normal stress
\( \sigma_n' \) = effective normal stress
\( \tau/\sigma_n' = \mu \) = effective stress ratio, coefficient of friction
\( \tau_r/\sigma_n' \) = effective residual stress ratio, residual coefficient of friction
\( \phi' \) = effective friction angle of shearing resistance
\( \phi_r' \) = residual friction angle of shearing resistance

**Note:** Those symbols which do not appear here are defined where they appear in the text.
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1.1 THE SHEAR STRENGTH OF COHESIVE SOILS

The shear strength of clay is the most important aspect of geotechnical engineering. The bearing capacity of shallow or deep foundations, slope stability, and a retaining wall design, for example, all depend upon shear strength of the clay.

When a soil is subjected to shear strain, the shear resistance steadily increases. For any applied effective normal stress, there is a limit to the resistance that the soil can offer, which is known as the peak shear strength. In some cases an experimental test is stopped just after this point and the strength measured is referred to as the shear strength of the soil. If shearing is continued far beyond the maximum value of shear strength, the resistance of a clay decreases until a constant value is reached, which is known as the residual strength.

The alteration of the soil fabric causes changes in the strength properties of the soil, usually a loss in strength. The strength of the soil with its original fabric is termed the peak strength, and its strength under large deformation conditions, in a shear zone or on a shear surface, is referred to as the residual strength. In the latter case, the clay particles orientate themselves parallel to the direction of shearing. This results in the development of slickensided surfaces. Generally, problems involving initially unsheared soil must be approached using peak shear strengths where this peak strength is
the maximum shear stress that can be sustained, while the residual strength is often relevant to soil problems where there is a preformed failure surface.

The peak strength of normally consolidated clay exceeds the ultimate (residual) value because of rupture of cemented bonds, particle reorientation, and other factors contributing to sensitivity which are associated with the latter.

The shear strength of a heavily overconsolidated clay leads to a rupture of cemented bonds and swelling that causes strength loss beyond the peak to the residual at a constant final water content, regardless of the initial state of the soil.

In the soil mechanics literature, there are many different ways in which the shear strength of a soil may be expressed. In most cases, a Mohr envelope, in which shear strength (usually peak or residual) is plotted as a function of normal stress on the failure plane, or a modified Mohr diagram, where shear strength is plotted against the average of the major and minor principal stresses of failure, is used.

![Failure Envelope Representing Soil Strength](image.png)

**FIGURE 1.1:** FAILURE ENVELOPE REPRESENTING SOIL STRENGTH
Total and/or effective values of normal stress can be used. A straight line is then fitted to the resulting curve over the normal stress range of interest. Thus the shear strength $\tau$ is given by an equation of the form

$$\tau = C + \sigma_n \tan \phi$$  \hspace{1cm} (1.1)

or in effective stress terms

$$\tau = C' + \sigma_n' \tan \phi'$$  \hspace{1cm} (1.2)

For residual strength

$$\tau_{\text{res}} = C_r + \sigma_n' \tan \phi_r'$$  \hspace{1cm} (1.3)

since

$$C_r \approx 0$$

$$\tau_{\text{res}} = \sigma_n' \tan \phi_r'$$  \hspace{1cm} (1.4)

For most clays, however, the residual Mohr envelope is curved. For design purposes, most engineers have used the 'best fit' linear envelope. The question of the validity of this approximation is the main objective of this research. It is especially important at normal stresses where the curvature is most marked.

It should be noted that for the purpose of this research the term low normal stresses refers to that zone of normal stresses where the curvature of the residual strength envelope is most marked.
1.2 GENERAL ASPECTS AND PHYSICAL MEANING OF THE RESIDUAL STRENGTH OF CLAY

Residual strength of the soil is defined by Skempton (1964, 1985) as the minimum constant drained shear strength value attained for soil (at a slow rate of shearing) at large displacement. The displacement necessary to cause such a strength is usually far greater than that corresponding to the peak strength and fully softened state (critical state) in over-consolidated clay, as shown in Figure 1.2.

Generally the post-peak drop in drained shear strength of heavily over-consolidated clay may be considered as taking place in two stages. Firstly at a relatively small displacement the strength decreases to the 'fully softened', or 'critical' state, because of an increase in water content (dilatancy). Secondly, after a large displacement, the strength decreases to the residual value because of the orientation of Platey clay minerals parallel to the direction of shearing. The post-peak drop in strength of normally consolidated and lightly over-consolidated clay is due only to particle orientation.

The present understanding of the mechanism of residual strength has been presented by Lupini et al (1981). They concluded that there are three modes of shearing associated with residual strength, which are:

a) Turbulent mode. This mode usually occurs when soils either have a high proportion of rotund particles, or have Platey particles for which particle orientation does not occur.
DEFINITION OF PEAK AND RESIDUAL STRENGTHS

FIGURE 1.2 SHEAR STRESS VERSUS DISPLACEMENT FOR A TYPICAL CLAY

(b) DIFFERENCE BETWEEN NORMALLY CONSOLIDATED (N.C.) AND OVERCONSOLIDATED (O.C.) CLAY
b) **Sliding mode.** This corresponds to the case in which a low strength shear surface of strongly orientated, low friction platy particles forms.

c) **Transitional mode.** This mode involves both turbulent and sliding modes.

The transition from one mode to another is related to the packing and porosity of the rotund particles present.

It was found by many investigators that for most clay minerals and some natural soils, the residual friction angle $\phi'_r$ is markedly stress dependent below an average normal effective stress ($\sigma'_n$) of 150-200 kPa. Beyond this $\phi'_r$ becomes almost independent of the normal effective stress at which the clay particles possess a perfect parallel orientation. Consequently the drained shear strength reaches the lowest constant value (residual state) as shearing along such surfaces needs the lowest work ever required.

This subject will be highlighted in this thesis.

1.3 **AIMS AND RESEARCH PHILOSOPHY**

This research comprised two main phases. The preliminary phase included the following:

1. To compare and evaluate the present methods of measuring the residual strength of clay using both the reversible shear box and the Bromhead ring shear apparatus.
2. To find the most appropriate method of measuring residual strength by the shear box and ring shear apparatus separately.

3. Modifying the shear box and the ring shear apparatus if necessary in order to both suit the best technique of measuring obtained from (2) and to overcome testing difficulties.

The second phase which was the major phase included the following:

1. The determination of the full shape of the residual failure envelope using the recommended method obtained from the first phase for each particular clay.

2. To investigate the shape of the full residual envelope on both loading and unloading sequences.

3. To determine the failure envelope of the soil when sheared against concrete surfaces.

4. To investigate the physical cause of the curvature depending on the laboratory test results and the soil fabric investigation results.

5. To evaluate the engineering significance of such a problem and to correlate it to the soil properties.
1.4 MATERIAL AND METHOD OF TESTING

Three different types of natural soil have been used in the current research, two of them are common British soils (London clay and L. Lias clay) and a third type is a silty clay from Baghdad (Iraq) termed Baghdad clay for the purpose of this research. The shear strength tests were carried out using the conventional shear box and a Bromhead ring shear, each being connected to a data recording system. In each case modifications have been carried out to improve the standard equipment.

The standard shear box was simply modified by introducing a reversing switch, which changes the direction of shearing when the limit in either direction is reached. At the same time a message is relayed to the data recording system.

The main modification to the Bromhead ring shear apparatus was to include vanes on the top and bottom platens so that shearing would occur at the mid-height of the sample, instead of near to the top platen, with minimum extrusion of soil. To allow for this the initial sample thickness was increased from 5 to 10 mm.

Soil classification and chemical tests were carried out according to the British standards (BS 1377: 1975).

Soil fabric studies have been carried out using the X-ray diffractometer to determine the mineralogy of the clays. The results were estimated by comparing the test patterns with those prepared for known materials by the American Society for Testing and Materials.
Scanning electron microscope (SEM) photographs were taken for samples of both the shear surface and the unsheared face to examine the nature of the residual failure surface precisely. Samples were prepared using the recommended method of Gillot (1976).

1.5 THESIS CONTENT

This thesis contains eight chapters and two appendices.

Chapter 2 contains the literature survey, which includes a general historical review of the development of the subject. A survey of the laboratory methods of measuring the residual strength of clay and the factors affecting it are also included.

Chapter 3 is a description of the apparatus used in this research. Full details of the measuring units, data recording system, modifications and calibrations are all described.

Chapter 4 presents a description of the soils. The type of samples, their geological description and other properties (liquid limit, plastic limit, sulphate content, etc) are all presented in this chapter.

Chapter 5 is a presentation of the experimental programme, sample preparation, different testing techniques and all other classification and chemical tests.

Chapter 6 contains all the residual strength tests which have been carried out using the shear box and the ring shear apparatus.
Chapter 7 is a comprehensive discussion with a general conclusion for the whole work with special attention to the problem under investigation, the dependency of residual friction angle on the effective normal stress. Physical and theoretical explanations are described in view of the test results and the fabric studies.

Chapter 8 presents suggestions for further research in the field of residual strength of clay.
CHAPTER 2
REVIEW OF STUDIES ON RESIDUAL STRENGTH OF CLAY

2.1 INTRODUCTION

The post-peak reduction in the drained shear strength of cohesive soils has been recognised and studied extensively in the laboratory and field by many investigators. The ultimate post-peak drained shear strength of clay is termed 'residual strength'.

The major understanding of residual strength was presented by Skempton [1964] in the Fourth Rankine Lecture when he showed that the strength along any discontinuity in a clay mass is governed by the residual strength of the clay.

Consequently, most of the work which has been done during the last 26 years on drained residual strength and brittleness of cohesive soils has concentrated on geotechnical problems concerning old landslips, bedding shears in folded strata, sheared joints on faults, embankments post-failure and problems of stability in general [Peck, 1967; Skempton and Petley, 1967; Chandler, 1970, 1974, 1977, 1979; Calabresi and Manfredini, 1973; Chowdhury and Bertoldi, 1977; Branhead, 1978 and others].

Residual strength is generally determined from one or more of three types of test:

a) Reversing shear box test
b) Triaxial compression test
c) Ring shear test.
The ring shear apparatus provides lower bound results, whereas the reversible shear box provides upper bound values and the triaxial compression test produces a significantly higher strength.

Field values of residual strength have been determined by performing a back analysis on the slides.
2.2 GENERAL HISTORICAL REVIEW

The full development of the residual strength subject can be best described by considering the following six stages:

2.2.1 The Early Stage

This stage is generally characterised by the early recognition of the post-peak reduction in the drained shear strength of clay, and the development of a series of load-controlled torsion and ring shear apparatus.

As reported by La-Gatta [1970], the first image of the residual strength concept, or more precisely the post-peak strength reduction, appeared in the Soil Mechanics literature in 1846. The paper by Alexander Collin pointed to the decrease of shear strength along landslides. He recognised that cut slopes which are stable during construction, might fail at a later time due to softening of soil.

Later on, in 1936, Hvorslev conducted ring shear tests and demonstrated a post-peak reduction in strength following large strains.

Tiedman [1937] published results of rotation shear tests on remoulded and undisturbed clay. He found that at very large displacements the shear strength was a fraction of the peak strength, and he labelled it 'pure sliding resistance'. Strengths of 20-60% of the peak value were found after 100-250 mm.
Haefeli [1938, 1951] reported test results on remoulded clays obtained by a stress-controlled ring shear apparatus with a divided confining ring. The post-peak deformations were too limited to establish residual conditions. He described the reduced strength as 'remaining shear strength'.

Hvorslev [1938, 1939] introduced the term 'ultimate minimum shear strength' to describe the approximately constant strength measured at large deformation. He found it to be independent of test duration and stress history.

Henkel and Skempton [1954] recognised that the average undrained shear strength of an overconsolidated clay involved in the Jackfield slide grossly overestimated the long term stability of the slope, and that if the term 'C' is ignored a reasonable estimation was achieved.

Borowicka [1961, 1965] carried out reversal shear box tests on artificial clay soils. He found that for clays with higher colloidal content, \( \phi_r' \) drops continuously to a very low final constant value, while for clays of low colloidal content it did not. Borowicka attributed such observation to the fact that the scale-like and flake shaped colloids adjust themselves towards the shear plane until they form a shiny shear surface.

Skempton and Brown [1961] reported from their investigation of the landslide at Selset that C' is operative for the very long term slip in heavily overconsolidated intact clay, but in fissured clay, C' was found to be equal or close to zero due to the local over stressing and softening of clay in the vicinity of the fissures.
Horn and Deere [1962] carried out interparticle friction tests on different minerals. They found that micaceous material had a lower friction angle than rough massive shaped minerals.

By that stage it seemed that the residual strength concept had been recognised by the fact that most of the cohesive soils showed a significant reduction in the post-peak drained shear strength at large displacements. It is also nearly recognised that such reduction is due to the perfect orientation of the platy type clay particles in the shear zone.

2.2.2 Skempton's [1964] Contribution

The major understanding of residual strength was presented by Skempton [1964] in the Fourth Rankine Lecture when he showed that the strength along any discontinuity in a clay mass is governed by the residual strength of clay. He reviewed the effect of residual strength on slope stability and reported residual strength measurements using the shear box with multiple reversals of shear.

Skempton postulated a general correlation between residual strength and clay fraction, suggesting a relatively smooth transition with increasing clay content from a residual strength equivalent to \((\sigma_0 \tan \phi' \cdot c')\) for a granular soil to a low residual strength related to sliding between low friction clay particles in soils with a high clay fraction. He recognised also the importance of progressive failure and introduced the term 'residual factor', \(R\),

\[
R = \frac{\tau_f - \tau_r}{\tau_f - \tau_c} \tag{2.1}
\]

where

\(\tau_f\) is the shear stress on the active failure surface,

\(\tau_r\) is the shear stress on the residual failure plane,

\(\tau_c\) is the cohesion of the soil.
R is a measure of the proportion of the slip surface in a clay slope along which the strength has fallen to the residual value.

Finally he claimed that the residual strength of a clay is unique for a particular normal effective stress level and does not depend on previous consolidation history, sample preparation or initial water content.

The general finding of this contribution is that whenever a pre-existing shear surface occurs, the residual strength must be known, as it will exert a controlling influence on engineering design.

Due to the engineering significance of this contribution many investigators have paid attention to this subject in different ways. Bishop, Webb and Lewin [1965] reported measurements of residual strength of London clay using the triaxial apparatus. They found the triaxial test to suffer from too little displacement for residual strength measurement.

Chandler [1966] devised correction factors for such measurements in the triaxial test.

Herrman and Wolfskill [1966] found the triaxial test to overestimate residual strength, and the reversible shear box and ring shear to give comparable results. They found $\phi'$ to depend on $\sigma_n'$ and placement conditions, and the displacement required for $\phi'$ to depend on $\sigma_n'$.

In 1966 Petley carried out an extensive academic study on the shear strength of clay at large displacement. He measured the residual strength by means of both the direct shear apparatus and the triaxial
compression apparatus on different types of soils; Hamriver sand and kaolinite (undisturbed, slurried conditions and pre-cut samples). Petley concluded that the residual strength is not dependent on previous stress history. He added that there is a distinct tendency for the values of $\phi_r'$ to decrease with increasing clay fraction, although at a clay fraction above 60%, only a further small reduction appears to occur. Below a clay fraction of about 25%, the residual angle of shearing resistance seems to be approximately equal to the peak angle of shearing resistance.

Later on in 1966, Skempton, from his experience at the Vaiont landslide, showed $\phi_r'$ to be reached after a relatively small displacement in a clay with a small clay content.

In another investigation at Mangla Dam, Binnie, Clark and Skempton [1967] found that the shear zones have a higher average clay fraction, and hence lower residual strength parameters, than the rest of the clay bed.
2.2.3 Studies of Progressive Failure in Slopes

Studies of the development of sliding surfaces in slopes by progressive failure have been extensively covered by both Bishop and Bjerrum following Skempton's [1964] introduction of the subject.

In 1967 Bishop concluded that slope stability problems are usually associated with non-uniform mobilisation of shear strength (progressive failure), which occurs even in ideally homogeneous soils.

Bishop stated that the major difficulty in relating the average shear stress observed along the failure surface in the field to the relevant laboratory test depends in the first place on the difference between peak and residual strength, and secondly on the strain required for the difference to be established.

Again in 1971, Bishop referred to the studies of the distribution of stress in and beneath slopes, whether of excavations or embankments, by Bishop [1952], La Rochelle [1960], Bishop [1967] and Dunlop and Duncan [1970] which indicate significant non-uniformity of shear stress and stress ratio.

The possible error involved is directly related to the brittleness of soil defined by the parameter $I_B$ as:

$$I_B = \frac{I_f - I_r}{I_r} \quad 2.2$$

where $I_f = $ shear stress

$I_r = $ residual shear stress
He added that this parameter is strongly influenced by stress level, and in anisotropic soils depends on the orientation of the principal stress.

Bishop stated that the post-peak displacements are typically associated with a migration of water to the slip surface indicating a tendency for displacement to be associated with a temporary decrease in pore pressure. Therefore (on the point of limiting equilibrium for a first time failure of a previous intact slope) it follows that on some part of the slip surface the peak drained strength will be mobilised while the post-peak and pre-peak strengths will be operative over the remainder of the slip surface. A small further displacement, which will be associated with a decrease in overall shearing resistance, will be sufficient to bring the whole surface into the peak and post-peak state. Complete failure will then have occurred and the strength at all points still lies between the two limits of the peak and residual states.

He defined the factor $R_l$, which denotes the proportional drop from the peak to the residual strength as

$$R_l = \frac{\tau_f - \tau}{\tau_f - \tau_r} \quad 2.3$$

The distribution of $R_l$ values along the rupture surface and the range of values likely to be encountered in practice are at this stage mainly speculative. Although at least some part of the surface must be at an $R_l$ value equal to zero, it is less certain whether any other part will have reached the residual state.
Bjerrum [1967] postulated that the development of a sliding surface by progressive failure is possible in overconsolidated plastic clay provided that three conditions are satisfied. Firstly, the internal lateral stresses should be large enough to cause stress concentrations in front of an advancing sliding surface where the shear stresses exceed the peak strength. Secondly, the clay should contain a sufficient amount of recoverable strain energy to produce the necessary expansion of the clay in the direction of sliding to strain the clay in the zone of failure. In the third place, the residual shear strength should be relatively low compared to the peak shear strength.

Following the progressive failure studies, two distinguished researches concerning the factors affecting residual strength of clay were carried out by Kenney [1967 and 1977] in which he found that mineralogy is the most important factor controlling the residual strength of clay. Kenney's work will be presented later in Section 2.4.1 of this thesis.

Also in the 1960's fabric studies appeared in the residual strength literature for the first time.

Morgenstern and Tchalenko [1967a] studied the microstructure of the pre-peak, peak and post-peak shear induced fabric of a monomineralic clay, kaolin, subjected to direct shear. They also determined the residual angle of friction for the same clay using pre-cut samples.

They concluded that the shear induced fabric can be explained in terms of a combination of basal-plan gliding producing translation and rotation.
Morgenstern and Tchalenko [1967b] carried out microstructural observations on shear zones in different clay slips. They found that the displacement shears in general formed the boundaries of the shear zones and were between 10 μm and 100 μm in thickness, and occasionally much larger. The shear zones were found to be several millimetres to several centimetres thick.

The rest of the 1960's work is characterised mainly by the measurements of residual strength especially along natural slip surfaces.

Skempton and Petley [1967] classified the discontinuities in stiff clays according to their origin (depositional and structural) and with reference to the relative movement:
- Bedding surfaces (no movement)
- Joints and Minor shears (less than 1 cm movement)
- Principal displacement shears (more than 10 cm movement).

They found that, along principal slip surfaces in landslides and tectonic shear zones, the strength is at or close to the residual. Along minor shears, the strength is appreciably above the residual, whereas movements of not more than 5 mm are sufficient to bring the strength along the joints to its residual value.

Symon and Cross [1968] carried out triaxial tests with natural slip surfaces orientated at (45° + θ/2) and found that θ' had been reached, though a C' of 10 kPa was attributed to error.

Sembeneli and Ramirez [1969, 1971] found that shear rates influenced the results of torsion tests and that very slow rates should be used.
Skempton and Hutchinson [1969] studied reactivated landslides and mud flows. They found $\phi'_r$ varied by $-3^\circ$ to $+15^\circ$ over the whole site.

Skempton, Schuster and Petley [1969] carried out triaxial tests on blue London clay and found small peaks (due to surface irregularities) before $\phi'_r$ was reached within a few millimetres. They added that for practical purposes, the strength along joints and fissures may be taken as being at the residual value.

Webb [1969] found that the triaxial test can be used to obtain fairly close estimates of residual strength, particularly when failure occurs on pre-existing fissures or pre-cut planes.

By the beginning of the 1970's two pieces of academic research had been completed at Imperial College by Garga and James separately. Garga [1970] used a ring shear to test blue London clay, brown London clay, Weald clay and Cucaracha shale. His results showed an independency of loading sequence and specimen preparation, and dependency on the normal effective stress level.

James [1970] claimed that, except for an occasional wide discrepancy, the field and the laboratory values of residual friction angle obtained by the conventional multiple reversal shear box or triaxial tests on cut planes corresponded closely. Such a conclusion did not seem to him to be valid for the ring shear, where much greater continuous deformations are achieved. James added that, for practical purposes, $C'_r$ values should be taken as zero, rather than any other figure.
Following this work, Chandler [1970] tested samples of Lias clay containing a principal shear surface using the triaxial test and direct shear test, and found that both tests gave identical results (C' = 0, $\phi'_r = 18.5^\circ$).

Mesri and Oslon [1970] investigated the shear strength of montmorillonite using the triaxial test. They found that the effective stress failure envelopes of calcium and sodium montmorillonite were curved. The decrease in slope of the effective stress envelope could be due to greater parallelism of clay particles at higher consolidation pressure.

Skempton [1970] concluded that there are some overconsolidated clays (notably those without fissures) in which the strength appropriate to first time slides is nearly equal to the undisturbed peak value and includes an important C' term. The use of fully softened, or critical state strength in such clays, would lead to excessively low factors of safety, while many stiff fissured clays undergo a loss in strength in cutting towards the fully softened value, more or less in accordance with the critical state concept.

Smart [1970] developed a formula to estimate the residual angle of internal friction for a soil composed of clay and sand, which is:

$$
\tan \phi'_r = C^2 \tan \phi'_C + 2C (1-C) \tan \phi'_m + (1-C^2) \tan \phi'_S
$$

where:  
$C = $ clay fraction  
$\phi'_C = $ residual angle of internal friction of clay-sand  
$\phi'_S = $ residual angle of internal friction of sand-sand  
$\phi'_r = $ residual friction of angle of soil
2.2.4 The Development of the Present Sophisticated Ring Shear Apparatus

In the early 1970's two sophisticated ring shear apparatus were developed, the first one by La-Gatta [1970] at Harvard University, USA, and the other by Bishop et al [1971] which was built jointly by Norwegian Geotechnical Institute and Imperial College, University of London. Their development has presented significant contributions to the advancement of the measurement of residual strength. A description of the two ring shears will be shown later in Section 2.3.3 of this thesis. Because of the complicated features of these two ring shears, the need for a simpler one arose. Later on in 1979, Bromhead developed his simple ring shear apparatus at Kingston Polytechnic. Details of the apparatus are also described in Section 2.3.3.

In the period bounded between the development of the complicated and simple ring shear apparatus, a lot of measurements and observations have been recorded by many investigators regarding the residual strength of clay, and these are summarised as follows.

Cullen and Donald [1971] found from their experience with Australian soils that $\phi_r$ is independent of speed variations, shear box travel, handwinding and pre-cut failure planes, however it is related to the water content at the residual state for a given normal stress.

La-Gatta [1971] studied rate effects and found that if one goes fast enough $\phi_r$ is affected, and that $\phi_r$ increases with rate.

Chattopadhyay [1972] included the term (R-A), the interparticle stress due to the physico-chemical environment to give $\tau_{res} = [\eta n-(R-A)]\tan\phi_r$. Therefore for inactive minerals (kaolinite and attapulgite) there is no effect, but for sodium montmorillonite there is.
Marsh [1972] developed a shear box to enable the direction of shear to be reversed so that the sheared specimen may be returned to its initial position prior to the commencement of a further formal shear.

Palladino and Peck [1972] found that stress relief during deglaciation caused horizontal slips along the bedding planes of overconsolidated clay, reducing the strength to residual.

Blondeau [1973] measured the residual strength of different French soils using the shear box, the triaxial compression test and the ring shear apparatus. He found that pre-cut samples in the shear box gave values dependent on the number of cutting passes and their direction. With the triaxial test, the rubber membrane elongation led to a significant error. However the ring shear test was thought to be very sophisticated and it was only used for comparative tests.

Calabresi and Manfredini [1973] have carried out drained shear tests using the shear box on intact samples with different types of structural discontinuities of the jointed overconsolidated clay of S. Barbara mine. They found $\phi = \phi_p$ and $C'$ nearly equal to zero along joints and bedding planes, and that $\phi_r'$ was reached after a small displacement. Along faults, the shear strength had already reached the residual value. Intact clay was more brittle parallel to bedding planes.

Townsend and Gilbert [1973, 1974, 1976] found that $\phi_r'$ depends on clay fraction and mineralogy, it is not related to index properties and did not vary between the ring shear test and shear box test, or with sample preparation method. $\phi_r'$ is independent of $\sigma_n'$ above 150 kPa, and approximates to a straight line through the origin.
Chandler and Skempton [1974] suggested that pre-existing slip surfaces are governed by $\phi'_{r}$ and that using $C' = 0$ is unduly conservative (absolute lower bound).

Osien [1974] has reported that triaxial tests could not be carried out to a high enough strain to approximate residual strength.

Bucher [1975] found that the agreement between the reversal shear box test and ring shear test results was generally good, and that $\phi'_{r}$ is independent of stress history, sample preparation and temperature.

Saito and Miki [1975] defined the 'plastic ratio' (PI/PL) to relate $\phi'_{r}$ to index properties, and formed a chart based on other researchers' data. [PI/PL = 1-2 and LL > 50 gave $\phi'_{r} = 10^\circ - 20^\circ$; and PI/PL > 2 gave $\phi'_{r} = 5^\circ - 10^\circ$.]

As reported by Lupini et al [1981], Blondeau and Josseaume [1976] found that the triaxial test overestimated $\phi'_{r}$ and ring shear tests were found to be too involved and time consuming.

Cancelli [1977] carried out field and laboratory investigations for the stability of a landslide in Italy. He found discrepancies between the laboratory ring shear and shear box tests, and field residual values.

Chandler [1977] carried out back analyses of Barnsdale and other Lias clay landslides which showed residual strength parameters of $C' = 0$ and $\phi'_{r} = 11^\circ$. He claimed that analysis of individual segments of landslides enabled the degree of curvature of residual strength envelopes to be established.
Chowdhury and Bertoldi [1977] found that $\phi_r' \sim \phi_p'$ for a sandy soil and $\phi_r' < \phi_p'$ for plastic clay, with the latter having a curved envelope, thought to be due to a lack of the strong particle orientation found at high $\phi_r'$. The water content in the shear zone was found to be independent of the initial value. Initial water content and stress history have no effect on $\phi_r'$.

Kanji and Wolle [1977] presented a new technique for measuring the residual strength of clay using a standard direct shear device and composite specimens of soil and polished rock or hard surfaces, in which shearing occurs along the contact surface. He found a lower $\phi_p'$ at a smaller displacement, and a rapid post-peak drop in strength and a constant minimum strength at a limited displacement.

Wesley [1977] investigated the shear strength properties of the halloysite and allophane clays of Java, Indonesia using the triaxial machine and ring shear apparatus for this purpose. He found that for allophane clays $\phi_r'$ is approximately equal to $\phi_p'$ due to the absence of platy particles, and for halloysite clays $\phi_r'$ was lower than $\phi_p'$, though it was still high.

Bromhead [1978] carried out stability analyses of three large landslides in the Coastal London clay cliffs at Herne Bay, Kent. He achieved a curved strength envelope for field and shear box (pre-cut plane) results.

Seycek [1978] carried out a large number of reversal shear box tests on tertiary clays of the North Bohemian brown-coal basin, Czechoslovakia. He presented his shear box data and published data to show a correlation between $\phi_r'$ and $I_p'$. 

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Chandler [1979] reported a case record of a renewal of movement in the Wansford landslide. He found that \( \phi' \) measured by the ring shear is 9.4°, whereas back analyses gave \( \phi' = 9.8° \). So the ring shear test underestimated the field residual strength angle.

By the beginning of the 1980's Lupini propounded the present understanding of the mechanism of residual strength in terms of the mode of shearing which basically depends on the mineralogy of the clay.

2.2.5 The Present Understanding of the Mechanism of Residual Strength

Lupini [1960] and Lupini et al [1981] have presented significant contributions to the advancement of the understanding of the mechanism of residual strength.

They carried out three series of tests on different soil mixtures. Three modes of residual behaviour are recognised: a turbulent mode, a transitional mode and a sliding mode, the mode depending on dominant particle shape and coefficient of interparticle friction.

The turbulent mode occurs when behaviour is dominated by rotund particles, or, possibly, in soils dominated by platy particles when the coefficient of interparticle friction between these particles is high. In this mode residual strength is high, no preferred particle orientation occurs and brittleness is due to dilatant behaviour only. The residual friction angle depends primarily on the shape and packing of rotund particles and not on the coefficient of interparticle friction. A shear zone, once formed, is only a zone of different porosity and it is considerably modified by subsequent stress history.
The sliding mode occurs when behaviour is dominated by platy, low friction particles. A low strength shear surface of strongly orientated platy particles then develops. The residual friction angle depends primarily on mineralogy, pore water chemistry and on the coefficient of interparticle friction. A shear surface, once formed, is not significantly affected by subsequent stress history. Brittleness during first shearing is due primarily to preferred particle orientation.

The transitional mode occurs when there is no dominant particle shape, and involves turbulent and sliding behaviour in different parts of the shear zone. The properties of the soil in residual shear change progressively across the transitional range from those typical of turbulent shear to those typical of sliding shear. In this mode the residual friction angle is sensitive to small changes in grading of the soil, and the changes in grading required to cross this range entirely are, typically, small.

They concluded that the correlations between residual strength and both soil index properties and grading cannot be general. Such parameters may be valuable in studying the residual strength of a particular variable soil deposit, provided that they properly reflect changes in the more fundamental properties of particle shape, grading, mineralogy, pore water chemistry, etc.

In the period between Lupini's investigation of residual strength and Skempton's second contribution [1985], in which he summarised much of the residual strength work that followed his first lecture in 1964, the following work on residual strength has been undertaken.
Bucher and Kyuluie [1980] investigated the residual shear strength of two tropical sedimentary clays, Weathered Mudstone ($I_p = 25.5$) and Accra Shale ($I_p = 26.9$). His ring shear test results showed quite different peak angles of shearing resistance ($\phi_p' = 33.4^\circ$ and $25.8^\circ$) respectively, while $\phi_r'$ was the same at approximately $22^\circ$.

Hutchinson, Bromhead and Lupini [1980] found that the values of $\phi_r'$ for a particular sample of Gault clay measured in the Bishop and Bromhead ring apparatuses were very similar ($\phi_r' = 12^\circ$ for clay with a low liquid limit and $\phi_r' = 7^\circ$ for a high liquid limit).

Petley [1980] presented three case histories in which he found that the ring shear tests gave lower values of residual strength than those obtained from shear box and triaxial tests. The latter two were in good agreement with the mobilised shear strength determined from back analysis.

Miedema, Byers and McNeary [1981] carried out work on the shear box of Nespelem overconsolidated clay and found $\phi_r'$ to decrease with increasing PI and LL, though there was no variation with displacement rate.

Voight and Faust [1982] suggested heat generation in slip zones, caused a fluid pressure increase and a strength reduction.

Steward and Cripps [1983] found $\phi_r'$ of shale to vary with pore water chemistry and mineralogical composition, both of which are altered by weathering reactions.
Bromhead and Curtis [1983] have carried out a series of consolidated drained shear box tests on natural surfaces in London clay. They found the ring shear test to be more consistent and quicker than the shear box test, though similar results of $\phi_r'$ were obtained for London clay.

Boyce [1984] reported residual strength results on Zimbabwe soils using the Bromhead ring shear and shear box apparatus. Boyce found $\phi_r'$ to be approximately equal to $\phi_p'$ for coarse silt and fine sand dominated clays. Silty clays have a higher $\phi_r'$ than highly plastic clays, unless there are significant amounts of micaceous particles.

Chandler [1984] found that clays with PI < 20-25 have good agreement between $\phi_l'$ laboratory and $\phi_f'$ field. High PI clays can exhibit rapid strain softening and progressive failure, which causes $\phi_r'$ laboratory to vary from back analysed values obtained from the field. He found that $\phi_r'$ from the ring shear test underestimates by 2 or 3° $\phi_r'$ field where C' is taken as zero.

Anderson, Yong and Suliman [1985] showed that bored piles cause residual shear planes which require only small movements for $\phi_r'$ to be reached. The insertion of casing can likewise reduce $\phi_l'$ to $\phi_r'$.

Hawkins and Privett [1985] confirmed that residual failure envelopes are curved, most markedly below $\phi_r' = 200$ kPa.

In 1985 Lemos et al and Vaughan et al examined earthquake behaviour in relation to residual strength. They recognised that there are five different strengths during fast shearing and they related the gain or loss of strength during earthquakes to clay fraction. Their findings will be presented later in Section 2.4.7.
2.2.6 Skempton's [1985] Contribution

In the Rankine Lecture of 1964, Skempton drew attention to the nature and significance of residual strength when he demonstrated that wherever pre-existing shear surfaces occur, the residual strength must be measured, as it will exert a controlling influence on engineering design. As a result there was an increasing interest in the residual strength and its relating subjects.

In 1985 Skempton presented an extensive review of the most important work following the Fourth Rankine Lecture concerning the residual strength and brittleness of cohesive soils, with its application to the relevant geotechnical problems. In this later contribution, Skempton made the following comments:

1. For most clays the relationship between residual strength and normal effective stress is non-linear, so when comparing one clay with another a standard pressure should be used.

2. Ring shear tests tend to give values of residual strength somewhat lower than the field values for higher clay fraction materials. The typical difference in the angle of shearing resistance is 1° to 2°.

3. The clay minerals can have little effect on residual strength when the clay fraction is less than 20%, as the strength is then controlled largely by the sand and silt particles. Conversely, with clay fractions exceeding 50%, residual strength depends almost entirely on the sliding friction of clay particles and therefore depends on their character.
4. Any relationship between residual strength and clay fraction for natural material should cover a wide range of particle size and have essentially the same clay mineralogy throughout.

5. The variations in strength within the usual range of slow laboratory tests (0.002-0.1 mm/min) are negligible, and that for clays, the increase in strength becomes pronounced at rates exceeding 100 mm/min when some qualitative change in behaviour occurs. This is probably associated with disturbance of the original ordered structure producing 'turbulent' shear.

The work that followed Skempton's 1985 contribution and up to the time being of completion of the present research is briefly as follows.

Bromhead and Dixon [1986] considered that if a sufficiently large number of measurements were taken, \( \phi'_r \) would be the same for both field analyses and laboratory measurements.

Hawkins and Privett [1986] and Hawkins [1988] confirmed that \( \phi'_r \) is stress dependent, therefore it is not possible to have a single correlation chart, but a series of charts for \( \phi'_r \) values determined at different normal effective stresses.

Mersi and Cepeda-Diaz [1986] found \( \phi'_r \) after 50 mm displacement of pre-cut samples in the shear box and warned against confusing large strain strength due to strain softening in soft clays (loss of interparticle chemical bonds) and residual (orientation) strength.
Skempton and Coates [1986] have postulated from their experience with the earthworks failures at the Carsington dam site that the critical state condition cannot be generalised and most of such failures occurred at the residual condition.

Anderson and Sulaiman [1987] and Anderson [1988] found that there is a close correspondence between the angle of shearing resistance mobilised on the shaft pile elements and the residual angle of shearing resistance of clay.

Anderson and Hammond [1988] found multi-stage testing in the Bromhead ring shear to be good for clays exhibiting turbulent or transitional behaviour, but to give lower values than single tests for clays exhibiting sliding behaviour (due to flattening of the shear zone caused by subsequent consolidation).

Fell, Sullivan and Macgregor [1988] have shown from their investigation of landsliding in sedimentary rocks in the Sydney Basin that sliding occurred on well defined near horizontal slide planes parallel to the bedding. The reversal direct shear results were not in good agreement with results obtained from back analysis of landsliding.

Crefsheim [1988] found that pre-cut samples gave lower values of $\phi_r'$, but that $\phi_r'$ determined without pre-cutting agreed most closely with back analyses.

Mathews [1988] found good agreement between ring shear and shear box results, but poor correlation with back analyses (attributed to complex pore pressure regimes).
Collatta et al (1989) proposed a correlation between the residual friction angle $\phi_r'$, gradation and the index properties of cohesive Italian soils.

Maksimovic (1989) proposed an expression for the non-linear residual failure envelope as follows:

$$\tau = \sigma_n' \tan \phi_r' + \Delta \phi_r' \left(1 + \sigma_n' / \sigma_n^* \right)$$
2.3 REVIEW OF THE METHODS OF MEASURING RESIDUAL STRENGTH OF CLAY

There are three different types of apparatus that can be used to measure the residual strength of the clay. These are:

- triaxial compression machine
- direct shear box
- torsional apparatus (disc-shaped samples) or ring shear apparatus (annular samples).

The residual strength can be measured using the triaxial compression machine or direct shear apparatus, which are readily available in most geotechnical engineering laboratories, by modifying these standard items of equipment and their experimental procedure. However the ring shear apparatus is specialist testing equipment.

Before the development of the ring shear apparatus, most published measurements had been made in the shear box using multiple reversal techniques [Skempton, 1964; Skempton and Petley, 1967; Kenney, 1967; Chowdhury and Bertoldi, 1977; and others]. Some measurements had also been made using the triaxial compression machine [Chandler, 1966; Webb, 1969; and others], although there is general agreement on the limitations of the use of the apparatus to measure the residual strength of clay as it overestimates the strength [Herrman and Wolfskill, 1966; Blondeau, 1973; and others]. However with samples containing shear planes or pre-cut planes inclined approximately at $(45^\circ + \phi/2)^\circ$ to the horizontal, a better result can be obtained. The latter technique is not as simple as the shear box technique.
Many investigators have found that there is reasonable agreement between residual strength measurements using the ring shear apparatus and the reversible shear box [Herman and Wolfsill, 1966; Bucher, 1975; Townsend and Gilbert, 1976; Bromhead and Curtis, 1983; and others]. It is thought, however, that the ring shear test usually produces lower bound measurements, whereas the latter provides an upper bound value [Hutchinson, Somervill and Petley, 1973; Hawkins and Privett, 1985; and others].

2.3.1 Triaxial Compression Machine

The triaxial test has not been used to any great extent in measuring residual strength of clay due to its main limitation of not being able to provide enough strain to develop the residual strength of the soil sample [Blondeau, 1973; Oslo, 1974; and others]. In addition there are secondary defects which are usually associated with the test, such as:

1. The complex stress distribution across the failure plane which is usually produced if the test is continued beyond the peak strength.
2. Effect of membrane restraint.
3. Change in cross-sectional area.
4. Effect of filter drains.
5. End friction imposed by the formation of horizontal components of load after the formation of a shear plane.

In order to improve the test for the measurement of residual strength such difficulties should be minimised. Chandler [1966] developed corrections for the effects of the change of the cross-sectional area and the restraint of the rubber membrane.
The area correction formula produced by Chandler is expressed as follows:

\[ A = \left[ 2r^2 \sin^{-1} \left( \frac{x}{r} \right) + 2x \sqrt{r^2 - \frac{x^2}{r^2}} \right] \times r \]

\[ \chi = \frac{\Delta h \cot \alpha}{2} \]

where \( A \) is the corrected area, \( 2r \) is the diameter of the sample and \( \alpha \) is the angle of the horizontal at which the shear plane has been cut. The rubber membrane correction presented by Chandler is valid up to 12% axial strain.

For the same purpose Webb [1969] reported a method of analysis of residual strength which takes into account the horizontal component of load after the formation of a shear plane in the conventional triaxial test. Webb has presented another area correction formula as follows:

\[ A_s = d_p^2 \sec \left[ 1 - \frac{8}{\pi} (e - e_p) \right] \cot \theta / 4 \]

where:
- \( A_s \) = contact area at residual strength
- \( A_p \) = contact area at peak strength
- \( \theta \) = angle of failure surface
- \( e \) = total nominal axial strain
- \( e_p, d_p \) = axial strain and diameter respectively at peak deviator stress.

In spite of all of the above problems the test can sometimes be used to obtain fairly close estimates of residual strength, particularly when failure occurs on a pre-existing shear surface or pre-cut plane, due to the fact that such samples need relatively small strain to achieve the residual state [Petley, 1966; Skempton and Petley, 1967; Symon and Cross, 1968; and others].
Table 2.1 shows some available residual strength data obtained from the triaxial test by several investigators. As shown in this table most of the published data were obtained before the development of the new ring shear apparatus, and most of these tests were carried out on cut plane or slip surface samples.

2.3.2 Reversible Shear Box
The shear box has a limited shearing displacement and since a very large displacement is required to define the residual strength of the soil in comparison to that required to define the peak strength of the same soil, the laboratory determination of the residual strength requires unconventional apparatus and/or techniques for producing large enough displacement. The conventional shear box involves shearing the specimen a limited distance in one direction. To determine the residual strength of the soil, the shearing displacement achieved by the apparatus should be increased. To achieve that, the shear box can be modified to enable the direction of shear to be reversed, so that the sheared specimen may be returned to its initial position prior to the commencement of a further forward shear. These operations are repeated until sufficient displacement has accumulated. Such a device is known as the "reversible shear box". Marsh [1972] successfully used such a box.

The reversible shear box has been widely used for measurements of residual strength especially before the recent development of the ring shear apparatus [Skempton, 1964; Borowika, 1961, 1965; Skempton and Petley, 1967; Oullen and Donald, 1971; Marsh, 1972; Chowdhury and Bartoldi, 1977; Miedema, Byers and McNearuy, 1981; and others].
<table>
<thead>
<tr>
<th>Investigators</th>
<th>Year</th>
<th>Soil Type</th>
<th>Sample Type</th>
<th>Soil Parameters</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chandler</td>
<td>1966</td>
<td>Keuper Marl</td>
<td>Pre-formed</td>
<td>0.0 27.5</td>
<td>LL=34, PL=11, CF=17</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Shear planes</td>
<td></td>
<td>√</td>
</tr>
<tr>
<td>Petley</td>
<td>1966</td>
<td>Oxford Clay</td>
<td>Out-plane</td>
<td>0.0 15.5</td>
<td>LL=55, PL=25, CF=44</td>
</tr>
<tr>
<td>Binne, Clark &amp; Skempton</td>
<td>1967</td>
<td>Mangla Clay</td>
<td>Along</td>
<td>0.0 18.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>bedding</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Across</td>
<td>0.44 26.0</td>
<td></td>
</tr>
<tr>
<td>Skempton &amp; Petley</td>
<td>1967</td>
<td>Reworked</td>
<td>Slip</td>
<td>0.0 16.0</td>
<td>LL=71, PL=31, CF=58</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Atherfield Clay</td>
<td>surface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Symon &amp; Cross</td>
<td>1968</td>
<td>Weald Clay</td>
<td>Slip</td>
<td>7.0 10.0</td>
<td>LL=79, PL=31, CF=58</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>surface</td>
<td>-2.0 12.0</td>
<td>LL=89, PL=38, CF=66</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.0 9.0</td>
<td>LL=89, PL=31, CF=65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Sevenoaks Bypass</td>
</tr>
<tr>
<td>Skempton, Schuster &amp;</td>
<td>1969</td>
<td>Blue London Clay</td>
<td>Joint &amp;</td>
<td>1.3 16.0</td>
<td>Wraysbury</td>
</tr>
<tr>
<td>Petley</td>
<td></td>
<td></td>
<td>fissure</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>surface</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Webb</td>
<td>1969</td>
<td>London Clay</td>
<td>Pre-ex.</td>
<td>0 13.0</td>
<td>LL=70, PL=42, CF=53</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>fissures</td>
<td>0 11.5*</td>
<td>*Loading cap with free lateral movement</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pierre shale</td>
<td>0 18.5</td>
<td>LL=140, PI=100, CF=56</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0 16.0</td>
<td>LL=89, PI=47, CF=27</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0 24.0</td>
<td></td>
</tr>
<tr>
<td>Chandler</td>
<td>1970</td>
<td>Lias clay</td>
<td>Principal</td>
<td>0 18.5</td>
<td>LL=61, PI=32, CF=52</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>shear surface</td>
<td></td>
<td>The site near Uppingham, Rutland</td>
</tr>
<tr>
<td>Blondeau</td>
<td>1973</td>
<td>Lias clay</td>
<td>Pre-cut</td>
<td>0 13.0</td>
<td>LL=53, PI=25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>samples</td>
<td></td>
<td></td>
</tr>
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</table>
Generally, this apparatus was used to measure residual strength of soil for three major types of samples:

i) Laboratory samples (remoulded)
ii) Laboratory samples (undisturbed)
iii) Pre-sheared samples (containing pre-formed failure surface or natural shear surface).

The first type of sample, the remoulded laboratory sample, is the most common type because of its simple preparation. Samples are usually prepared by breaking up the air dried material to form a powder passing the 425 μm sieve. Distilled water is then added to the clay and it is thoroughly mixed at a water content usually between its liquid limit and plastic limit. The soil is usually kneaded into a mould and then pushed into the shear box [Petley, 1966; Kenney 1967 and 1977; Chowdhury, 1977; and many others]. The use of undisturbed samples is relatively limited due to the difficulties which are usually associated with these samples such as trimming processes, inadequate saturation and pore pressure variations. On the other hand many investigators such as Skempton [1964], Ollen and Donald [1971] and others have claimed that residual strength of clay does not depend on the sample preparation, which in turn encourages the investigators to use remoulded samples as the easiest method is preferable.

Durham [1976] tried to improve the test in this direction when he developed a special trimming device for obtaining undisturbed samples, which is composed of an unconfined compression machine with a special specimen cutter. However, this development proved to be no great advancement.
Shear box samples are usually sheared forward and backward until sufficient displacement has accumulated. A characteristic stepped displacement shear stress curve was produced in which succeeding peaks are due to the particle orientation or the formation of a new failure surface with each cycle. In spite of this obvious disadvantage, quite reasonable results can often be obtained [Skempton, 1964; Petley, 1966; Chattopadhyay, 1972; Calabresi and Manfredini, 1973; and others]. This technique will be presented and discussed in more detail later on in this thesis.

Pre-sheared samples containing natural shear surfaces have been tested by many investigators to measure the residual strength of clay [Skempton, 1964; Skempton and Petley, 1967; Skempton, Schuster and Petley, 1969; Calabresi and Manfredini, 1973; and others]. Tests carried out on such surfaces are usually slow drained shear tests. This test is considered very suitable as a method of measuring field residual strength parameters, even without the facility of the reversal technique. This arises from the fact that field samples containing a slip surface need only a very small displacement (typically only a few millimetres) to reach the residual strength of the soil. However the process of obtaining such samples is not easy and can be costly, and the trimming of test specimens poses further problems. In addition, specimens for shear box tests must have the shear surface aligned precisely with the plane of separation of the shear box, and this may be impossible where the shear surface is not flat in nature. Furthermore small compressions taking place during the consolidation stage of the tests lead to the shear surface settling slightly.
Generally, care should be taken to locate the slip surfaces as exactly as possible in the plane of the box and to arrange the sample so that shearing follows the natural direction of movement.

A new testing technique to obtain the residual strength of clay soils has been developed using the standard shear box device and composite specimens of soil and polished rock, or other hard surfaces, sheared along their contact [Kanji, 1974 and Kanji and Wolle, 1977]. The rapid drop in strength with displacement along the contact surfaces tested suggested that the hard surface plays an important role in helping the rapid orientation of the clay particles in the soil on a flat plane in the direction of shearing.

In conclusion, it can be said that this type of test is quite acceptable for measuring residual strength of clay, especially along pre-sheared surfaces (laboratory or field samples), as only a small displacement (in the order of 1-2 cm) is usually required to develop the residual strength and this can be achieved by one shearing cycle. In addition the function of the shear box enables the use of composite samples of clay and concrete, steel or any other material is required.

Different testing techniques have been used during this research. The advantages and disadvantages of these techniques will be discussed and evaluated later on in this thesis.

Table 2.2 presents data of residual strength obtained from the shear box. It is very difficult to tabulate the full available data on this subject, however what are shown here are the most famous published results.
### TABLE 2.2: RESIDUAL STRENGTH DATA OBTAINED BY THE SHEAR BOX

<table>
<thead>
<tr>
<th>Investigators</th>
<th>Year</th>
<th>Soil Type</th>
<th>Sample Type</th>
<th>Soil Parameters</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skempton</td>
<td>1964</td>
<td>Selset</td>
<td></td>
<td>0 30 LL=26, PL=13, CF=17</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Jari</td>
<td></td>
<td>0 18 LL=70, PL=27, CF=47</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>London</td>
<td></td>
<td>0 16 LL=82, PL=29, CF=55</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clay</td>
<td></td>
<td>0 13 LL=53, PL=28, CF=69</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Watton's</td>
<td></td>
<td>0 19 LL=44, PL=22, CF=36</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wood</td>
<td></td>
<td>0 16 LL=83, PL=30</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Jackfield</td>
<td></td>
<td>0 15 LL=82, PL=29</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>London</td>
<td></td>
<td>0 13 LL=53, PL=28, CF=69</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>clay</td>
<td></td>
<td>0 19 LL=44, PL=22, CF=36</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0 16 LL=83, PL=30</td>
<td></td>
</tr>
<tr>
<td>Petley</td>
<td>1966</td>
<td>Oxford</td>
<td>Un.</td>
<td>0 15.5 LL=55, PL=25, CF=44</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clay</td>
<td>Slu.</td>
<td>0 15.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>slips</td>
<td>0 14.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Avonmouth</td>
<td>Slu.</td>
<td>0 25.0 LL=64, PL=28, CF=40</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Wiener-</td>
<td>Slu.</td>
<td>0 25.0 LL=52, PL=24, CF=33</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tegel</td>
<td></td>
<td>0 25.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Fiddler's</td>
<td>Slu.</td>
<td>0 24.0 LL=43, PL=22, CF=30</td>
<td></td>
</tr>
<tr>
<td>Kenney</td>
<td>1967</td>
<td>London-</td>
<td>Slu.</td>
<td>0 9.6 LL=72, PL=29, CF=57</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>Wray'sbury</td>
<td></td>
<td>Materials containing montmorillonite.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>London-</td>
<td>Slu.</td>
<td>0 9.0 LL=66, PL=24, CF=53</td>
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<tr>
<td></td>
<td></td>
<td>Walthamstow</td>
<td></td>
<td>Soils containing hydrous mica</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Ottawa-</td>
<td>Slu.</td>
<td>0 28.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canada</td>
<td></td>
<td>Soils containing hydrous mica</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Labrador-</td>
<td>Slu.</td>
<td>0 26.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Canada</td>
<td></td>
<td>Soils containing hydrous mica</td>
<td></td>
</tr>
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44
<table>
<thead>
<tr>
<th>Investigators &amp; Year</th>
<th>Soil Type</th>
<th>Sample Type</th>
<th>Sample Parameters</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skempton &amp; Petley 1967</td>
<td>Weathered mudstone</td>
<td>Un.</td>
<td>(0) (13)</td>
<td>LL=37, PL=26, CF=69 Walton's Wood 'Intact slip'</td>
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<tr>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Brown London Clay</td>
<td>Un.</td>
<td>(0) (12)</td>
<td>LL=83, PL=32, CF=56 Guilford (intact slip)</td>
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<tr>
<td></td>
<td>Atherfield Clay</td>
<td>Un.</td>
<td>(0) (12)</td>
<td>LL=75, PL=29, CF=57 Sevenoaks (intact slip)</td>
</tr>
<tr>
<td></td>
<td>Upper Sinalick Clay</td>
<td>Un.</td>
<td>(0) (14) (17.5) (16)</td>
<td>LL=60, PL=28, CF=55 Marga (minor shear)</td>
</tr>
<tr>
<td></td>
<td>Blue London Clay</td>
<td>Un.</td>
<td>(1.5) (16) (3.5) (18.5)</td>
<td>LL=73, PL=28, CF=55 Wraysbury (joint surface)</td>
</tr>
<tr>
<td>Chandler 1970</td>
<td>Lias Clay</td>
<td>Un.</td>
<td>0</td>
<td>18.5</td>
</tr>
<tr>
<td>Cullen &amp; Donald 1971</td>
<td>O.C. Clay Fault Zone</td>
<td>Un.</td>
<td>15.0</td>
<td>PL=20-25% Kaolinite soil PL=22% Low mica content</td>
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<td>Blondeau 1973</td>
<td>Lias Menton Nancy</td>
<td>Un.</td>
<td>0 0.7 1.8</td>
<td>9.0 28.0 19.0</td>
</tr>
<tr>
<td>Calabresi &amp; Manfredini 1973</td>
<td>S.Barbara O.C. clay</td>
<td>Un.</td>
<td>0</td>
<td>11.0</td>
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</table>

45
<table>
<thead>
<tr>
<th>Investigators</th>
<th>Year</th>
<th>Soil Type</th>
<th>Sample Type</th>
<th>Soil Parameters</th>
<th>Remarks</th>
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</thead>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$C'_r$ kPa $\phi'_r$ deg</td>
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<tr>
<td>Petley</td>
<td>1980</td>
<td>Weathered Oxford clay</td>
<td>Cut-plane Un. slip sur.</td>
<td>0 0 16.0 16.0</td>
<td></td>
</tr>
<tr>
<td>Miedema, Byers and McNeary</td>
<td>1981</td>
<td>Hard, fracture Varved clay</td>
<td>Un. Pre-cut</td>
<td>0 0 11.0 L=66, PL=41, CF=60</td>
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</tr>
<tr>
<td>Bronhead &amp; Curtis</td>
<td>1983</td>
<td>Brown to grey London clay</td>
<td>Natural and artificial slip surface</td>
<td>0 0 10.0 L=69, PL=27, CF=40</td>
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</tr>
<tr>
<td>Hawkins &amp; Privett</td>
<td>1985</td>
<td>Cotham Member</td>
<td>Pre-cut</td>
<td>7.0 5.3 100 x 100 mm</td>
<td></td>
</tr>
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<td></td>
<td></td>
<td></td>
<td>13.5 6.3 60 x 60 mm</td>
<td></td>
</tr>
<tr>
<td>Mesri and Cepeda-Diaz</td>
<td>1986</td>
<td>Highly O.C. Clay</td>
<td>Pre-cut</td>
<td>0 0 L=39, PL=20 CF=43 (chinline, red)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0 0 L=82, PL=30 CF=40 (Pierre)</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0 0 L=288, PL=44 CF=88 (Bearpaw)</td>
<td></td>
</tr>
</tbody>
</table>
The following could be concluded in the light of these data:

1. Most of the work was initiated after Skempton [1964] and vanishes by the 1980s, the decade of the ring shear apparatus.
2. Tested samples were mostly undisturbed for clay with structural discontinuities, slip surface, joints etc.

2.3.3 Ring Shear Apparatus

All previous work on residual strength of soil has found the ring shear or torsion shear test to be a very useful method. This arises from the fact that there is no change in area of cross-section as the test proceeds and the sample can be sheared through an uninterrupted displacement of any magnitude.

Bishop et al [1971] stated that any measurement of the residual strength of soil should satisfy the requirements that the normal and shear stresses at failure should be as uniform as possible in the apparatus used, which must be capable of transmitting the desired combination of normal and shear stresses to the sample. The main disadvantage of the ring shear is the possible non-uniform mobilisation of shear stress across the sample. In addition, for purposes other than the measurement of residual strength it has all the disadvantages of the shear box, such as high local concentration of strain and uncertainty about the directions of the principal stresses as the test proceeds.

Bishop et al [1971] have presented a comprehensive review of the development of a ring shear. The principal features of various forms of torsion and ring shear are shown in Figures 2.1 and 2.2 respectively.
<table>
<thead>
<tr>
<th>SHAPE OF SURFACE</th>
<th>LOADING SYSTEM</th>
<th>SAMPLE TYPE</th>
<th>REFERENCE</th>
</tr>
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<tbody>
<tr>
<td>CIRCLE</td>
<td>SOLID CYLINDER LOADED NORMALLY, LOWER PLATE TWISTED</td>
<td>A.I.C.E. (1971)</td>
<td></td>
</tr>
<tr>
<td>CIRCLE</td>
<td>SOLID CYLINDER LOADED NORMALLY &amp; TWISTED</td>
<td>STRECK (1958)</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>FEARNLEY et al. (1959)</td>
<td></td>
</tr>
<tr>
<td>CIRCLE</td>
<td>SOLID CYLINDER LOADED NORMALLY &amp; TWISTED</td>
<td>LANCER (1918)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H. HUDSON, 1939)</td>
<td></td>
</tr>
<tr>
<td>CIRCLE</td>
<td>SOLID DISC LOADED NORMALLY &amp; TWISTED</td>
<td>SEMENY &amp; KRAMER (1966)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>LA CARRA (1970)</td>
<td></td>
</tr>
<tr>
<td>CIRCLE</td>
<td>SOLID DISC LOADED NORMALLY, ANNUUS TWISTED</td>
<td>RIDGWAY (1970)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H. HUDSON, 1939)</td>
<td></td>
</tr>
<tr>
<td>CIRCLE</td>
<td>SOLID DISC LOADED NORMALLY &amp; TWISTED</td>
<td>CHARI (1956)</td>
<td></td>
</tr>
<tr>
<td>CYLINDER</td>
<td>HOLLOW CYLINDER LOADED RADially &amp; TWISTED</td>
<td>CIESIELING &amp; US. ENGINEER OFFICE, BOSTON, MASS.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(H. HUDSON, 1939)</td>
<td></td>
</tr>
</tbody>
</table>


FIGURE 2.1: PRINCIPAL FEATURES OF VARIOUS FORMS OF TORSION AND RING SHEAR APPARATUS. SOLID CYLINDER AS DISC OR HOLLOW CYLINDER
### Figure 2.2: Principal Features of Various Forms of Torsion and Ring Shear Apparatus. Annular Disc

<table>
<thead>
<tr>
<th>Shape of Failure Surface</th>
<th>Loading System</th>
<th>Sample Type</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Unconfined Annular Disc Loaded Normally &amp; Twisted</td>
<td>Cooling &amp; Smith (1933, 1936)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Grune &amp; Meffert (1924)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Theemann (1933)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Woodside (1937, 1938)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Woodside &amp; Kaufman (1952)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Zemanek Wolfssell (1956)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>La Catta (1970)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Lavoie (1984)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Carr &amp; Walker (1984)</td>
</tr>
<tr>
<td>Annulus</td>
<td><img src="image-url" alt="Diagram" /></td>
<td>Annular Disc Loaded Normally &amp; Twisted</td>
<td>Scarlett &amp; Todd (1984)</td>
</tr>
</tbody>
</table>

Bishop et al (1971)
At the present time, there are three well known ring shear apparatuses. These are:

1. Bishop ring shear.
2. Harvard ring shear.

The first and second types are specialist apparatus used to measure the residual strength of clay mainly for research work. They are still rather uncommon, complicated and expensive. These features have prevented the test from becoming a routine procedure in commercial laboratories. The Bromhead type gave a marked advancement in this direction. In general, the Bromhead apparatus is simple in design and inexpensive in construction.

Tests on both remoulded and undisturbed samples can be made quite rapidly and the results are comparable with those obtained from the much more sophisticated equipment.

2.3.3.1 Bishop ring shear
This type of ring shear has been built jointly by the Norwegian Geotechnical Institute and Imperial College, University of London. The development of this strain-controlled ring shear has presented a significant contribution to the advancement of the measurement of residual strength. The general layout of the apparatus is shown in Plate 2.1, and full details are presented by Bishop et al [1971]. The sample (outside diameter = 15.24 cm, inside diameter 10.16 cm, initial thickness = 1.9 cm) can be subjected to a maximum nominal normal stress of 1000 kPa and a maximum nominal shear stress of 500 kPa. The annular sample is laterally confined between two pairs of rings and is
loaded normally through annular platens. The upper and lower confining rings are held together by locking screws, which are removed before shear commences to allow for a confining gap. Vertical normal load on the sample is maintained by dead weight. Samples are sheared by steadily rotating the lower half while the upper half reacts against the torque arm.

Generally, the main difficulty in carrying out a ring shear test is that of maintaining a small but adequate clearance between the two pairs of metal rings enclosing the upper and lower sections of the annular test specimen, in order to minimise metal to metal friction and loss of soil during shear. A novel method of overcoming this difficulty was proposed by the senior author. This was achieved by controlling the confining ring gap by using four pairs of screws to connect the upper confining ring to a yoke, so that the ring can be raised or lowered by differential screws on the crosshead.

2.3.3.2 Harvard ring shear
La-Gatta [1970] has developed a new rotational shear machine at Harvard University, the general layout of which is shown in Plate 2.2. It is composed of four units:

1. The shearing unit. It can accommodate 7.11 cm diameter disc-shaped specimens or annular samples of 7.11 cm outside diameter and 5.08 cm internal diameter. The specimen thickness may vary from 0.1 cm to 2.5 cm.

2. Vertical loading system. A dead weight loading system allows a maximum vertical stress ($\sigma_v$) of 10,000 kPa to be applied to the disc-shaped specimen and 20,000 kPa to the annular specimen.
Top View

scale: .5" = 1"

PLATE 2.2: HARVARD ROTATION SHEAR MACHINE
HARVARD ROTATION SHEAR MACHINE PARTS LIST

1. Top plate
2. Adjustable transducer clamp and transducer
3. Locking nut
4. Adjustable restraining arm
5. Vertical post
6. Moment transfer plate
7. Hollow spacer
8. Loading platen
9. Porous stainless steel discs
10. Confining ring
11. Confining ring support pins
12. Confining ring support pins
13. Lucite reservoir
14. Turntable
15. Worm gear
16. Rotating base
17. Stationary base
18. Tilt dial
19. Four-point contact radial bearing
20. Thrust bearing
21. Centering pin
22. Teflon bushing
23. Stainless steel loading platen
3. Torque application. Torque is applied to the specimen by a mechanical transmission system which provides ten speeds of rotation varying from one revolution in 43 minutes to one revolution in 28 days.

4. Moment measuring system. Two force transducers are located in the shearing unit to measure a force couple from which the shear stresses are computed.

Further details are presented by La-Gatta [1970].

2.3.3.3 Bromhead ring shear
In 1979, Bromhead developed a simple ring shear apparatus, which was built and evaluated at Kingston Polytechnic. It is now commercially available and known as the Bromhead Ring Shear. A general layout of the machine is shown in Plate 2.3.

As a general principle, an annular soil sample 5 mm thick, with inner and outer diameters of 70 mm and 100 mm respectively, is confined radially between concentric rings. It is compressed vertically between porous bronze loading platens by means of a lever loading system and dead weights.

A rotation is imparted to the base plate and lower platen by means of a variable speed motor and gearbox driving through a worm drive. This causes the sample to shear, the shear surface forming close to the upper platen (which is artificially roughened to prevent slip at the platen/soil interface). When this apparatus appeared commercially residual strength tests gradually became a routine test in different commercial laboratories in the UK due to the simplicity and inexpensive nature of the apparatus.
BROMHEAD RING SHEAR PARTS LIST

1. Consolidation dial gauge arm
2. Load hanger
3. Lever loading arm
4. Hinged gear cover
5. Proving ring adaptors
6. Bearing adjustment rods
7. Torque arm assembly
8. Proving ring bracket
9. Location stops
10. Loading yoke
11. Turret assembly
12. Yoke screw adjustment
13. Counterbalance weight
14. Gearbox
15. Oil filler plug
16. Oil level plug
17. Gear change lever
18. Quick release clamps
19. Parking stops
20. Motor
21. Clutch
22. Control panel
23. Adjustable degree scale
24. Confining ring assembly
25. Knurled nuts
26. Lower platen clamping screws
27. Porous bronze loading platens
28. Soil
29. Centring pin
30. Lever arm stop
31. Perspex water bath
32. Hand wheel
33. Worm wheel
Experience with this apparatus will be discussed and evaluated extensively later on in this thesis.

Skempton [1985] reported that the ring shear tests described by Bishop et al [1971] tend to give residual strengths for high clay fraction material, which are somewhat lower than the field values by an average value of 1.5° in the angle of shearing resistance. Similarly Chandler [1984] stated that such differences may reach 2° to 3° on $\phi_r'$ ($C_r' = 0$). He attributed that to the additional concentration caused during ring shearing.

However Bromhead and Curtis [1983] indicated that with the Bromhead ring shear machine, agreement with field residual strength is obtained for London clay. If that is absolutely true, agreement could be obtained by the Bishop type also, since Bromhead [1979] found that the two apparatuses gave almost identical results on two samples of Gault clay from Folkestone Warren. Again in 1986 they stated that any difference between ring shear results and field results might disappear if a large body of data was taken into account to reduce possible experimental errors.

On the other hand Cunningham [1986] found from his personal experience with both ring shears that the Bromhead ring shear produced less scatter than the Bishop ring shear, in addition to being less time consuming and considerably cheaper.

Table 2.3 shows examples of the published data using the well known ring shear apparatuses. It appears from the London clay results, that close results could be obtained using different types of ring shear apparatus.
### TABLE 2.3: RESIDUAL STRENGTH DATA OBTAINED BY RING SHEAR APPARATUS

<table>
<thead>
<tr>
<th>Investigators</th>
<th>Year</th>
<th>Soil Type</th>
<th>Sample Type</th>
<th>Soil Parameters</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hvorslev</td>
<td>1952</td>
<td>Panama</td>
<td>Un.</td>
<td>0 kPa 4°</td>
<td>LL=145, PI=69, OC</td>
</tr>
<tr>
<td></td>
<td>1960</td>
<td>Much Cake</td>
<td>Un.</td>
<td>0 kPa 9.3°</td>
<td>LL=145, PI=69, NC</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Providence Clay</td>
<td>Un.</td>
<td>0 kPa 10.5°</td>
<td>LL=104, PI=69, OC</td>
</tr>
<tr>
<td>La Gatta</td>
<td>1970</td>
<td>London Clay</td>
<td>Un.</td>
<td>0 kPa 11.3°</td>
<td>LL=71.5, PL=22.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Olcaracha Shale</td>
<td>Dis.</td>
<td>0 kPa 8.3°</td>
<td>LL=156, PL=42</td>
</tr>
<tr>
<td>Bishop et al</td>
<td>1971</td>
<td>Blue London Clay</td>
<td>Dis.</td>
<td>0 kPa 9.5°</td>
<td>LL=72, PL=29, CF=57, Wraysbury</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Brown London Clay</td>
<td>Un.</td>
<td>0 kPa 9.4°</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Dis.</td>
<td>0 kPa 10.0°</td>
<td>LL=6.6, PL=24</td>
</tr>
<tr>
<td>Bromhead</td>
<td>1979</td>
<td>Gault Clay</td>
<td>Dis.</td>
<td>0 kPa 9.4°</td>
<td>LL=69, PL=27, CF=10</td>
</tr>
<tr>
<td>Bromhead &amp; Curtis</td>
<td>1983</td>
<td>London Clay</td>
<td>Dis.</td>
<td>0 kPa 11.0°</td>
<td>LL=77, PL=27, CF=45</td>
</tr>
</tbody>
</table>
2.4 SUMMARY OF THE FACTORS AFFECTING RESIDUAL STRENGTH OF CLAY

2.4.1 Effect of Mineralogy

Mineralogy, rather than the clay fraction or the index properties of clay, has been found by many investigators to be the most important factor controlling residual strength [Horn and Deere, 1962; Skempton, 1964, 1985; Petley, 1966; Chattopadhyay, 1972; Kenney, 1967, 1977, Townsend and Gilbert, 1973, 1974, 1976, Steward and Cripps, 1983, and others].

Kenney [1967] found that mineral shape is an important factor governing the residual shear strength. For massive minerals aggregates of rounded particles had smaller values of $\phi_r$ than aggregates of angular particles. The residual strength coefficient of massive minerals is dependent on particle shape, and independent of particle size. The residual strength of clay containing hydrous mica and montmorillonite is lower than clay containing massive minerals.

Kenney [1977] denoted the residual strength of mineral mixtures by $R_\phi$, where

$$R_\phi = \frac{\tan \phi_r \text{ mixture} - \tan \phi_r \text{ clay mineral}}{\tan \phi_c \text{ massive mineral} - \tan \phi_r \text{ clay mineral}}$$

Therefore the residual strength of mineral mixtures and natural soils is dependent on the mineral composition and relative volumes of clay-mineral matrix and massive mineral.

Ramiah and Purushothamaraj [1971] studied the effect of initial structure on the residual strength of a saturated kaolinite clay using the reversal shear box. They reported that the initial structure of a
kaolinite clay (compacted, slurried and consolidated, both isotropically and anisotropically) had practically no influence on the value of the resulting residual friction angle ($\phi_r'$) at a given normal stress.

Chattopadhyay [1972] concluded that the residual strength of a clay mineral depends not only on its mode of cleavage at larger strains but also on the types and total amount of bonding energies available along its cleavage planes at the shear zone particle contacts. Therefore kaolinite and montmorillonite possess low residual friction angles, while attapulgite exhibits a very high residual friction angle.

Skempton [1985] claimed that the clay minerals can have little effect on residual strength when the clay fraction is less than 20%, as the strength is then controlled largely by the sand and silt particles. Conversely, with clay fraction exceeding 50%, the residual strength depends almost entirely on the sliding friction of clay particles and therefore depends on their character.

In conclusion, mineralogy is the most important factor that affects residual strength since it controls the shape and porosity of particles, and therefore the shear behaviour and residual strength. Clay minerals which have a plate-like shape, like montmorillonite, have a low residual strength, whereas soils containing primarily micaceous minerals and massive minerals could be expected to have a higher residual strength.
2.4.2 Effect of Normal Effective Stress Level

Most of the investigators have shown that for most clays the relationship between residual strength and normal effective stress is curved [Herrmann and Wolfskill, 1966; Kenney 1967; Skempton and Petley, 1967; La-Gatta, 1970; Garga, 1970; James, 1970; Mesri and Oslon, 1970; Bishop et al, 1971; Cullen and Donald, 1971; Chattopadhyay, 1972; Bucher, 1975; Chandler, 1976; Townsend and Gilbert, 1974, 1976; Chowdhury and Bertoldi, 1977; Bromhead, 1978; Hutchinson, Bromhead and Lupini, 1980; Charles, 1982; Bromhead and Curtis, 1983; Atkinson, 1984; Hawkins and Privett, 1985; Anderson and Suliaman, 1987; Anderson, 1988 and many others]. Figure 2.3 presents a comprehensive survey of such variations.

The curvature is most pronounced at an effective normal stress ($\phi_r'$) below 150-200 kPa and sometimes greater. This stress corresponds to the point on the envelope which shows a transition from a curved to a straight line.

Kenney [1967] has shown that soils with a high montmorillonite content show a variation of $\phi_r'$ with effective normal stress, whereas soils composed of heavy non-clay minerals do not. Kenney's observation is highly related to the perfect clay particle orientation usually produced at stresses beyond 200 kPa.

Bishop et al [1971a] found that there is a variation in $\phi_r'$ with effective normal stress for Brown London clay, whereas there is not for Blue London clay. He related the variation in the former type to the fact, found by Kenney [1967], that Brown London clay has a high montmorillonite content.
FIGURE 2.3 VARIATION OF $\phi'_r$ WITH $\sigma'_n$ OBTAINED FROM DIFFERENT PUBLISHED DATA.
Residual friction angle ($\phi_r'$) and residual friction coefficient ($\tau_r'/\sigma_n'$) versus effective normal stress ($\sigma_n'$) for various natural soils. (1) and (22) Bishop et al (1971); (2) Skempton (1964); (3) Chandler (1966); (4) and (20) Chowdhury and Bertoldi (1977); (7) and (17) Hutchinson et al (1973); (8), (9) and (12) Skempton and Petley (1967); (10) and (16) Petley (1980); (11) and (35) Petley (1969), in Bishop et al (1971); (13) and (19) Hutchinson et al (1980); (14) and (15) Esu and Calabresi (1969); (21), (25), (26) and (31) Chandler (1976); (23) and (24) Garga (1970) in Bishop et al (1971); (27) Insley et al (1977); (28) Blondeau and Josseaume (1976); (29) Calabresi et al (1980); (30) Maugeri (1976); (32) Cancelli (1979); (33), (34) and (36) Al-Layla (1970); (37) Hutchinson and Costelow (1976); (5), (6), (18), (38) and (39) Coteochia and Federico (1980).
Chattopadhyay [1972] found that, in the low normal stress range (0-200 kPa), the true friction angle ($\phi_r'$) of every clay mineral decreases markedly as the normal stress is increased, and above a certain normal load (approximately 200 kPa) the true $\phi_r'$ becomes independent of the normal stress.

Chandler [1977] claimed that back analysis techniques of individual segments of landslides enabled the degree of curvature of the residual envelope to be established since actual values of the mobilised shear strength could be determined for each slice. Plotting the actual residual strength against the average normal pressure on each slide, the whole residual strength envelope could be furnished.

Chowdhury and Bertoldi [1977] found that the curved residual strength envelope is associated with soils of high clay fraction and high plasticity. Thus this is in turn related to the question of perfect clay particle orientation. These observations highlight the following facts:

1. Reference to average residual angles is not very useful because the $\phi_r'$ value is not necessarily independent of normal stress. A decrease in residual friction angle or residual strength as the effective normal stress increases is usually observed. Such dependency is more marked with low values of effective normal stress.

2. As the failure envelope becomes a straight line with increasing $\sigma_n'$, $\phi_r'$ decreases. The decrease in $\phi_r'$ is presumably associated with an increase in the degree of orientation of the clay particles under high normal stress.
3. As most of the landslides examined by Skempton [1964, 1985], Chandler [1974, 1977, 1979] and others have a relatively shallow depth, the variation of $\phi'_r$ with effective normal stress should be taken into consideration (in other words, the complete residual failure envelope should be taken into consideration) when analysing such slides.

The subject of the variation of residual friction angle with effective normal stress will be examined and evaluated in detail later on in this thesis, as it is the main objective of this research.

2.4.3 Effect of Clay Fraction and Index Properties

Many investigators have proposed correlations between the residual friction angle and clay fraction or index properties in order to find a simple and easy tool for estimating residual strength.

Skempton [1964] concluded that the residual friction angle depends on the nature of the clay particles, and that there is a most definite tendency for $\phi'_r$ to decrease with increasing content of clay particles.

Skempton [1964, 1985] presents a correlation between $\phi'_r$ and clay size fraction (Figure 2.4), specifying [Skempton, 1985] approximate bounds for plasticity index divided by clay fraction (PI/CF) and pointing out that lower $\phi'_r$ values are expected for clay shales containing montmorillonite and having PI/CF > 1.5.

Similar correlations have been presented by Petley [1966], Smart [1970], see Figure 2.5, Blondeau [1973] and Hawkins and Privett [1985].
FIGURE 2.4: RELATION BETWEEN $\phi_r$ AND CLAY FRACTION
FIGURE 2.5: VARIATION OF TAN $\phi'$ VERSUS CLAY CONTENT (Smart 1970)
Kenney [1967] concluded that residual strength is primarily dependent on mineral composition and that it is not related to plasticity or to clay size fraction. However Skempton [1985] has incorporated Kenney's statement by asserting that any relationship between residual strength and clay fraction covering a wide range of particle size should have essentially the same mineralogy throughout.

Similarly Hawkins and Privett [1986] stated that in making such correlations, the experimental determination of the actual proportion of clay grade material should be examined very carefully and also that the effect of mineralogy should be considered.

Voight [1973] has claimed that since residual strength is dependent primarily on the mineralogy of the soils [Kenney, 1967] and these mineralogical factors affect Atterberg index parameters, the plasticity index appears to be a useful field guide to the important engineering property of residual strength of natural soil (Figure 2.6).

Kanji [1974] also demonstrated a relationship existing between $\phi'_r$ and plasticity index (Figure 2.7). He presented a relationship which can be expressed as:

$$\phi'_r = \frac{46.6}{(PI)^{0.446}}$$

Vaughan and Walbanke [1975] have presented a similar relationship for a number of British sedimentary clays and Boulder clays (Figure 2.8). Other similar correlations have been presented by Blondeau [1973], Bucher [1975] and Seycoek [1978].

69
FIGURE 2.6: RELATION BETWEEN RESIDUAL STRENGTH AND PLASTICITY INDEX (Voight, 1973)
FIGURE 2.7: RELATION BETWEEN PI AND $\phi_r$ (Kanji, 1974)

FIGURE 2.8: RELATION BETWEEN PI AND $\phi_r$' ( Vaughan and Walbanke, 1975)
Cancelli [1977] and Mesri and Cepeda-Diaz [1986] presented a correlation between $\phi_r'$ and the liquid limit of different soils, shown in Figures 2.9 and 2.10 respectively.

Saito and Miki [1975] proposed a correlation between $\phi_r'$ and the 'plastic ratio' $I_p/W_p$.

As shown before (in Section 2.4.2), for most clays the relationship between residual strength and normal effective stress is non-linear. Skempton [1985] stated that when comparing one clay with another it is best to fix on a 'standard' stress. Thus the value of $\phi_r'$ at this stress can be taken as a characteristic parameter of clay. Hawkins and Privett [1985] have presented such correlations which are presented in Figure 2.11.

Figure 2.11a shows the variation of $\phi_r'$ with plasticity index at four effective normal stresses. Ingold's [1975] line is included for comparison. Figure 2.11b shows the relationship between $\phi_r'$ and the percentage grain size less than 2 µm (clay fraction). The zone given by Skempton [1964] is shown by dashed lines for comparison. This type of presentation is considered useful.

Finally, it can be concluded that there is no correlation which can act as a substitute for actual residual shear testing.

2.4.4 Effect of Pore Water Chemistry

The effect of pore water chemistry on the residual strength of clay has not received much attention, however some workers have covered part of the problem.
FIGURE 2.9: RELATION BETWEEN $\phi_x'$ AND LL (Cancelli, 1977)

FIGURE 2.10: RELATION BETWEEN $\phi_x'$ AND LL (Mesri and Cepeda-Diaz, 1986)
FIGURE 2.11: a) RELATION BETWEEN $\phi^r$ AND PL
b) RELATION BETWEEN $\phi^r$ AND CF
(Hawkins and Privett, 1985)
Kenney [1967, 1977] found that the residual strength properties of clay minerals and mineral mixtures are influenced by system chemistry. The higher the cation concentration and the larger the valence, the greater the increase in residual strength was attributed to a net increase of attraction between the clay particles in parallel arrangement.

La-Gatta [1970] has shown however that the pore fluid concentration has only a slight effect on residual strength results.

Ramiah, Dayala and Purushothamanaray [1970] investigated the influence of varying pore water chemistry on the residual strength of silty clay. They found that a change from flocculating to dispersive conditions reduced the residual friction angle from 33° to 28° for soil having Ip = 17 and CF = 8%.

Chattopadhyay [1972] extended the concept of effective stress to include the interparticle stress due to the physico-chemical environment. Mathematically this was furnished by adding the term (R-A) to the Coulomb-Terzaghi relationship, so residual shear strength can be better defined by the equation

\[ \tau_{res} = [\sigma_n - (R-A)] \tan \phi' \]

where (R-A) is the net interparticle stress due to the physico-chemical environment. Therefore the effective stress is not the only stress governing the residual strength of clay minerals. For inactive clay minerals (such as kaolinite and attapulgite), the governing stress is the grain to grain contact stress only, and the conventional Coulomb-Terzaghi relationship for residual strength, as defined by
\[ \tau_{\text{res}} = c'_{\text{n}} \tan \phi'_{\text{r}} \] is adequate to express the residual shear strength. However, the case of sodium-montmorillonite showed higher net (R-A) stresses because of its high cation exchange capacity and high specific surface.

No firm general conclusion can be drawn herein and it seems that there is still a need for more research to determine the effect of pore water chemistry on the residual strength of clay.

### 2.4.5 Effect of Sample Conditions, Sample Type and Stress History

Most of the investigators following the Fourth Rankine Lecture have confirmed the fact reported by Skempton, that the sample conditions (remoulded, undisturbed, with or without a pre-cut plane) and stress history have no appreciable effect on the measurements of residual strength of soils [Petley, 1966; Kenney, 1967; La-Gatta, 1970; Garga, 1970; Bishop et al, 1971a; Cullen and Donald, 1971; Ramiah and Purushothamaraj, 1971; Early and Skempton, 1972; Calabresi and Manfredini, 1973; Townsend and Gilbert, 1973, 1974, 1976; Bucher, 1975, Chowdhury and Bertoldi, 1977; Bromhead and Curtis, 1983].

Skempton [1964], as mentioned before, stated that reversal shear box tests on undisturbed and slurried clays gave identical answers, and that shear box tests on pre-cut samples gave slightly lower results.

Similarly, De Beer [1967] reported strain-controlled ring shear tests performed on Boom clay using pre-cut samples. He found that the effect of pre-cutting the sample (before consolidation) reduced the residual friction angle. The same observation was found also by Chowdhury and Bertoldi [1977].
Qullen and Donald [1971] concluded that the reversal shear box is a convenient apparatus for measuring both peak and residual strength of soils. A number of variables of test technique were isolated and examined to determine their effects on the measured residual strength, including speed variations, shear box travel, handwinding and pre-cut failure planes. They found that the test results were insensitive to significant changes in the test procedure, and that the residual friction angle is a function of the water content at the residual state (for a given normal load).

Ramiah and Purushothamaraj [1971] studied the effect of initial structure on the residual shear strength of saturated kaolinite clay using the reversal shear box. They reported that the initial structure of a kaolinite clay (compacted, slurried and consolidated isotropically and anisotropically) had practically no influence on the value of the resulting residual friction angle of $\phi_r'$ at a given normal effective stress.

Townsend and Gilbert [1973, 1974, 1976] have examined the effects of equipment type and associated procedure, specimen type and loading sequence. They found that the friction angle ($\phi_r'$) is independent of specimen preparation procedure and density. Comparable results were obtained on intact, pre-cut and remoulded samples.

The residual shear strength of any clay is the lowest shear strength obtained when the clay particles orient themselves parallel to the direction of shearing. Therefore the initial structural orientation of the clay particles, which is related to the sample condition, sample type and stress history should have no effect on the residual strength results. The available observations and findings obtained by
the many investigators mentioned in this section have proved this fact to a satisfactory degree.

2.4.6 Effect of Initial and Final Water Content

The initial moisture content has been found by many investigators [Horn and Deere, 1962; Skempton, 1964; Cullen and Donald, 1971; Chowdhury and Bertoldi, 1977; and others] to have no influence on the measured residual strength. The values of the final moisture content in the shear zone at the residual state are likewise independent of the initial moisture content [Bishop et al, 1971; Kenney, 1967; and others].

In principle, the shear zone is a surface of strongly orientated particles and outside this zone there is negligible particle orientation. These changes result in a change in the moisture content on the slip surface as it is related to the structural orientation of the clay particles. Therefore the residual shear strength should be related to final water content on the shear zone rather than the initial water content or the water content outside the shear zone.

2.4.7 Effect of Shear Rate

The drop-off in strength to the residual value is primarily due to the orientation of clay particles in the shear zone. This orientation of clay particles might be expected to be independent of the rate of shearing. However the rate of shearing should not be fast enough to lead to the development of pore water pressure. Therefore as long as the shearing rate is within the range of the drained condition, the rate of shearing should have no effect on the measured residual strength. Such a fact has been confirmed by many investigators, who found that the variations in strength within the usual range of slow
laboratory tests are negligible [Petley, 1966; Kenney, 1967; Skempton and Hutchinson, 1969; Olllen and Donald, 1971; Lupini, 1980 and Skempton, 1985].

Skempton [1985] claimed that variation in strength within a range of 0.002-0.01 mm/min is negligible. On the other hand La-Gatta [1971] found that the residual strength of London clay increased by 18% when the rate of displacement was increased from $5.6 \times 10^{-3}$ mm/min to $2.8 \times 10^{-3}$ mm/min.

From field observations on reactivated landslides and mud-flows, it is known [Skempton and Hutchinson 1969] that the highest daily rate of movement is in the order of $5.78 \times 10^{-3}$ mm/min and the lowest rate is about $6.34 \times 10^{-7}$ mm/min. The strength variation over this entire range was found to lie between -3 and +5%. On the other hand Senebenelli and Ramirez [1969, 1971] concluded that the shear rates should be extremely slow if reliable residual strength results are to be obtained.

The effect of the usual slow rate of shearing on the measured residual strength is included in this research and it will be discussed later.

Apart from the laboratory slow rate of shearing which is usually carried out for residual strength testing, the effect of fast rates of shearing experienced in earthquakes have been investigated by Lemos, Skempton and Vaughan [1985] and Vaughan, Lemos and Tika [1985].

Lemos et al [1985] have investigated the effect of earthquake loading on shear surfaces in slopes. The soil was first sheared slowly to establish the drained residual strength (Figure 2.12). It was then
FIGURE 2.12: TYPICAL BEHAVIOUR DURING FAST SHEAR STAGES (Lemos et al., 1985)
allowed to rest. Fast shearing was performed with continuous monitoring of stress and displacement.

Three strengths can be identified: a threshold strength at which displacement on the shear surface recommences (b) a fast maximum strength (c), and a fast minimum strength (d). Shearing was then stopped and shear stress reduced immediately to prevent any displacement by creep. After a rest for consolidation, shearing was continued at the slow drained rate. A new slow peak strength (e) was generally observed, which was higher than the ultimate residual strength (a). This strength (a) was recovered after further displacement. They found that strengths (e) and (a) are usually the same in soils showing turbulent shear, in which residual strength does not depend on particle orientation, but they only occur at fast rates of displacement. The gain in strength (e) is due to disordered of the orientated clay particles in the shear zone. The presence of rotund particles facilitates this process, so the increase in strength will be higher. They concluded that with soils showing sliding shear, there is no risk that movements will continue post-earthquake unless sufficient excess pore pressure has been induced in the mass of the slope, whereas with transitional residual shear, fast displacement causes an eventual drop in strength to less than the slow drained value. If this strength loss were induced during an earthquake, it could lead to large, fast and potentially catastrophic movements.

In a similar research, Vaughan, Lemos and Tika [1985] concluded the following:
1. With soil samples having a clay fraction greater than 28%, a positive rate effect could be observed. This means that a significant initial increase in strength is usually developed when the soil is initially under a slow residual strength and suddenly the loading becomes fast. This strength increase is available to resist the effect of earthquake shaking.

2. Soils with very low clay contents behave like sand and show a negligible influence of displacement rate on ultimate strength.

3. Soils having clay fractions varying between 12% and 40% may show a negative rate effect, although soils in this grading range do not necessarily show this effect.

Later on Skempton [1985] concluded that, for clay, the increase in strength becomes pronounced at rates exceeding 100 mm/min, when some qualitative change in behaviour occurs. This is probably associated with disturbance of the original ordered structure producing a 'turbulent' shear.

2.4.8 Effect of Temperature
Little investigation has been made to study the effect of temperature on the residual strength of clay.

In the laboratory Bucher [1975] found that for temperatures ranging between approximately 10°C to 60°C the residual friction angles of two medium plasticity clays were substantially unaffected.
In the field, Voigt and Faust [1982] reported that frictional heat is inevitably produced along landslide slip zones. Under specific circumstances fluid pressure in the slip zone can rise as a consequence of this frictional heating. The effect of fluid pressure rise is to induce loss of strength during sliding, in effect a decrease in kinetic frictional resistance. Under some circumstances loss of strength can occur rapidly. Such cases are potentially hazardous, in as much as moderate slide motion can be converted into catastrophic descent.

In conclusion, knowledge of residual strength values over a temperature range similar to that produced during landslide movement is required.

2.5 SUMMARY

The residual strength of soils is an important parameter in assessing the stability of soil slopes. Measurement of residual strength requires a large displacement of the soil along a failure surface. It seems probable that this phenomenon is the cause of many failures both in natural and man-made slopes after they have remained stable for many years. If the shear strength of the clay gradually decreases towards residual value, a time comes when the strength is insufficient to resist the disturbing forces, and failure results.

There are several factors that affect the measured value of residual strength of clay.
Mineralogy was found to be the most important factor that affects residual strength. Pore fluid chemistry also influences the measured strength. Sample preparation methods, stress history, initial water content and rate of shearing have no appreciable effect on residual strength of soils.

The general shape of the residual strength failure envelope is curved. The actual cause of the curvature was obscure for most of the investigators. Therefore the cause, shape and significance of this curvature is the main objective of this research.
3.1 REVERSIBLE SHEAR BOX

3.1.1 Introduction
The shear box apparatus is the oldest form of shear test and was first used by Coulomb in 1776. Briefly, the soil is held in a box that is split across its middle. A confining force is applied followed by the application of a shear force so as to cause relative displacement between the parts of the box. The magnitude of the shear is recorded as a function of the shear displacement. The normal load is applied by dead weight, whereas the shear force is often applied by a motor acting through gears (strain-controlled tests).

The shear box is often referred to as the direct shear test because the shear stress at failure is often plotted against the normal stress applied, thus directly defining the Mohr-Coulomb failure envelope.

As the conventional shear box has a limited shearing travel, the effect of a large displacement required to define the residual shear strength of soil can be obtained by returning the split box to its starting position after completing the extent of its travel and shearing again, as described in Chapter 2. This process can be repeated a number of times until a steady (residual) value of shear strength is observed. Therefore this type of modified shear box is known as the "reversible shear box".
3.1.2 Modifications

The shear box has been modified to enable the direction of shear to be reversed automatically so that the specimen may be returned to its initial position prior to the commencement of a further forward shear. The mechanism operates to control the movement of the shear box and a switching device which automatically reverses the drive motor between pre-set limits of travel. A general view of the modified apparatus is shown in Plate 3.1.

Figure 3.1 shows the circuit diagram for the control system of the modified shear box. The modifications, which were entirely electrical, comprised:

1. Reversing switch: the original limit switch was replaced by another automatic switch which is capable of changing the direction of shearing when the limit in either direction is reached.

2. Channel interface socket: this additional socket was introduced to inform the data-recording system when the shearing limit is reached in either direction so that the recording system takes account of the new shearing cycle.

3.1.3 Normal Loading System

The vertical normal load on the sample is maintained by dead weights applied via a 10:1 ratio lever arm. The lever's fulcrum on the base of the apparatus is adjustable vertically, so that the lever can be levelled to accommodate changes in sample thickness. An adjustable counterweight is provided to balance the lever and main shaft.
PLATE 3.1: REVERSIBLE SHEAR BOX
FIGURE 3.1 CIRCUIT DIAGRAM OF THE CONTROL SYSTEM FOR THE MODIFIED SHEAR BOX
3.1.4 Shear Force Measuring System
An LVDT (Linear Variable Displacement Transducer) having a total range of 12 mm is fitted to the dial gauge (0.002 mm/div, 12 mm travel) of the proving ring (2.0 kN capacity) was used to measure the shearing force. The body of the transducer is clamped to the backing plate of the dial gauge and the core follows the linear compression of the proving ring. Calibration of the transducer is carried out with the proving ring in its normal position on the shear box frame by compressing the ring to a desired value using the hand control on the screw jack.

3.1.5 Horizontal Displacement Measuring System
A dial gauge (0.01 mm/div, 25 mm travel) was used to measure the horizontal travel of the shear box. The complete assembly is mounted horizontally on the frame of the shear box apparatus.

Horizontal displacements can also be calculated with a high degree of accuracy by multiplying the shearing rate of the test by the corresponding elapsed time.

3.1.6 Vertical Deformation Measuring System
An LVDT having a total range of 12 mm is fitted to the plunger of the dial gauge (0.002 mm/div, 12 mm travel) used to measure the vertical deformation during the consolidation and shearing stages.

The complete assembly is mounted vertically onto the top of the loading yoke of the apparatus.

Direct dial gauge readings are taken from time to time during testing for all of the measuring systems mentioned above, for checking purposes.
3.2 RING SHEAR APPARATUS

3.2.1 Introduction
As shown before most of the previous work on the measurements of residual strength of soil has shown that the ring or torsion shear test is a very useful method.

The expensive and time consuming nature of the ring shear, La-Gatta [1970] and Bishop et al [1971], has prevented the test from becoming a routine procedure in commercial laboratories. The development of the Bromhead apparatus has led to an advancement in this direction, since this apparatus is simple, robust and inexpensive. For more details see Bromhead [1979].

This type of ring shear apparatus was used in the current research at Loughborough University, see Plate 3.2.

The testing programme of this research was initiated by testing Lias Clay using the original Bromhead apparatus in the way recommended by Bromhead [1979].

Lias clay is a well known British clay. The soil is generally a blue fissured highly overconsolidated clay. Geological, geotechnical and chemical properties of this soil are presented in Sections 4.1.2 and 4.1.3 of this thesis. Many trial tests were carried out using the original apparatus designed by Bromhead [1979]. As soon as shearing commenced, the readings of the two proving rings became unbalanced and eventually one ring showed a value of zero. Some imbalance of the two proving rings is usually acceptable. Such imbalance is usually caused as a result of a non-uniform sample or by poor centering of the
PLATE 3.2: BROMHEAD RING SHEAR APPARATUS
mechanical parts of the apparatus. Differences in the stiffness of the commercial proving rings may also have an effect. However excessive imbalance implies friction on the centre pivot and/or tilting of the loading platen. Different sample preparation techniques and strain rates were tried but the problem persisted. Attempts were made to overcome this imbalance in shear forces by carefully equalising the two proving rings before shearing. Also, small pieces of rubber were introduced between the tip of each proving ring and the torque arm in order to reduce the stiffness of the proving ring system. It was hoped that this would reduce the effect of any small eccentricity of rotation. In all these trials no satisfactory measurement of residual strength was achieved.

Continuous monitoring of the test suggested that there was excessive water penetration from the water bath to the top surface of the sample through the clearance gap between the top platen and lower platen (sample container) necessary for correct functioning of the apparatus.

Examination of the top surface of the sample seemed to indicate that adhesion of the clay and the porous bronze was unreliable, and that a kind of remoulding was taking place on part of the surface.

3.2.2 Modifications
The modification concept adopted was to hold the top and bottom surfaces of the sample adjacent to the platens by means of vanes distributed uniformly on the top and bottom platens (Plate 3.3). Each platen was modified by the addition of 24 vanes, these being 3 mm in height and made from stainless steel (0.93 mm thick) glued into slots machined into the porous bronze drainage rings. The bottom vanes have a width of 15 mm (equal to that of the sample) and the top vanes have
a width of 14.4 mm to give 0.3 mm clearance on each side during rotation (see Plate 3.3). As a consequence of adding vanes, two secondary modifications are required:

1. The depth of the sample container is increased from 5 mm to 10 mm. This means that drainage lengths and hence pore pressure dissipation times are increased, but this does not cause a problem with the slow rates of strain commonly used in residual strength work (see Plate 3.4).

2. The shape of the torque arm is changed from rectangular in side elevation (16 mm high) to tapered (22 mm at the outer end as shown in Plate 3.5) to allow for the increase in sample thickness.

Tests using this modified apparatus gave almost similar readings on each proving ring with a maximum difference of 1-3%. Such a difference could be referred simply to the difference in stiffness of the two proving rings.

The main advantage of this modification is that a single flat failure surface in the centre of the sample is more likely to be produced, and this can be seen in Plate 5.8 in Chapter 5 which shows the failure surface on the top platen after testing.

Further evaluation and discussion are presented later on in this thesis.
PLATE 3.3: Modified Ring Cell
PLATE 3.4: LOWER RING SHEAR PLATEN

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PLATE 3.5: TORQUE ARM OF THE MODIFIED APPARATUS
3.2.3 Normal Loading System
This system is the same as that described in Section 3.1.3 for the reversal shear box.

3.2.4 Torque Measuring System
Two LVDTs each having a total range of 12 mm were fitted to the dial gauge of the proving rings (0.5 kN capacity), which were used to compute the torque. The bodies of the transducers were clamped to the backing plate of the dial-gauge and the core followed the linear compression of the proving rings. Calibration of the transducers was carried out with proving rings in their normal positions by compressing the ring to a desired value using the hand control on the screw jack.

3.2.5 Radial Displacement Measuring System
Direct displacement measurement was taken from the graduated disc attached to the lower platen. However, direct displacements can be computed from the rate of shearing multiplied by the elapsed time. A good correlation was found between the two methods, so the second method was used due to its simplicity.

3.2.6 Vertical Deformation Measuring System
This system is exactly the same as that described in Section 3.1.6 for the reversal shear box.

Direct dial gauge readings for the measuring systems mentioned above were taken from time to time during testing for checking purposes.
3.3 CALIBRATIONS

All the dial gauges and proving rings used in this research were supplied new and midway through the research, were subjected to a full re-calibration by the original manufacturer (Wykeham Farrance Engineering Ltd).

3.3.1 Dial Gauges
Commercial dial gauges were manufactured to comply to the specified tolerances of non-linearity (BS 907:1965). Readings may be verified by mounting the gauge in a bench stand on a flat surface and inserting an engineer's slip gauge of accurately measured thickness under the anvil after setting the gauge to read zero.

3.3.2 Proving Rings
Every load ring is supplied with a manufacturer's calibration certificate. This is obtained from the calibration data by plotting each calibration force (which is achieved by applying dead loads to the loading frame) against the corresponding dial reading as a graph, and drawing a mean straight line from the origin through the points. The gradient of the line is the mean calibration value. Proving rings were calibrated for both loading and unloading sequences.

3.4 DATA RECORDING SYSTEM

The data recording equipment is a system which can convert displacements of the transducer into an electrical signal which may be stored or printed. This system is presented in Plate 3.6 and it is composed of the following units:
PLATE 3.6: Data Recording System
A. Commodore Pet, computer 8032 series
B. Wykeham Farrance, four channel interface
C. IEE to R232, interface converter
D. Commodore Tractor, printer 4032
E. Commodore, twin disc drive 8050.

The data recording system used with the reversible shear box and ring shear apparatus, includes controls for scaling the voltage output from each transducer to a potentiometric chart recorder. The chart recorder enables the progress of the test to be observed continuously and also records changes in displacement and shear strength which occur between the scanning intervals of the data logger.
4.1 LIAS CLAY

4.1.1 Sampling
Block samples (about 200 x 200 x 20 mm) were taken from the site of the Rugby cement factory at Rugby. The soil is generally a blue fissured highly over-consolidated clay with layers of limestone occurring in the clay strata.

Exposed surfaces of the soil profile were produced due to the continuous extraction of the clay for the manufacturing process. Large pieces of the soil were dug away from these surfaces and then trimmed again approximately to the size of the tins.

The samples were waxed at the site and delivered with care to the laboratory for testing.

4.1.2 Geology
The term Lias (Lyas), first used in geological work by John Strachey in 1719, is an old West of England quarryman's term, and was applied primarily to thin beds of muddy and shelly limestone which occur in the lower part of the Lower Lias. The term is now applied to the predominantly argillaceous formations of the Lower Jurassic.

Generally, this clay is a marine deposit composed of clays, shales and limestone beds. The argillaceous components of the Lias are heavily over-consolidated, something like 900 metres of superincumbent sediments having been removed during the history of the formation.
The Lias Clay of the East Midlands outcrops from the Vale of Evesham and continues through Rugby, Leicester and Melton Mowbray to the north-eastern corner of the region and thence into Lincolnshire. This belt has an average width of about 15 miles. Near Rugby the basal bed of the Lower Lias Clay is highly bituminous and pyritic, and infills hollows in the topmost bed of the White Lias. It consists of an alternation of fine-grained argillaceous limestone with grey calcite mudstones and shales. They have been extensively dug for the manufacture of lime and cement. The limestone is usually confined to some horizons within the lower zones of the Lower Lias, the rest of the formation, consisting of bluish-grey slightly calcareous mudstone and shales, is very poorly exposed. The maximum thickness of the lower Lias Clay around Rugby reaches between 200 and 230 metres.

Illite is typically the dominant mineral. Montmorillonite may be present at some locations, generally in the Lower Lias.

4.1.3 Material Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
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<td>Natural Water Content, W%</td>
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</tr>
<tr>
<td>Liquid limit, LL%</td>
<td>52</td>
</tr>
<tr>
<td>Plasticity Index, PI%</td>
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</tr>
<tr>
<td>Specific Gravity, $G_s$</td>
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</tr>
<tr>
<td>Organic Content, %</td>
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</tr>
<tr>
<td>Soluble Sulphate Content %, $SO_4^-$</td>
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</tr>
<tr>
<td>pH</td>
<td>7.7</td>
</tr>
</tbody>
</table>

The soil is composed of:

1. Fine sand = 1.0%
2. Medium to fine silt = 42%
3. Clay = 57%.
4.2 **LONDON CLAY**

4.2.1 **Sampling**
Block samples (about 200 x 200 x 200 mm) were taken from a newly excavated trench about 2 metres deep in Essex. The soil is generally Brown London Clay, one of the most widely tested British soils.

The samples were waxed on site and delivered with care to the laboratory for testing.

4.2.2 **Geology**
London Clay is a marine Eocene deposit, well described in the literature. The greater part of London Clay is a stiff, dark or bluish-grey clay which weathers at outcrop to brown. The depth of weathering typically extends to 7 to 10 metres below the ground surface where the transition to the blue clay occurs.

A characteristic of London Clay is the septaria, or concretions of argillaceous limestone, occurring as layers of nodules and, in some cases, containing numerous fossils. The lowest part of the formation is a sandy bed with black flint-pebbles and occasional layers of sandstone and is known as the Basement bed. In the vicinity of London the London Clay passes up into the more sandy Claygate Beds, but in other places these are missing and the London Clay is covered abruptly by the Bagshot Sand. It is probable that the oxidation of iron pyrites, present in the bluish clay, resulting in the formation of sulphuric acid which attacks the calcareous shells and nodules and forms crystals of gypsum, is responsible for the absence of London Clay fossils in most localities.
Illite is the dominant mineral in the London Clay formation, kaolinite is subsidiary and vermiculite occurs in traces. Montmorillonite may be present in varying amounts, generally up to 15% in the London Basin, although between 15 to 30% of this mineral may be present in the Hampshire Basin where deposition was under shallower marine conditions (James, 1970).

4.2.3 Material Properties

- Natural Water Content, W% = 28
- Liquid limit, LL% = 73
- Plasticity Index, PI% = 46
- Specific Gravity, $G_s$ = 2.69
- Organic Content, % = 2.4
- Soluble Sulphate Content %, $SO_4^-$ = 0.087
- pH = 7.7

The soil is composed of:
1. Medium to fine sand = 3.0%
2. Medium to fine silt = 30%
3. Clay = 67%.

4.3 BAGHDAD CLAY

4.3.1 Sampling

Soil samples were taken from a site situated in the south-east of Baghdad (15 kilometres from the city centre).

Samples were taken by a U100 tube from a depth of 2.0 metres below the ground surface. Thirty eight mm diameter tube samples were extracted from the U100 tube then waxed and delivered by air to England for the purpose of this research.
4.3.2 Geology

Iraq is situated on part of the ancient Tethyan geosyncline which has been affected by tectonic movements in pre-Cambrian times. The Tigris and Euphrates rivers cross the country from north to south, then merge into the Arabian Gulf.

The capital, Baghdad, is situated in the Mesopotamian plain in the depositional basin of the twin rivers; the whole area of Baghdad is covered by recent river sediments transported from the north of Iraq by the two rivers.

Generally, the formation of sedimentary soils can be presented by considering sediment formation, sediment transportation and sediment deposition respectively.

a) Sediment Formation

The most important manner of forming sediments is by physical and chemical weathering of rocks on the earth's surface. Generally silt, sand and gravel-sized particles are formed by physical weathering, while clay sized particles are formed by the chemical weathering of rocks. Therefore Baghdad soil has formed as a result of physical and chemical weathering, as shown from the general soil profile of Baghdad.

The formation of clay particles from rock can take place either by build-up of the mineral particles from components in solution, or by the chemical breakdown of other minerals.
b) **Sediment Transportation**

Sediments can be transported by water, air, ice, etc. The two rivers Tigris and Euphrates were the transportation agents of Baghdad sediments. Transportation affects sediments in two major ways (i) it alters particle shape, size and texture by abrasion, grinding, impact and solution, (ii) it sorts the particles. If the agent is water, as in this case, the size of the soil particles is reduced through solution with a considerable sorting and rounding of sand and gravel particles.

c) **Sediment Deposition**

After soil particles have been formed and transported, they are deposited to form a sedimentary soil. The three main causes of deposition in water are velocity reduction, solubility decrease and electrolyte increase.

Baghdad city is situated within the flood plain of the two rivers, so the area is covered by successive flood deposits from the rivers. Velocity reduction after flooding seems to be the main deposition reason in this case.

### 4.3.3 Material Properties

- Natural Water Content, W\% = 27
- Liquid limit, LL\% = 72
- Plasticity Index, PI\% = 49.5
- Specific Gravity, G_s = 2.45
- Organic Content, % = 1.4
- Soluble Sulphate Content %, S04^- = 0.089
- pH = 8.0
The soil is composed of:

1. Fine to medium sand = 5.0%
2. Medium to fine silt = 40%
3. Clay = 45%.
CHAPTER 5
TESTING PROCEDURE AND TECHNIQUE

5.1 TESTING PROGRAMME

An overview of the extensive testing programme carried out for this research is presented in Figure 5.1. In addition to tests to measure residual strength on three different clays, there are tests for classification, chemical composition and soil fabric investigations.

The residual shear strength tests consist of two main phases. The first phase includes an examination and evaluation of different measuring techniques, using both the reversible shear box and the Bromhead ring shear apparatus, in order to find the best measuring techniques. Lias clay was used for these preliminary tests. The second phase concerned the measurement of residual strength of three types of soil using the recommended method of testing obtained from phase one. Tests of this phase were highlighted on the zone of low normal stresses.

Classification and chemical tests were carried out to determine the soils' properties, such as the Atterberg limits and clay fraction, which are usually used to estimate the residual strength of clay.

Soil fabric investigations, which include an X-ray diffraction and scanning electron microscope photographs, were performed to determine the mineralogical features of the soils and to investigate the nature of the failure surface at different normal effective stresses respectively.
Testing Programme

Ring Shear Tests
- Original Method of Testing
  - Lau Clay Only
  - \( r = 0.178 \text{ mm/min} \)
- Modified Grasshead Apparatus
  - Fast Shearing followed by Slow Shearing
    - \( r = 0.178 \text{ mm/min} \)
- Multi-Stage Testing
- Single Stage Testing

Chemical Tests
- (All Types of Soils)
  - Organics Content Test
  - Inorganic Content Test
  - PFG Test

Classification Tests
- (All Types of Soils)
  - Liquid Limit Test
  - Plasticity Limit Test
  - Specific Gravity Test
  - Particle Size Dial Test
  - Sedimentation Test

Fabric Tests
- (All Types of Soils)
  - Void Ratio Diffusion Test
  - Scanning Electron Microscope Technique

Reversal Shear Box Tests
- 100mm x 100mm Shear Box
  - Lau Clay Only
  - \( r = 0.014 \text{ mm/min} \)
  - \( r = 0.024 \text{ mm/min} \)

1. Insert Samples Subject to Fast Shearing for Several Revolutions Followed by Slow Shearing
2. Pre-Cut Samples Subject to Slow Shearing

3. Insert Samples Subject to Fast Shearing for Several Revolutions Followed by Slow Shearing

4. Multi-stone Shear Box
5. 100mm x 100mm Shear Box
6. Multi-Stage Testing
7. Single Stage Testing

Reversal Shear Box
- Recommended Method of Testing
  - (All Types of Soils)
    - 1: Sample Preparation No. 3
    - 2: Multi-Stage Testing
    - 3: 100mm x 100mm Shear Box
    - 4: \( r = 0.016 \text{ mm/min} \)
    - 5: \( r = 0.016 \text{ mm/min} \)
  - Baghdad Clay Shearing: Clay to Clay & Concrete to Clay
  - Both Loading and Unloading Testing were Performed to Define the Complete Failure Envelope.

FIGURE 5.1: TESTING PROGRAMME
5.2  REVERSIBLE SHEAR BOX TESTS

5.2.1 Sample Preparation
As described previously, many investigators found that the initial structure of the clay has no influence on its measured residual strength, as the latter is governed by the final arrangement of clay particles when they orientate themselves parallel to the plane of shearing. Consequently it was decided to use reconstituted samples in the current research.

Soil samples were prepared as follows:
1. Initially, samples were prepared by breaking the air dried material with a pestle and mortar to form a powder passing a 425 µm (No 36) sieve.

2. De-aired distilled water was then added and the clay thoroughly mixed at a water content near its liquid limit. The clay was then allowed to dry out to a moisture content near its plastic limit.

3. The mould of the shear box was placed on the flat glass plate, and clay was tamped inside the mould in two or three layers, keeping the mould firmly in contact with the glass plate. Then the sample ends were trimmed flat using a palette knife.

4. The porous plates were boiled for thirty minutes in de-aired distilled water to remove air bubbles.
5. The two halves of the shear box were assembled and the sample was pushed into the shear box using a wooden dolly having outside dimensions identical to the inner dimensions of the shear box. Drops of oil were used to lubricate the dolly.

Plate 5.1 shows the assembly of the shear box sample.

Samples with a pre-cut plane were prepared initially using two different techniques as follows:

1. The remoulded samples were cut into two identical halves using a wire saw.

2. Samples of two identical thicknesses (10 mm) were prepared separately in a (100 mm) thickness mould. The two halves were pushed one after the other consecutively into the shear box to form a pre-cut sample.

Composite samples of concrete and clay were prepared in a similar way, the lower concrete half was cast in a mould (100m x 100m x 10 mm) dimensions. The soil sample was then kneaded into the shear box over the concrete face to the top of the shear box, then trimmed and smoothed using a palette knife to form a pre-cut composite sample. See Plate 5.2.

5.2.2 Test Procedure and Testing Technique
To begin shear box testing, 100 x 100 mm samples of Lias clay were subjected to various possible techniques to determine the quickest, reliable measurements of the residual strength of the clay. To obtain that the following tests were carried out in order to find the best
PLATE 5.1: ASSEMBLY OF SHEAR BOX
PLATE 5.2: 100 x 100 mm COMPOSITE LIAS CLAY SHEAR BOX SAMPLE OF CLAY AND CONCRETE
method of the formation of the shear plane. Three tests were carried out using a slow rate of shearing \( r_s = 0.024 \text{ mm/min} \) and a faster rate of shearing \( r_f = 0.24 \text{ mm/min} \) as required.

With the slow rate of shearing, a shearing cycle can be completed within 12 hours, therefore if shearing started for example at 9.00 am it would be completed at 9.00 pm and vice versa. Such testing time was very convenient, especially before the use of the electronic data recording system. The fast rate of shearing was selected to be ten times the initial slow rate of shearing. These series of tests were carried out to find the best technique of formation of the shear plane.

5.2.2.1 Intact samples subjected to slow shearing

Initially a sample of Lias clay was consolidated in the shear box at a normal effective stress of 100 kPa. Dead loads were applied in increments of 20 kPa up to the final consolidation pressure. The square root method and logarithmic methods were both used to determine \( t_{90} \) and \( t_{100} \) respectively. Slow shearing was commenced after giving freedom to the two halves of the shear box to move one over the other. When the shearing limit was reached, the proving ring was relaxed and the motor reversed to start shearing in the opposite direction.

The test was stopped and repeated many times at different stages due to the problems associated with excessive extruded soil induced at the initial stage of shearing. Tilting or twisting of the upper half of the shear box due to an excessive settlement caused by poor sample preparation or by fast load application was another problem. To reduce the effect of these problems (see Plate 5.3), samples were prepared later with water contents slightly above or within the plastic limit,
PLATE 5.3: SHEAR BOX SAMPLE SHOWING MARKED TILTING
consolidated, sheared slowly and carefully with continuous monitoring of the dial gauges. The extruded soil was readily dispersed by jetting with de-aired distilled water. Eventually, a soil sample was sheared for ten cycles of continuous forward and backward shearing until the residual value was achieved.

This type of shearing was very slow as it needs more than one week to test one sample under each normal load. In addition the shearing of the sample with forward and backward reversals forms an undulating shear surface which leads to high measured values of residual strength in some cases. Examination of the topography of the failure surface after finishing the test has proved this, see Plate 5.4a.

5.2.2.2 Pre-cut samples subjected to slow shearing

Samples with a pre-cut plane were prepared in two separate half thicknesses (10 mm each). This method was found by experience to be easier than the method of cutting the sample by a wire saw. The soil sample was consolidated to the same normal load and method of load application as above. Shearing then commenced with forward and backward reversals until the residual state was reached, three or four cycles were sufficient for this to be achieved.

The main drawback of this method was that the two parts of the soil sample stuck together, therefore reducing the effect of pre-cutting. However, this test involved less extruded soil and there was little tilting or twisting of the upper half.
a) Shear failure surface at its residual state developed by slow forward and backward shearing

b) Shear failure surface at its residual state developed by precutting technique

PLATE 5.4: 100 X 100 mm SHEAR BOX FAILURE SURFACE OF LIAS CLAY
5.2.2.3 Intact sample subjected to fast shearing for several reversals followed by slow shearing

The third method of forming the failure surface was to subject the consolidated remoulded sample to fast shearing for several reversals at a rate of 0.24 mm/min for a displacement of 150-200 mm. After such displacement, a smooth plane failure surface was produced. The two halves of the sample were then returned to their initial position and left overnight under the same normal load in order to dissipate the developed pore water pressure. Slow shearing was commenced the next day, two cycles being (almost) enough to find the residual strength of the clay.

The topography of the shear surface after the completion of the test showed a flat failure surface as shown in Plate 5.4b.

This method gave a steady shearing strength in the following cycles and was considered to be the best method because it gave the lowest value of residual strength, was less time consuming and it did not require operator skill in preparing pre-cut samples. For this reason this method was used in carrying out the residual strength testing for the three types of soils examined.

The only drawback of this method is the possibility of the excessive extrusion of soil at the beginning of shear, which led to the inevitable loss of fine material from the shear surface and it is the fine fraction which has an important influence on the residual strength of the soil. To reduce that it is recommended that the fast shearing should not be higher than ten times the slow rate as used in this research. The failure surface was planar and approximately horizontal on inspection.
As the aim of the first part of the research was to determine the best method of testing, other tests were carried out (as shown in Figure 5.1) to study other effects, which are:

1. **Shear box size:** To investigate the effect of box size, residual strength tests were carried out using the 60 mm x 60 mm shear box at different normal loads, using the best method of pre-forming the failure surface obtained. The results were compared to those obtained from the 100 mm x 100 mm shear box using the same technique, for the larger box were slightly lower. This suggests that the 60 mm x 60 mm shear box is more prone to frictional problems, as the relative steel/clay contact area is larger with the smaller shear box.

2. **Rate of shearing:** An additional test was carried out at a normal pressure of 100 kPa at a rate of shearing of 0.0096 mm/min. Variations of test results with other previous results were negligible and fell within the accuracy of this type of test. Therefore the rate of shearing was found to have no effect but a rate of 0.024 mm/min is recommended to ensure drained conditions.

3. **Multi-stage testing:** Multi-stage testing involves the establishment of the residual strength condition at one normal stress, followed by the measurement of residual strength at other normal stresses on the same sample. This technique saves time because the displacement which was required to establish the residual strength at each new value of normal stress was not more than 10-15 mm. In addition, there was less soil extrusion, especially after the first normal stress of shearing.
In conclusion and as a result of this part of testing, the recommended technique for reversal shear box tests is to use the 100 mm x 100 mm shear box, with the residual shear plane formed by fast shearing, followed by residual strength measurement at a rate of about 0.024 mm/min. Multi-stage testing can be used if the sample appears to be in good condition after the previous measurement. In practice, samples cannot be used for more than three to four normal loads in loading and unloading sequences.

5.2.3 Interpretation of Test Results
For a set of residual strength tests, Mohr-Coulomb envelopes can be drawn for the residual strength conditions, as shown in Figure 5.2.

The shear strength is generally represented by the equation:

\[ \tau_f = C' + \sigma_n' \tan \phi' \]  

(5.1)
Residual strength can be represented by:

\[ \tau_r = C'_r + \sigma'_n \tan \phi'_r \]  

(5.2)

The value of \( C'_r \) is often very small. Therefore

\[ \phi'_r = \tan^{-1} \left( \frac{\tau_r}{\sigma'_n} \right) \]  

(5.3)

With such a test, the shear stress at failure is often plotted against the normal stress applied, thus directly defining the Mohr-Coulomb failure envelope, assuming it to be a straight line for a particular \( \phi'_r \) at a specific normal effective stress.

Therefore \( \phi'_r \) can be computed simply from equation 5.3 in which \( \tau = \) shear stress = (final proving ring reading - initial proving ring reading) \times ring calibration ring factor \( \div \) area, \( \sigma'_n = \) normal stress = normal force \( \div \) area.

5.2.4 Experimental Errors

The main experimental errors arising in direct shear testing are the variations in the stress and strain conditions across the sample, and a change in the cross-sectional area as the test proceeds. Since this research concerns average values for stress and strain, such variations were not taken into account. Petley [1966] has accounted for the effect of the change in the cross-sectional area and the effect of friction produced on shearing the metal frames and soil. He introduced the following formula:

\[ \frac{\tau}{\sigma} = \tan \phi x N \]  

(5.4)
where N is a factor depending mainly on the relative shearing of clay to clay and metal to clay plus the amount of displacement, values of N were found by him to range from 1.022 to 1.063 for the 60 mm x 60 mm shear box. Such values are expected to be lower in the 100 mm x 100 mm shear box as the relative contact area of clay to steel is less than that for the smaller box.

In the current research, final residual strength readings were obtained with the box at the central position, in order to find the lowest value of $\phi_r$ that can be achieved. Therefore the problem of the area correction did not arise.

The other problem which is usually associated with the tests is the continuous soil extrusion which usually leads to two different problems. Firstly the measurement of shearing resistance along the failure surface will be higher than the actual shearing resistance due to the friction imposed by this accumulated soil built up around the outside of the sample. Extruded soil can readily be dispersed by jetting with de-aired distilled water, therefore the effect of side friction can be reduced. The second associated problem is the tilting and twisting of the upper half of the shear box. This can be partially solved by reducing the initial water content to that slightly above the plastic limit and reducing the increments of the applied load to a smaller amount in order to reduce the amount of soil initially squeezed out.

Friction could be also imposed by some parts of the machine including: 1. Friction imposed between the box which converts the proving to the shear box and the restraining bracket. This was reduced by lubrication.
2. The metal contact between the loading yoke and the bearing point of the top half of the shear box. A ball bearing connection could reduce such friction to an acceptable value, however such connection was not employed for this work.

The proving ring was calibrated for both compression and tension, the calibration chart was used as required.

5.3 BROHEAD RING SHEAR TESTS

5.3.1 Sample Preparation
Samples of Lias clay were tested initially using the original Bromhead apparatus according to the method presented by Bromhead [1979]. Remoulded samples were prepared, as described for the shear box in Section 5.2.1 steps 1-2. The lower porous platen and confining rings were assembled and the remoulded sample kneaded into the annular cavity. The top of the sample was then made level with the top of the confining rings and the upper platen placed in position, located on the centering pin.

After modification, soil samples were prepared as follows:
1. Remoulded samples were prepared as described in Section 5.2.1 steps 1-2.

2. Clay samples were kneaded into the bottom half of the sample container with the fingers. Final trimming using a rough surfaced plastic scraper manufactured for this purpose (see Plates 5.5 and 5.6) to produce a soil sample with 2 mm above the 3 mm height vanes.
a) Kneaded clay at its plastic limit

b) Sample preparation of the bottom half

PLATE 5.5: METHOD OF PREPARING THE BOTTOM HALF OF THE RING SHEAR SAMPLE
a) Use of plastic scraper to form the bottom half of the sample

b) Final form of the bottom half of the sample

PLATE 5.6: FINAL SHAPE OF THE BOTTOM HALF OF THE RING SHEAR SAMPLE BEFORE TESTING
3. The top half was made by fitting purpose-made inner and outer moulds around the vanes on the loading platen. Clay was kneaded into the annular cavity. Final trimming flush with the top surfaces of the mould (2 mm above the vanes) was done with a palette knife (see Plate 5.7). On dismantling the moulds, clay was in the form of a suspended ring held by the vanes.

4. The top platen was put inside the sample container on the top of the bottom surface leaving a gap of about 0.3 mm between the container and the top side of the sample to allow for free movement during rotation. The advantage of this procedure is that a single flat surface in the centre of the sample is more likely to be produced. Also, a specimen comprising two halves placed together essentially behaves as a pre-cut sample reducing the displacement required to reach residual strength.

In the case of composite samples of concrete and clay, the lower platen was replaced by a concrete ring of the same dimensions (100 mm outside diameter, 70 mm inside diameter, 5 mm thick) cast in a special mould. Adhesive material was used to fix the lower concrete ring into the bottom of the sample container.

5.3.2 Test Procedure and Testing Technique
The preliminary tests were carried out on Lias clay using the original Bromhead apparatus. Consolidation and shearing stages were performed according to the recommended method of Bromhead [1979] as follows:

1. The soil sample was consolidated under the first normal stress, and \( t_{90} \) and \( t_{100} \) were determined using the square root method and the logarithmic method respectively.
a) Sample preparation of the top half of the sample

b) Final shape of the top half of the sample

PLATE 5.7: METHOD OF PREPARING THE TOP HALF OF THE RING SHEAR SAMPLE
2. The sample was sheared undrained for about 4-5 revolutions to develop the failure surface.

3. The sample was reconsolidated overnight to get rid of pore water pressure developed during undrained shearing.

4. Slow shearing was recommenced to measure the residual stresses under the fully drained condition.

5. In the case of multi-stage testing, the sample was reconsolidated to the next normal effective stress, providing a new value of $t_b$, and the procedure of slow shearing was followed again.

As presented previously in Section 3.2.1, many trial tests were carried out in the original Bromhead apparatus. However they were all unsuccessful because of the unbalanced forces exerted on the sample as discussed in Section 3.2.2. The testing technique which was used with the modified apparatus was slightly different from that recommended by Bromhead [1979].

The initial stage of shearing was carried out slowly at a rate of 0.0178 mm/min up to a displacement of 50 mm, then the shearing rate was increased to 0.178 mm/min to reduce the time required for testing up to a displacement of 200-250 mm. This displacement was found by experience to be necessary to develop the residual state. The rate of displacement was then slowed again to the initial rate of shearing in order to get true values of residual strength after full dissipation of the pore water pressure, which might have built up during the fast stage. This type of shearing was found to be better
than that recommended by Bromhead [1979], as it reduces the greatest quantity of exuded soil—usually produced during the formation of the shear surface during the initial stage of high speed shearing.

This technique was used in single stage testing and for the first normal stress only using the multi-stage technique, as the displacement required to achieve the residual strength for the next effective stress on the failure surface developed by the first normal stress is very small (10-15 mm) in most cases. However in most of the cases, shearing under each normal stress was left overnight for checking purposes.

Volume change readings were taken from the vertical dial gauge during the shearing stage, however such readings may be false as the volume change has two components: (a) the dilation or contraction of the soil due to shear and consolidation and (b) the loss of material due to 'squeeze'. In practice it is very difficult to separate these two. To overcome this problem, samples were left under each normal stress to shear overnight to ensure drained conditions.

Two series of tests were carried out using the modified ring shear. In the first series, tests were performed on Lias clay using 100 kPa normal stress and two different rates of shearing, 0.178 mm/min and 0.0178 mm/min. No significant differences were observed between the results for the two rates of shearing.

The time of shearing should be as follows:

\[ t = \frac{20h^2}{3C_v} \]  

(5.5)

where \( h \) = sample thickness

\( C_v \) = coefficient of vertical consolidation.
In all the cases, samples were left overnight to ensure full drained conditions.

In the second series, two sets of multi-stage tests were conducted to find the residual failure envelope at a rate of 0.0178 mm/min. It was found that the total time required to establish the complete residual strength envelope is less than half the time required using the single stage testing technique.

In conclusion, the recommended technique of residual strength measurement using the modified Bromhead ring shear apparatus is to use the method of slow-fast-slow shearing only for the first or second normal stress, followed by slow shearing for the other normal stresses using the multi-stage technique. The rate of shearing is recommended to be about 0.0178 mm/min.

By experience, it was found that multi-stage testing should not be used for more than four normal stresses on each fresh sample. However reloading testing on the same sample down to the first normal stress can be performed if the sample is in good condition.

This method was used to test the other types of soils under investigation, namely London clay and Baghdad clay.

The same testing procedure was used to test the composite samples of Baghdad clay and concrete.
5.3.3 Interpretation of Test Results

At the residual state, the torque $T$ transmitted through the sample is given by:

$$T = \frac{2}{3} \pi \tau (R_2^3 - R_1^3)$$  \hspace{1cm} (5.6)

where $R_1$, $R_2$ are respectively inner and outer sample radii.

Since the torque is given by the mean load on the proving rings multiplied by the distance between them

$$T = (F_1 + F_2) L/2$$  \hspace{1cm} (5.7)

where $F_1$, $F_2$ are proving ring forces and $L$ is the distance between them.

Thus

$$\tau = \frac{3 (F_1 + F_2)L}{4 \pi (R_2^3 - R_1^3)}$$  \hspace{1cm} (5.8)

The normal effective stress $\sigma_n'$ is given by

$$\sigma_n' = \frac{P}{\pi (R_2^2 - R_1^2)}$$  \hspace{1cm} (5.9)

where $P$ is the total vertical load. Hence

$$\tau/\sigma_n' = \tan \phi' = \frac{3 (F_1 + F_2)(R_2^2 - R_1^2) L}{4 (R_2^3 - R_1^3) P}$$  \hspace{1cm} (5.10)

For the modified Bromhead ring shear apparatus, $L = 155$ mm, $R_2 = 49.7$ mm, $R_1 = 35.3$ mm.
5.3.4 Evaluation and Experimental Errors

The main difference between this modified apparatus and the original apparatus is the location of failure surface, which is usually formed at the mid-depth of the sample. In the original apparatus the failure surface is produced on the top of the sample, therefore there should be no side friction around the failure surface. However the problem of continuous reduction in specimen thickness limits the duration of the test. In addition, adhesion between the clay and the porous bronze was unreliable and some sort of remoulding usually took place on part of the surface. This was attributed to an increase in water content on the top of the failure as water penetrated through the clearance (which is necessary for correct functioning of the apparatus) between the top platen and the sample container. Such problems were overcome in the modified apparatus by causing the failure surface to be within the body of the sample at approximately its mid-height.

In the Bishop type of apparatus a confining gap between the upper and the lower confining rings can be controlled to minimise the metal to metal friction. Therefore a failure surface could be formed at the middle height of the sample. However the difficulty with this apparatus, as found by Cunningham [1986], arises with the use of the differential screw which is usually used to control the confining gap. The gap proved difficult to alter, which was probably due to soil particles being jammed in between the confining rings. This could lead to accumulation of coarse particles around the failure surface, which would result in the development of a side friction along the failure surface. In addition this type of confining gap gives rise to the fine clay particles being exuded and continuous extrusion leads to a non-horizontal failure surface, which in turn leads to the problem of non-uniform stress distribution across the failure surface and higher residual strength values.
The modified Bromhead apparatus was an improvement as the confining gap has a perpendicular orientation to the shear surface, so, as predicted and as proved by testing later, less soil was extruded. The gap is usually filled with soil during the early stage of consolidation and prevents or minimises other soil particles being extruded during the shearing stage.

Visual inspection of the failure surfaces (Plate 5.8) provides verification as the failure surface is almost flat. However the accumulation of soil particles around the failure surface will induce some friction. Such friction losses cannot be determined in any way. Both the original Bromhead and Bishop types also suffer from this problem.

As shown in Chapter 2, many investigators have found that the ring shear test underestimates the field residual strength of clay. This is due to the type of shearing as it does not simulate three dimensional conditions. So, if this is absolutely true, the orientation of the sample in the modified apparatus with respect to the sample container give hints to the development of a ring shear apparatus which could account for the three dimensional case. To do that more modifications and simulation are required.

Apart from improvements produced in the modified apparatus so far, there are other sources of error which cannot be overcome. These are:

1. The friction imposed by the contact of the torque arm and the main shaft. However, such friction can be reduced significantly by lubrication.
PLATE 5.8: RING SHEAR FAILURE SURFACE
2. Side friction could be produced by the extrusion of soil in the outside gap and in the central void. To reduce that a Teflon coating should be used, the central void can be cleaned from time to time using the water flushing technique developed by Lawrence [1984]. To do so, additional holes must be drilled in the top platen.

3. The difference in stiffness between the two proving rings leads to unbalanced torque application, which results in some frictional losses. However in practice it is very difficult to have two identical proving rings. In addition, the contact between the torque arm and the proving rings gives rise to another possible error in the normal stress. A ball bearing contact can reduce the friction to an acceptable amount.

4. The stress concentration caused by vanes is another source of error. So far we are dealing with average stresses, any improvement in this direction would be welcomed.
5.4 CLASSIFICATIONS AND CHEMICAL TESTS

5.4.1 Index Properties Tests
Liquid limit tests were carried out according to BS 1377: 1975 Section 2.2.2, using the Casagrande apparatus. Plastic limit determinations were carried out according to BS 1377: 1975, Section 2.3.

5.4.2 Specific Gravity Test
Specific gravities of the tested soils were determined according to BS 1377: 1975, Section 2.6.

5.4.3 Particle Size Distribution and Sedimentation Tests
Particle size distribution curves were obtained according to BS 1377: 1975, Section 2.7.1, while clay fraction tests were carried out using the sedimentation method and according to BS 1377: 1975, Section 2.7.3.4.4.

5.4.4 Organic Content, Sulphate Content and pH Tests
Organic content, soluble sulphate content and pH tests were carried out according to BS 1377: 1975, Sections 3.1, 3.3 and 3.4 respectively.
5.5 SOIL FABRIC TESTS

5.5.1 General Aspects of Soil Fabric

The determination of the mechanical properties of soil such as strength, compressibility and permeability is of great importance for engineers in the field of Soil Mechanics. These soil properties are controlled directly by the structure of the soil particles and their arrangement or fabric. Therefore study of the soil structure is necessary to understand its behaviour and properties, and to place soil classification on a more scientific basis.

The particle arrangements in soils remained largely unknown until the development of suitable optical X-ray diffraction, and electron microscope techniques making direct observations possible since about the mid-1950s.

The term 'fabric' is used by some interchangeably with the term 'structure' which refers to the arrangement of particles, particle groups, and pore spaces in a soil.

Generally, the fabric of a soil has two components. The first, termed original fabric, has its origin in the composition of the sediment and its sedimentary history. The second occurs if the soil has been subjected to post-depositional shear strains and is termed shear-induced fabric. Emphasis is generally placed on the "microfabric" which is the level of fabric requiring at least an optical microscope for study, whereas "macrofabric", which refers to those features that can be seen with the unaided eye or a hand lens, is also of great importance.
As shown by Mitchell [1974], particle associations in clay suspensions can be described (as presented by van Olphen [1963]) as follows:

a) **Dispersed.** No face-to-face association of clay particles.

b) **Aggregated.** Face-to-face association of several clay particles.

c) **Flocculated.** Edge-to-edge or edge-to-face association of aggregates.

d) **Deflocculated.** No association between aggregates.

5.5.2 X-Ray Diffraction Tests

5.5.2.1 Background

The fundamental process that occurs when X-rays strike a crystal is one of scattering (or diffraction) and can be regarded conveniently as one of reflection. The reflection is similar to that of light in a mirror, but there is an important difference in that the X-rays penetrate below the surface of the crystal and rays reflected from successive atomic layers may or may not be in phase. The condition for maximum reflected intensity is that the contribution from successive planes should be in phase. If the interplanar spacing is \( d \), this condition is expressed by \( n\lambda = 2d \sin \theta \) (see Figure 5.3).

![Path difference formula](image)

**FIGURE 5.3** Bragg's law
This is the well known Braggs law which is illustrated in Figure 5.3, where:

\( d \) = interplanar spacing

\( \theta \) = Bragg angle

\( \lambda \) = X-ray wavelength

\( n \) = integer

Bragg's law is usually given as \( \lambda = 2d \sin \theta \), dropping the \( n \).

The technical method used herein is called the powder method which is quite common for determining clay mineralogy. X-ray powder diffraction work, with such material, involves a number of difficulties. Clay minerals are less perfectly crystallised and they are extremely fine grained, so that reflections are broader, intensities are lower and there are fewer measurable reflections. The structures of clay minerals are often disordered, so that reflections have very variable profiles and some are so diffuse as to be unobservable.

Difficulties may arise also because many clay minerals have platy morphology, so there is an enhancement of preferred orientation of the specimen particles which may affect the depth of reflection.

A further complication in dealing with clay minerals, is that many specimens have a variable content of inter-layer water molecules and of exchangeable cations, which leads to variation in the powder pattern \( d \) values. Also mixed layer clay minerals occur with varying proportions of two components, e.g. illite-montmorillonite or chlorite-kaolinite. These result in broadened reflections, some of which occur at non-Bragg positions.
5.5.2.2 Sample Preparation

The ideal sample for use with an X-ray diffractometer takes the form of a thin layer of fine-grained (1-10 μm) powder. A slightly thicker layer may be acceptable, since it gives greater intensity, but "transparency" errors will be more serious. The most easily prepared, and therefore most commonly used specimen mounting is the 'smear' on a glass plate. A microscope slide of suitable dimensions is used as a base for spreading a thin layer of powdered sample.

A slurry of powder can be prepared in acetone. This settles and is left behind when the acetone rapidly evaporates. An alternative is the cavity mount, the one used herein, in which a depression is etched in a glass plate and is filled with powder, and the surface smoothed off.

The above methods tend to produce specimens with preferred orientation, particularly if the crystallites have a platy or fibrous habit. More sophisticated methods of overcoming preferred orientation are presented by Zussman [1967].

5.5.2.3 Equipment and Procedure

The X-ray diffractometer is shown in Plate 5.9. The specimen is in the form of a flat layer of powder at the centre of a circle with radius r. On the circumference of the circle lies X, the source of X-rays (defined by a narrow slit perpendicular to the plane of the circle) and also a receiving slit, R, behind which is placed a Geiger or proportional counter. In the zero position, the angle θ is zero and the receiving slit is in line with the direct beam. Both sample and counter can be driven by a motor to rotate about the axis of the circle, and by means of a 2:1 reducing gear the angular velocity of the sample is made accurately half that of the counter. Thus at any
Photograph of an X-ray diffractometer with vertical goniometer circle, in position against the window of a vertical X-ray tube. The beam path is normally entirely shielded, but a circular cover has been removed to show the glass plate (centre of photograph) on which the powder specimen is spread. The cable at top right of the photograph leads from the scintillation counter to the scaler and rate-meter circuits. At bottom right are shown the dial giving the Bragg angle 2θ, the manual control of angular setting and a cog-wheel which is part of the motorised goniometer drive.

PLATE 5.9: X-RAY DIFFRACTOMETER
position, X-ray reflections can occur from those crystallographic planes that lie parallel to the sample surface, as long as they have the correct d value to fulfil the Bragg law.

5.5.2.4 Analysis of the X-Ray Patterns
The complete X-ray diffraction pattern, either film or strip chart record, will consist of a series of reflections of different intensities and values of θ. Each reflection must be accounted for in terms of some component of the sample. The first step in the analysis is to determine the values of d/n for the particular type of radiation (which determines λ) using the equation:

\[ n\lambda = 2d \sin \theta \]  \hspace{1cm} (5.11)

The test pattern may be compared directly with patterns for known materials. The American Society for Testing and Materials maintains a card file of patterns indexed on the basis of the strongest lines in the patterns.

Mineralogical test results are presented in Appendix A.

5.5.3 Scanning Electron Microscope
5.5.3.1 Background
The electron microscope is a valuable tool for the study of soils, because with modern electron microscopes, it is possible to resolve distances to less than 100 Å, thus making study of small clay particles possible.

At the present time, there are two major classes of electron microscope, the transmission instrument and the scanning electron
microscope. In the transmission microscope, the beam passes through the sample or replica, whereas a reflection arrangement is employed in the scanning microscope. It is also possible to employ reflection methods with the transmission microscope.

The scanning electron microscope (SEM) represents a more recent development. With this instrument, there is a reflection geometry between the beam incident on the sample and the image-forming beam, so the electrons do not have to pass through the material, and this simplifies sample preparation.

Because the image is produced using electrons as the information carrying medium, the depth of focus of the image is greater than that for the light microscope. Hence an impression is gained of the three-dimensional nature of the surfaces being investigated. Most of this information conveyed by the scanning electron microscope image is topographical.

5.5.3.2 Sample Preparation
Dried materials are needed for replication and for direct viewing on the electron microscope because the specimen has to be placed in the high vacuum system, where water molecules would evaporate. They would then collide with the electrons and cause a loss of resolution. Air drying may be inadequate because many soils shrink markedly, and fabric relations may be changed; it is also possible that the morphology of thin minerals may be affected.

Gillot [1976] has described successfully three methods of sample preparation which are:
1. **Air Drying.** Air drying shrinkage will effect the greatest changes in the fabric when the sample is weak and the environment in which it is found is not subject to wetting and drying cycles. Air-dried sensitive soils show a relatively dense fabric when compared with the more open fabric of the same area when the sample is frozen on a cold stage.

2. **Freeze Drying.** Freeze drying reduces shrinkage and distortion of the sample because ice is removed by sublimation. The surface energy of the solid-vapour interface is commonly higher than that of the liquid-vapour interface, but no meniscus forms in pores and capillary spaces. Since ice is much less mobile than liquid water, fluid migration into finer pores is largely prevented as moisture is removed. The method, however, has certain drawbacks.

When the sample is frozen, the conversion of liquid water to ice in larger void spaces will be accompanied by an increase in specific volume of about 9%. So sample dilation at the outset may induce fabric change.

3. **Critical Point Method.** Critical point drying involves raising the temperature and pressure of the liquid phase to values above those of the critical point, at which the physical properties of a liquid and its vapour become the same. At this point, the interface vanishes so surface tension forces cease to exist.

However, the evolution of gas from pore fluids, osmotic effects, and pressure build-up, due to inadequate permeability of the sample during fluid expansion, are all possible causes of fabric damage.
Due to the unavailability of the apparatus required to prepare samples at these critical points, only air-drying and freeze-drying techniques were used in this research.

SEM photographs were taken of both the failure surface and unsheared surfaces for soil samples after being sheared at different normal stresses. These photographs are presented in Appendix B of this thesis. Observations and discussions will be presented in Chapter 7.
CHAPTER 6
TEST RESULTS

6.1 INTRODUCTION

This chapter presents the test results obtained from the shear box and ring shear apparatus. X-ray diffraction test results and scanning electron microscope photographs are presented in Appendix A and Appendix B respectively. It should be noted that the results of preliminary test programmes, which accounted for approximately 25% of the total number of tests are not presented as they were not valuable.

A series of tests was conducted on Lias clay, using various possible testing techniques to determine the method which gave the quickest reliable measurement of residual strength of clay. The favoured methods of testing were employed to test the London clay and Baghdad clay.
6.2 LIAS CLAY

6.2.1 Reversible Shear Box Test Results
Lias clay was tested using 100 x 100 mm and 60 x 60 mm shear box samples. Initially two rates of shear, 0.24 mm/min and 0.024 mm/min, were applied to ten cycles of forward and backward shearing in order to develop residual shearing conditions within the 100 x 100 mm sample. No appreciable difference in the results was obtained, but the amount of soil extracted was significantly higher using the faster rate of shearing. This caused sample tilting and an undesirable level of deflection. Therefore a rate of shearing of 0.024 mm/min was adopted.

6.2.1.1 Series Li-1
This series comprises tests of the type mentioned above. Six fresh samples were sheared at a normal stress of 30, 50, 70, 150, 220 and 280 kPa respectively using the slow forward and backward shearing cycles at a rate of 0.024 mm/min.

The test results are presented in Figures 6.1 to 6.20. Figures 6.1 to 6.12 show the shear stress-displacement relationships. Figure 6.13 shows the residual failure envelope, and Figure 6.14 shows the variation of \( \tan^{-1} \phi_r' \) with normal stress. Figures 6.15 and 6.16 give some examples of volume change displacement relationships. Figures 6.17 to 6.20 show some examples of the consolidation test results, using both the square root method and the logarithmic method respectively.
6.2.1.2 Series Li-2
This series includes a few tests on 100 x 100 mm shear box samples subjected to fast forward and backward shearing up to about 200 mm displacement, to form the failure surface. Slow shearing was applied the following day at a rate of 0.024 mm/min. Two cycles each of backward and forward shearing were easily sufficient to develop the residual shear strength.

Typical test results are presented in Figures 6.21 to 6.23. Figures 6.21 and 6.22 show the shear stress-displacement relationships for these tests, and Figure 6.23 shows the volume change displacement relationship.

6.2.1.3 Series Li-3
Series Li-3 comprises multi-stage tests which were carried out using 60 x 60 mm pre-cut shear box samples. These proved to be less time consuming than the single-stage test. Series Li-3 consisted of slow shearing tests conducted on pre-cut samples produced by 10 cycles of fast forward and backward shearing prior to the commencement of slow shearing.

Two fresh samples were tested under normal stress sequences of 27-54-108 kPa and 162-216-324 kPa respectively.

The test results are presented in Figures 6.24 to 6.28. Figures 6.24 and 6.25 show the shear stress-displacement relationships of the final forward and backward slow shearing cycles. Figure 6.26 shows the residual failure envelopes and Figure 6.27 shows the variation of \( \tan^{-1} \), with normal stress. Figure 6.28 gives examples of consolidation test results using the square root method.
6.2.2  Modified Bromhead Ring Shear Test Results

As discussed in Chapter 5, no reliable test results were obtained using the original Bromhead apparatus. Therefore only those obtained from the modified apparatus are presented here.

6.2.2.1  Series Li-4

Series Li-4 comprises 11 tests carried out on samples consolidated individually at normal stresses of 26, 52, 78, 104, 130, 156, 208, 260, 312, 416 and 520 kPa respectively and then sheared at a rate of 0.178 mm/min.

Test results are presented in Figures 6.29 to 6.34. Figures 6.29 to 6.32 show the shear stress-displacement relationships, Figure 6.33 the residual failure envelope, and Figure 6.34 the variation of $\tan^{-1} \phi_r'$ with normal stress.

6.2.2.2  Series Li-5

Experience with the modified apparatus proved that multi-stage testing was less time consuming. Series Li-5 comprises tests of this type. Samples were sheared initially at a slow rate of 0.0178 mm/min for a displacement of about 50 mm. The shearing rate was then increased to 0.178 mm/min until a displacement of 250 mm had been achieved, and then slowed again to the original slow rate of shearing until the residual state had been reached. This technique was found to be necessary only for the first or second normal stresses. Thereafter a slow shearing of about ten millimetres was sufficient to develop the residual strength at the subsequent normal stresses.
Two fresh samples were employed which were loaded and sheared under normal stress sequences of 26-52-156-260-364 kPa and 78-104-208-312-416 kPa respectively.

The test results are presented in Figures 6.35 to 6.38. Figures 6.35 and 6.36 show the shear stress-displacement relationships. Figure 6.37 shows the residual failure envelope, and Figure 6.38 shows the variation of $\tan^{-1} \phi_r$ with normal stress.
6.3 **LONDON CLAY**

6.3.1 **Reversible Shear Box Test Results**

100 x 100 mm shear box tests were carried out using the best technique determined from testing the Lias clay. The London clay was consequently tested using slow forward and backward shearing on pre-formed failure surface samples developed by the fast forward and backward shearing technique usually employed before for Lias clay.

6.3.1.1 **Series Lo-1**

Series Lo-1 comprises tests of the type mentioned above using the multi-stage technique and a slow rate of shearing of a 0.016 mm/min. Two fresh samples were tested using the normal stress sequences of 40-60 kPa and 140-240-340 kPa respectively.

Test results are presented in Figures 6.39 to 6.42. Figures 6.39 and 6.40 show the shear stress-displacement relationships of the final forward and backward shearing cycles. Figure 6.41 shows the residual failure envelope, and Figure 6.42 shows the variation of \( \tan^{-1} \phi' \) with normal stress.
6.3.2 **Modified Bromhead Ring Shear Test Results**

London clay samples were tested using the modified apparatus. Multi-stage loading tests were carried out at a variable rate of shearing using an initial slow rate of 0.0178 mm/min, an intermediate fast rate of 0.178 mm/min and a final slow rate of 0.0178 mm/min again. This technique was used to reduce the time required for testing. It was found that following the first or second stages, slow shearing of approximately ten millimetres was sufficient to develop the residual strength of clay.

6.3.2.1 **Series Lo-2**

This series includes tests conducted as described above. Four fresh samples were tested using normal stress sequences of 26-130-234-338 kPa, 52-156-260-364 kPa, 78-182-286-390 kPa and 104-208-312-416 kPa respectively.

Test results are presented in Figures 6.43 to 6.49. Figures 6.43 to 6.46 show the shear stress-displacement relationships. Figure 6.47 shows the residual failure envelope, Figure 6.48 the variation of $\tan^{-1} \phi_r'$ with normal stress and Figure 6.49 gives examples of the consolidation test results using the square root method.

6.3.2.2 **Series Lo-3**

Series Lo-3 is similar to series Lo-2 but for unloading stress sequences. Three different fresh samples were consolidated in increments of 26 kPa up to the required normal stress then sheared as follows: 338-234-130-26 kPa, 364-260-156-52 kPa and 416-312-208-104 kPa respectively.
Test results are presented in Figures 6.50 to 6.54. Figures 6.50 to 6.52 show the shear stress-displacement relationships. Figure 6.53 shows the residual failure envelope and Figure 6.54 shows the variation of $\tan^{-1} \frac{\phi}{\gamma}$ with normal stress.
6.4 BAGHDAD CLAY

6.4.1 Reversible Shear Box Test Results
Baghdad clay was tested using 100 x 100 mm shear box samples in the same way as described in Section 6.3.1 for London clay.

6.4.1.1 Series B-1
Series B-1 includes tests of the type mentioned above using the multi-stage technique for two fresh samples and single-stage technique for the others. A slow rate of shearing of 0.016 mm/min was used. Four samples were tested as follows: 20 kPa, 40-140-240-340 kPa, 60 kPa and 80 kPa, 100-200-300-400 kPa respectively.

Test results are presented in Figures 6.55 to 6.60. Figures 6.55 to 6.58 show the shear stress-displacement relationships of the final forward and backward shearing cycles. Figure 6.59 shows the residual failure envelope and Figure 6.60 shows the variation of tan⁻¹ \( \phi_r' \) with normal stress.

6.4.1.2 Series B-2
Series B-2 comprised tests conducted on composite samples of clay and concrete in the same way as described in Section 6.4.1.1. The sample preparation technique was described before in Chapter 5. Three samples were tested using normal stress sequences of 20 kPa, 40-140-240-340 kPa and 100-200-300-400 kPa respectively.

Test results are presented in Figures 6.61 to 6.65. Figures 6.61 to 6.63 show the shear stress-displacement relationships of the final forward and backward shearing cycles. Figure 6.64 shows the residual failure envelope and Figure 6.65 shows the variation of tan⁻¹ \( \phi_r' \) with the normal stress.

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6.4.2 Modified Bromhead Ring Shear Test Results

Baghdad clay was tested using the modified ring shear in the same way as described in Section 6.3.2 for London clay.

6.4.2.1 Series B-3

This series comprises tests on four fresh ring samples tested using normal stress sequences of 26-130-234 kPa, 52-156-260-364 kPa, 78-182-286 kPa and 104-208-312-416 kPa respectively.

Test results are presented in Figures 6.66 to 6.71. Figures 6.66 to 6.69 show the shear stress-displacement relationships. Figure 6.70 shows the residual failure envelope and Figure 6.71 shows the variation of \( \tan^{-1} \phi' \) with normal stress.

6.4.2.2 Series B-4

This series includes tests similar to those described in series B-3 carried out on composite samples of clay and concrete. The sample preparation method was as described before in Chapter 5. Three fresh samples were tested using stress sequences of 26 kPa, 52-156-260-364 kPa and 104-208-312-416 kPa respectively.

The test results are presented in Figures 6.72 to 6.75. Figures 6.72 and 6.73 show the shear stress-displacement relationships. Figure 6.74 shows the residual failure envelope and Figure 6.75 shows the variation of \( \tan^{-1} \phi' \) with normal stress.

6.4.2.3 Series B-5

Series B-5 is similar to series B-4 but with unloading stress sequences. Two fresh samples were consolidated in increments of 26 kPa up to normal stresses of 364 and 416 respectively, then tested in the following sequences 364-260-156-52 kPa and 416-312-208-104 kPa respectively.
Test results are presented in Figures 6.76 to 6.79. Figures 6.76 and 6.77 show the shear stress-displacement relationships. Figure 6.78 shows the residual failure envelope and Figure 6.79 shows the variation of $\tan^{-1} \phi'$ with normal stress.
6.5 LABORATORY SHEAR TEST RESULTS

The results of the residual shear strength tests are classified as series as shown before and are presented in Figures 6.1 to 6.79 as follows:
SERIES Li-1
Figure 6.1 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Slow Forward & Backward Shearing Type
Rate of Shearing: 0.024 mm/min, $\sigma' = 30$ kPa

Figure 6.2 Shear Stress-Displacement Relationship
Lias Clay, 100mm x100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: 0.024 mm/min, $\sigma^'n$, 50 kPa

Figure 6.3 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: 0.024 mm/min, σ' n, 50 kPa

Figure 6.4 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: 0.024 mm/min, σ' n 70 kPa

Figure 6.5 Shear Stress-Displacement Relationship
Figure 6.6 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: - 0.024 mm/min, c’/n, 150 kPa

Figure 6.7 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100m Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing = 0.024 mm/min, σ'n, 150 kPa

Figure 6.8 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing = 0.024 mm/min, \( \sigma'_n \), 220 kPa

Figure 6.9 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: 0.024 mm/min, σ'n, 220 kPa

Figure 6.10 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Rate of Shearing: 0.024 mm/min, σ′n, 280 kPa
Forward & Backward Slow Shearing Type

Figure 6.11 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: 0.024 mm/min, \( \sigma' \text{m}, 280 \text{ kPa} \)

Figure 6.12 Shear Stress-Displacement Relationship
Lias Clay, 100mm x 100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing: - 0.024 mm/min, Pcr=150 kPa

Figure 6.13 Residual Failure Envelope
Lias Clay, 100mm x100mm Shear Box Tests
Forward & Backward Slow Shearing Type
Rate of Shearing :- 0.024 mm/min, P<sub>cr</sub>=150 kPa

Figure 6.14 Variation of \( \tan^{-1} \phi' \) vs Normal Stress.
Lias Clay, Shear Box Test
Size of Box=100mm x100mm
\( \sigma'_n, 150 \text{ kPa} \)

**Figure 6.15 Volume Change-Displacement Relationship**
Lias Clay, Shear Box Test
Size of Box=100mm x 100mm
σ'n, 150 kPa

Figure 6.16 Volume Change-Displacement Relationship
Lias Clay, Shear Box Test
Size of Box = 100mm x 100mm
σ'n, 50 kPa

Figure 6.17 Consolidation Stage, Sq. Root Method
Lias Clay, Shear Box Test
Size of Box = 100mm x 100mm
σ'n, 50 kPa

Figure 6.18 Consolidation Stage, Log Method
Lias Clay, Shear Box Test
Size of Box: 100mm x 100mm
N = 150 kPa

Figure 6.19  Consolidation Stage, Sq. Root Method
Lias Clay, Shear Box Test
Size of Box = 100mm x 100mm
N = 150 kPa

$t_{100} = 32$ (mins.)

Figure 6.20 Consolidation Stage, Log Method
SERIES Li-2
Figure 6.21 Shear Stress-Displacement Relationship

Lias Clay, 100mm x 100mm Shear Box Tests
Slow Shearing on Pre-Cut Sample
Rate of Shearing: 0.024 mm/min, \( \gamma \), 100 kPa

- 1st T.
- 2nd C.
- 3rd T.
- 4th C.
Lias Clay, 100mm x 100mm Shear Box Tests
Slow Shearing on Pre-Cut Sample
Rate of Shearing: 0.024 mm/min, σ' n 150 kPa

Figure 6.22  Shear Stress-Displacement Relationship
Lias Clay, Shear Box Test
Size of Box=100mm x 100mm, Pre-Cut Sample
\( \sigma' n \), 150 kPa

Figure 6.23 Volume Change-Displacement Relationship
SERIES Li-3
Lias Clay, 60mm x 60mm Shear Box Tests
Rate of Shearing = 0.024 mm/min

Figure 6.24 Shear Stress-Displacement Relationship
Lias Clay 60mm x 60mm Shear Box Tests
Rate of Shearing: ± 0.024 mm/min

Figure 6.25 Shear Stress-Displacement Relationship
Lias Clay, 60mm x 60mm Shear Box Tests
Rate of Shearing: 0.024 mm/min
Pcr = 200 kPa

Figure 6.26 Residual Failure Envelope
Figure 6.27 Variation of $\tan^{-1} \phi'_r$ vs Normal Stress

Lias Clay, 60mm x 60mm Shear Box Tests
Rate of Shearing :- 0.024 mm/min
Pcr = 200 kPa
Lias Clay, Shear Box Tests
Size of Box: 60mm x 60mm

+ $\sigma'_{n_1} = 216$ kPa

$\times$ $\sigma'_{n_2} = 324$ kPa

$t_{90} = 20 \text{ (mins.)}$

$t_{90} = 36 \text{ (mins.)}$

**Figure 6.28 Consolidation Stage, Sq. Root Method**
Figure 6.29 Shear Stress-Displacement Relationship
Figure 6.30 Shear Stress-Displacement Relationship

Lias Clay, Ring Shear Tests
Single Normal Load Tests
Rate of Shearing = 0.178 mm/min
Lias Clay, Single Normal Load Tests
Ring Shear Tests, Rate of Shearing: 0.178 mm/min

Displacement, mm

Shear Stress, kPa

Figure 6.31 Shear Stress-Displacement Relationship
Lias Clay, Ring Shear Tests
Single Normal Load Tests
Rate of Shearing = 0.178 mm/min

Figure 6.32 Shear Stress-Displacement Relationship
Lias Clay, Ring Shear Tests
Single Normal Load Tests, Pcr = 200kPa
Rate of Shearing : 0.178 mm/min

Figure 6.33 Residual Failure Envelope
Figure 6.34 Variation of $\tan^{-1}\phi'$ vs Normal Stress
SERIES Li-5
Figure 6.35 Shear Stress Displacement Relationship

Lias Clay, Ring Shear Tests, Rate of Shearing
Fast = .178 mm/min, Slow = .0178 mm/min
Loading Sequences

- $\sigma'' = 26$ kPa
- $\sigma'' = 52$ kPa
- $\sigma'' = 156$ kPa
- $\sigma'' = 260$ kPa
- $\sigma'' = 364$ kPa
Lias Clay, Ring Shear Tests
Loading Sequences, Rate of Shearing:
Fast = 0.178 mm/min, Slow = 0.0178 mm/min

Figure 6.36 Shear Stress-Displacement Relationship
Lia Clay, Ring Shear Tests
Loading Sequences $P_{cr} = 200$ kPa
Rate of Shearing $\approx 0.0178$ mm/min

Figure 6.37 Residual Failure Envelope
Figure 6.38 Variation of $\tan^{-1}\phi'$ vs Normal Stress
SERIES Lo-1
London Clay, 100mm x 100mm Shear Box Tests
Slow Shearing: 0.016 mm/min
Pre-Cut Samples

Figure 6.39 Shear Stress-Displacement Relationship
London Clay, 100mm x 100mm Shear Box Tests
Slow Shearing: -0.016 mm/min
Pre-Cut Samples

Figure 6.40 Shear Stress-Displacement Relationship
London Clay, 100mm x 100mm Shear Box Tests
Slow Shearing: -.016 mm/min  Pcr=150 kPa
Pre-cut Samples, Loading Sequences

Figure 6.41 Residual Failure Envelope
Figure 6.42 Variation of $\tan^{-1} \phi'$ vs Normal Stress
SERIES Loi-2
London Clay, Ring Shear Tests
Loading Sequences, Rate of Shearing:
Fast = 0.178 mm/min, Slow = 0.0178 mm/min

Figure 6.43 Shear Stress-Displacement Relationship
London Clay, Ring Shear Tests
Loading Sequences, Rate of Shearing:
Fast = 0.178 mm/min, Slow = 0.0178 mm/min

Figure 6.44 Shear Stress-Displacement Relationship
London Clay, Ring Shear Tests
Loading Sequences, Rate of Shearing:
Fast = 0.178 mm/min, Slow = 0.0178 mm/min

Figure 6.45 Shear Stress-Displacement Relationship
London Clay, Ring Shear Tests
Loading Sequences, Rate of Shearing:
Fast = 0.178 mm/min, Slow = 0.0178 mm/min

Figure 6.46 Shear Stress-Displacement Relationship
London Clay, Ring Shear Tests
Rate of Shearing: 0.0178 mm/min
Loading Sequences, Pcr = 200 kPa

Figure 6.47 Residual Failure Envelope
London Clay, Ring Shear Tests
Rate of Shearing: 0.0178 mm/min
Loading Sequences, Pcr = 200 kPa

Figure 6.48 Variation of $\tan^{-1} \phi_r$ vs Normal Stress
London Clay, Ring Shear Tests
Multi-Stages Testing:
- $\sigma'_n, 26$ kPa
- $\sigma'_n, 26-130$ kPa
- $\sigma'_n, 130-234$ kPa
- $\sigma'_n, 234-338$ kPa

Figure 6.49 Consolidation Stage, Sq. Root Method

$t_{90^s} < 10$ (mins.)
London Clay, Ring Shear Tests
Unloading Sequences
Rate of Shearing: -0.0178 mm/min

+σ' n, 338-234 kPa
● σ' n, 234-130 kPa
× σ' n, 30-26 kPa

Figure 6.50 Shear Stress-Displacement Relationship
London Clay, Ring Shear Tests
Unloading Sequences
Rate of Shearing: -0.0178 mm/min

Figure 6.51 Shear Stress-Displacement Relationship
Figure 6.52 Shear Stress-Displacement Relationship

London Clay, Ring Shear Tests
Unloading Sequences
Rate of Shearing: -0.0178 mm/min

Shear Stress, kPa

Displacement, mm

+ σ'\text{n}.416-312 kPa
● σ'\text{n}.312-208 kPa
× σ'\text{n}.208-104kPa
London Clay, Ring Shear Tests
Rate of Shearing = 0.0178 mm/min
Unloading Sequences $P_{cr} = 150$ kPa

Figure 6.53 Residual Failure Envelope
London Clay, Ring Shear Tests
Unloading Sequences Pcr = 100kPa
Rate of Shearing :: .0178 mm/min

Figure 6.54 Variation of Tan⁻¹φᵣ vs Normal Stresses
Figure 6.55 Shear Stress-Displacement Relationship
Baghdad Clay, 100mm x 100mm Shear Box Tests
Clay to Clay Shearing, Loading Sequences
Rate of Shearing = 0.0160 mm/min

Figure 6.56 Shear Stress-Displacement Relationship
Baghdad Clay
100mm x 100mm Shear Box Tests
Clay to Clay Shearing
Loading Sequences
Rate of Shearing = 0.0160 mm/min

Figure 6.57 Shear Stress-Displacement Relationship
Figure 6.58 Shear stress-Displacement Relationship
Baghdad Clay, 100mm x 100mm Shear Box Tests
Clay to Clay Shearing, Loading Sequences
Rate of Shearing: .016mm/min, Pcr= 150 kPa

Figure 6.59 Residual Failure Envelope
Baghdad Clay, 100mm x 100mm Shear Box Tests
Clay to Clay Shearing, Loading Sequences
Rate of Shearing = .016 mm/min, Pcr=150 kPa

Figure 6.60 Variation of $\tan^{-1} \phi$ vs Normal Stress.
SERIES B-2
Baghdad Clay, 100mm x100mm Shear Box Test
Clay to Concrete Shearing, Single Normal Load
Rate of Shearing = 0.0160 mm/min

Figure 6.61 Shear Stress-Displacement Relationship
Baghdad Clay, Clay to Concrete Shearing
100mm x 100mm Shear Box Tests
Loading Sequences
Rate of Shearing = 0.016 mm/min

Figure 6.62 Shear Stress-Displacement Relationship
Baghdad Clay, Clay to Concrete Shearing
100mm x 100mm Shear Box Tests, Loading Sequences
Rate of Shearing = .016mm/min

Figure 6.63 Shear Stress-Displacement Relationship
Baghdad Clay, Clay to Concrete Shearing
100mm x 100mm Shear Box Tests, Loading Sequences
Rate of Shearing = 0.016 mm/min, Pcr = 100 kPa

Figure 6.64 Residual Failure Envelope
Baghdad Clay, Clay to Concrete Shearing
100mm x 100mm Shear Box Tests, Loading Sequences
Rate of Shearing = 0.016 mm/min, Pcr = 100 kPa

Figure 6.65 Variation of $\tan^{-1} \phi'$ vs Normal Stress
SERIES B-3
Baghdad Clay, Ring Shear Tests
Clay to Clay Shearing, Loading Sequences
Rate of Shearing: Slow = 0.0178 mm/min, Fast = 0.178 mm/min

- $\sigma'$, 26 kPa
- $\sigma'$, 26-130 kPa
- $\sigma'$, 130-234 kPa

Figure 6.66 Shear Stress-Displacement Relationship
Figure 6.67 Shear Stress-Displacement Relationship
Baghdad Clay, Clay to Clay Shearing
Ring Shear Tests, Loading Sequences
Rate of Shearing =
Slow = 0.0178 mm/min, Fast = 0.178 mm/min

Figure 6.68 Shear Stress - Displacement Relationship
Baghdad Clay, Clay to Clay Shearing, Ring Shear Tests
Loading Sequences, Rate of Shearing =
Slow = .0178 mm/min, Fast = .178 mm/min

Figure 6.69 Shear Stress-Displacement Relationship
Figure 6.70 Residual Failure Envelope

Baghdad Clay, Ring Shear Tests, Clay to Clay Shearing
Rate of Shearing = 0.0178 mm/min
Loading Sequences, P_{cr} = 200 kPa
Baghdad Clay, Clay to Clay Shearing, Ring Shear Tests
Rate of Shearing = 0.0178 mm/min.
Loading Sequences, Pcr=200 kPa

Figure 6.71 Variation of $\tan^{-1}\phi'$ vs Normal Stress
SERIES B-4
Baghdad Clay, Clay to Concrete Shearing
Ring Shear Tests, Loading Sequences
Rate of Shearing =
Slow = .0178 mm/min, Fast = .178 mm/min

Shear Stress-Displacement Relationship

Figure 6.72 Shear Stress-Displacement Relationship
Baghdad Clay, Ring Shear Tests
Clay to Concrete Shearing, Loading Sequences
Rate of Shearing:
Slow = 0.0178 mm/min, Fast = 0.178 mm/min

Figure 6.73 Shear Stress-Displacement Relationship
Baghdad Clay, Clay to Concrete Shearing
Ring Shear Tests, Loading Sequences
Rate of Shearing = 0.0178 mm/min, Pcr=150 kPa

Figure 6.74 Residual Failure Envelope
Baghdad Clay, Clay to Concrete Shearing Ring Shear Tests
Rate of Shearing = 0.0178 mm/min
Loading Sequences, Pcr = 150 kPa

Figure 6.75 Variation of $\tan^{-1} \phi'$ vs Normal Stress
Baghdad Clay, Ring Shear Tests  
Clay to Concrete Shearing  
Unloading Sequences  
Rate of Shearing = 0.0178 mm/min  

Figure 6.76 Shear Stress-Displacement Relationship
Baghdad Clay, Ring Shear Tests
Clay to Concrete Shearing
Unloading Sequences
Rate of Shearing = 0.0178 mm/min

Figure 6.77 Shear Stress-Displacement Relationship
Baghdad Clay, Ring Shear Tests
Clay to Concrete Shearing, Unloading Sequences
Rate of Shearing: 0.0178 mm/min, Pcr=100 kPa

Figure 6.78 Residual Failure Envelope
Baghdad Clay, Ring Shear Tests
Clay to Concrete Shearing, Unloading Sequences
Rate of Shearing: -0.0178 mm/min, Pcr=150 kPa

Figure 6.79 Variation of Tan$^{-1}\phi_r$ vs Normal Stress
CHAPTER 7

DISCUSSION AND CONCLUSIONS

7.1 INTRODUCTION

The subject of residual strength of clay, as mentioned in different parts of this thesis, has been investigated extensively following the starting point of Skempton (1964). The main objectives of this research were to explore the best laboratory technique for measuring residual strength without using very complicated methods and then to shed some light on the problem of the residual strength at low normal stresses, in order to determine its significance from a practical point of view.

The apparatus modifications developed as part of this work were not very complicated, but were sufficient to overcome laboratory difficulties, particularly with the existing ring shear apparatus keeping it a simple ring shear. The soil fabric test results gave an indication and some assistance in understanding the problem under investigation.

This chapter comprises extensive discussion of the work in general and the main conclusions.
7.2 LABORATORY MEASUREMENTS OF RESIDUAL STRENGTH OF CLAY

7.2.1 Lias Clay
Lias clay, as mentioned before, was tested using the reversible shear box with 100 x 100 mm and 60 x 60 mm sample sizes and the modified Bromhead ring shear apparatus.

7.2.1.1 Reversible shear box tests
The shear box was used by many investigators before the development of the new ring shear apparatus for measuring the residual strength of clay, and it is still being used to measure the residual strength of undisturbed samples containing a natural slip surface.

As shown in Plate 7.1, Lias clay is a highly overconsolidated laminated clay, so it was very difficult to get good quality undisturbed shear box samples, therefore only reconstituted shear box samples were tested.

The effect of a large displacement was obtained by returning the split box to its starting position after completing the extent of its travel and shearing again. This process was repeated a number of times until a steady (residual) value of shear strength was observed.

The amount of cumulative displacement applied to shear the reconstituted 100 x 100 mm shear box samples was in excess of 200 mm. The mechanism of shearing in two opposite directions led to the development of additional resistance due to the rearrangement of the clay particles. This is very obvious in Figure 6.5 which shows a marked peak in the ninth tension cycle. In addition, and as a result of the large displacement, other experimental difficulties arose such
PLATE 7.1: LIAS CLAY OBTAINED FROM THE BLOCK SAMPLES
as the excessive amount of extruded soil and the undesirable level of deflection of the loading cap (see Plate 5.3). This led to higher side friction as imposed by the extruded soil around the plane of separation in addition to the horizontal friction component force resolved from the inclination of the applied load, such friction forces are usually difficult to measure or estimate. Failure surfaces as found on visual inspection were almost always undulant, leading to a higher non-uniform stress distribution across the failure surface and a possible higher strength than the residual strength.

No practical approach was able to overcome these problems absolutely however, it was reduced later to a certain limit by preparing the samples at an initial water content close to the clay's plastic limit in order to minimise the early quantities of the extruded soil. The extruded soil was removed from time to time by gentle water jetting around the separation plane of the shear box.

Due to the difficulties associated with the test, it was decided to use pre-cut samples as the latter need lower displacement. Three different pre-cutting techniques were employed to develop a pre-cut plane within the sample:

1. cutting by a wire;
2. preparing the sample in two identical halves; and
3. fast forward and backward shearing.

The third was found to be the best technique as it needed minimum skill of sample preparation; also preparing pre-cut samples outside the shear box leads to the problem of arranging the pre-cut plane within the plane of separation of the shear box which is not easy in practice. However, in any case great care is required to consolidate the specimen under the required normal stress.
Failure surfaces produced by fast shearing were almost flat and the amount of extruded soil produced in the following slow shearing was appreciably lower, as the greatest quantity of extruded soil was produced in the early stages of fast shearing which can be removed prior to the commencement of slow shearing. Typical results using the latter technique are presented in Figure 6.22 which shows the shear stress-displacement of 100 x 100 mm pre-cut samples sheared under a normal stress of 150 kPa at a rate of 0.024 m/min. Residual strength values were almost the same with a maximum difference of 2-3% between the tension and compression cycles. However, the third (tension) cycle shows a small peak due to the increase in particle to particle resistance built up as a result of the change in the shearing direction. However, the final residual strength values are almost the same.

Test observations revealed that on approaching the residual condition, the vertical movement dial gauge indicated very little or no further change in volume. This is demonstrated in Figure 6.23 which shows the volume change-displacement relationships. A constant volume change was achieved in the third tension cycle after a few millimetres of shearing, however the next compression cycle shows a marked increase which is due to the additional compression experienced by the sample at the outer edges due to some loss of fine material as a slurry, such increase was taken into account and considered as an out of control experimental error.

Shear box sample preparation needs a lot of skill and time, so to reduce these two factors, it was decided to carry out the rest of the tests using a multi-stage technique so that shearing could occur on a pre-determined failure surface produced under the previous normal
stress. Such technique was used with the 60 x 60 mm shear box samples. Typical examples are presented in Figures 6.24 and 6.25 which show some developed peaks for different normal stresses.

Examination of the failure surface on completion of the tests showed that the pre-determined failure surface was pushed downward and covered by some compacted extruded soil to recreate a new failure surface, therefore shearing across such a failure surface required an additional force to overcome the new developed strength. The latter could account for the brittle behaviour of the shear stress-displacement diagram, although multi-stage testing on pre-determined failure surfaces proved to be a practical technique. This is attributed to the total amount of displacement required to develop the residual condition which is in the region of 10-15 mm, while with the 100 x 100 mm reconstituted samples, a displacement in excess of 200 mm was required. Accordingly the amount of extruded soil was much lower in the former case and the samples’ condition after testing were quite reasonable.

Examples of the results of the consolidation stage are shown in Figures 6.17 to 6.20 using both the logarithmic method and the square root method. t_{100} values obtained are not higher than 100 minutes in most of the tests. However, it should be mentioned that all the samples were left to consolidate overnight to ensure full consolidation.

It is well known that water content is related to the shear strength of the soil. Few water content tests were carried out to examine the difference in water content between the failure surface and the other sides of the sample. On average the water content of the failure
surface was higher than that of the other sides of the sample by 10-14%. These values were influenced by water penetrating from the water bath of the cell on dismantling. The increase of water content on the failure surface is due to the immigration of water from the sample to the plane of separation in order to soften the failure surface during the shearing process to obtain minimum possible potential resistance.

Trials were undertaken to attempt measurement of clay fraction of the failure surface. However, they were unsuccessful due to the complex nature of the clay content test itself.

Residual strength values are summarised in Table 7.1.

**Table 7.1: Residual Strength Values Obtained by the Reversible Shear Box Samples for Liias Clay**

<table>
<thead>
<tr>
<th>Sample Size and Type</th>
<th>Minimum $\phi_r$ deg</th>
<th>Maximum $\phi_r$ deg</th>
<th>Maximum $\Delta\phi_r$ deg</th>
<th>$P_{cr}$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 x 100 mm reconstituted pre-cut samples</td>
<td>12.1</td>
<td>19.5</td>
<td>7.5</td>
<td>150</td>
</tr>
<tr>
<td>60 x 60 mm reconstituted pre-cut samples, multi-stage technique</td>
<td>13.0</td>
<td>23.0</td>
<td>10.0</td>
<td>200</td>
</tr>
</tbody>
</table>

Table 7.1 shows the difference in the results obtained. Smaller shear box samples gave higher residual strength values on both minimum and
maximum values of $\phi'$. Minimum values of $\phi'$ were obtained beyond the $P_{cr}$ of each type, while maximum $\phi'$ were obtained at normal stresses of 30 and 27 kPa respectively. The latter could not be considered as realistic residual strength values as their failure surfaces did not show any evidence of slickensides.

The difference in the minimum value of $\phi'$ is 0.9 degree and the difference in the maximum $\Delta\phi'$ is 2.5°. This means that at low normal stresses below $P_{cr}$ values of $\phi'$ are highly influenced by the size of sample, while for higher normal stresses the problem is less. Such difference is related to the difference in the friction losses between the two types of the samples, therefore the determination of residual strength values at low normal stresses requires more care as it is highly influenced by the testing technique.

This finding can be supported by the laboratory observation that with the smaller sample more soil was extruded. Trials were made to estimate the difference in weight of the extruded soil, but this was not possible.

It should be noted that the determination of the value of $P_{cr}$ is speculative and needs judgement. Its value should be determined by plotting $\tan^{-1} \phi'$ with normal stress as it gives a clearer presentation than the residual failure envelope.

The minimum value of $\phi'$ obtained in these tests is 12.1° which lies between two other values obtained from published data. Chandler (1970) found $\phi'$ on a principal shear surface to be 18.5° whereas Blondeau (1973) quotes a much lower value of $\phi'$ for the Lias clay of 9.0°.
7.2.1.2 Ring shear tests

As described before in Chapters 3 and 4, no reliable results were obtained with the original Branhead ring shear apparatus due to the problem of unbalanced forces exerted on the sample encountered with testing.

Lias clay was tested using two different rates of shearing which were: 0.178 mm/min and 0.0178 mm/min. In the fast shearing series, samples were sheared to a displacement in excess of 300 mm. In most of the cases the residual strength was developed at a displacement much lower than the total displacement applied. Figures 6.29 to 6.32, the shear stress-displacement relationships show that there is a fast drop from the peak values towards their residual values and this reflected the sample preparation method in which the clay is orientated as a pre-cut sample in the cell. In addition the failure surface is almost flat, so as soon as the asperities of the surface were destroyed the shear strength dropped to its residual value.

Initially it was expected that side friction between the upper half of the sample (which is stationary) and the side of the sample container (which rotates) might lead to errors. It is now believed that the friction, if any in this region, is very small because some soil is extruded at first from the (0.3 mm thick) vertical gap between the sample container and the loading platen but the process gradually diminishes. It is the benefit of the vertical gap that the soil is extruded initially to fill the gap which in turn prevents more soil being extruded in the vertical direction. Experience with the apparatus revealed that the amount of extruded soil could be significantly reduced by reducing the rate of shearing. Visual inspected proved this, see Plate 7.2.
This could be considered as the main improvement on the original apparatus as with the latter, the soil is usually extruded leading to a continuous reduction in the sample thickness, which ultimately limits the duration of testing and causes an unreliable failure surface.

With the modified apparatus, a multi-stage technique with variable rates of shearing was used which proved to be of a much less time consuming nature. In conclusion the following could be concluded:

1. The modified apparatus provides shearing on a flat surface at the middle height of the sample with little or no side friction and with less extruded soil compared to the original apparatus.

2. The orientation of the cell gap in the vertical direction prevents water penetration from the container to the sheared face as it usually filled with extruded soil at the early stage of consolidation prior to the commencement of shearing. This prevents the surface remoulding which usually occurred with the original apparatus.

3. It is believed that the modified apparatus could be used successfully for testing soils containing a high percentage of fine granular materials, whereas with the original apparatus or the Bishop type of ring shear such soils could not be tested since, as found by Cunningham (1986), the fine particles could become trapped in the horizontally orientated gap during the shearing process leading to high side friction forces.
Residual strength values obtained from the ring shear apparatus are summarised in Table 7.2.

**TABLE 7.2: RESIDUAL STRENGTH VALUES OBTAINED BY THE RING SHEAR APPARATUS FOR THE LIAS CLAY**

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Minimum $\phi_r'$ deg</th>
<th>Maximum $\phi_r'$ deg</th>
<th>Maximum $\Delta \phi_r'$ deg</th>
<th>$P_{cr}$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single-stage</td>
<td>10.8</td>
<td>12.5</td>
<td>1.7</td>
<td>250</td>
</tr>
<tr>
<td>Multi-stage</td>
<td>10.5</td>
<td>12.4</td>
<td>1.9</td>
<td>200</td>
</tr>
</tbody>
</table>

Table 7.2 provides residual strength values which are almost identical. Differences between results are within the usual range of experimental errors. It should be noted that the modified apparatus provided a smooth residual strength envelope with no irregularities and the tests were relatively quick as they can be satisfactorily completed within one week for practical work.

Maximum values of $\phi_r'$ were obtained at a normal stress of 26 kPa, the latter could not be considered as a residual strength value of the same reason discussed with the shear box testing.

Tables 7.1 and 7.2 show that the maximum difference in the minimum $\phi_r'$ is 2.5° and maximum difference in $\Delta \phi_r'$ is 8.1° throughout. The lowest values were obtained by the ring shear and the highest values were obtained by 60 x 60 mm shear box, especially $\Delta \phi_r'$. Such differences
are considered high for residual strength measurements. This leads to the obvious conclusion that the ring shear test is the best technique for measuring the residual strength of reconstituted samples; the 60 x 60 mm shear box is not suitable for such measurement especially at low normal stresses. For practical purposes and in the absence of ring shear, larger shear box sizes should be used.
7.2.2 London Clay

London clay was tested using the modified Bromhead ring shear apparatus and the 100 x 100 mm shear box. The selection of methods of testing were determined during the testing of the Lias clay.

7.2.2.1 Reversible shear box tests

100 x 100 mm pre-cut samples were tested at a slow rate of shearing of 0.016 mm/min.

The residual strength results obtained show that the minimum $\phi_r'$ is 8.10$^\circ$, maximum $\phi_r'$ is 12.0$^\circ$ and $P_{cr}$ is 150-200 kPa. The maximum $\phi_r'$ obtained at a normal stress of 40 kPa; visual inspection of the failure surface shows some evidence of residual strength condition, so such $\phi_r'$ value could be considered realistic.

Skempton and Petley (1967) found that $\phi_r'$ for London clay which has a PI of 53% is 12.0$^\circ$ as obtained from the drained shear box test.

The water content test results obtained showed that the average water content of the failure surface is 28% and for other sides of the sample is 24%. The difference in water content obtained here is due to the same reason discussed before in Section 7.2.1.1. Shear box test results will be compared later with those obtained from the ring shear apparatus.

Skempton (1964) noted that a zone of softened clay characteristic of London clay and for three cases the water contents were approximately 35% in the softened zone as compared to a water content of 30% on either side of it.
7.2.2.2 Ring shear tests

London clay was tested using the multi-stage technique with a variable rate of shearing, both for loading and unloading sequences. The residual strength results obtained are summarised in Table 7.3.

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Minimum $\phi_r'$ deg</th>
<th>Maximum $\phi_r'$ deg</th>
<th>Maximum $\Delta\phi_r'$ deg</th>
<th>$P_{cr}$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading sequence</td>
<td>7.90</td>
<td>11.8</td>
<td>3.9</td>
<td>200</td>
</tr>
<tr>
<td>Unloading sequence</td>
<td>7.8</td>
<td>9.0</td>
<td>1.20</td>
<td>100</td>
</tr>
</tbody>
</table>

Typical examples of residual strength results for London clay using different types of ring shear are presented in Table 2.3 (Chapter 2). The minimum values of residual strength tabulated above lie at the lowest end of the range of values obtained for London clay. This is believed to be due to the shape of the developed failure surface which was almost plane (without undulations), see Plate 7.2.

As presented on many occasions in this thesis, at residual condition clay particles arrange themselves in a direction parallel to the direction of shearing. Thus logically if the clay is sheared again in an unloading sequence down to the low normal stresses zone, residual strength values should be the same as those above this zone. What was found here was different. London clay exhibited values of residual
PLATE 7.2: RESIDUAL FAILURE SURFACE OF LONDON CLAY OBTAINED BY THE RING SHEAR APPARATUS
strength below 100 kPa normal stress, higher than the minimum value obtained beyond this stress.

Figure 7.1 demonstrates the variation of $\tan^{-1} \phi_r'$ with normal stress for London clay using three different testing techniques. On loading sequence shearing type, both ring shear and shear box gave almost the same variation, while with the unloading sequence ring shear tests, the variation is much less, however it is still significant. The latter suggests that there should be an appreciable amount of experimental error at low normal stress tests.

Throughout testing it was extremely difficult to get a perfect vertical load due to the slight non-uniform deformation of the sample or due to other mechanical defects. This inclination causes slight rearrangement of the clay particles in the shear zone, as the clay particles have a tendency to arrange themselves in a direction perpendicular to that of the major principal stress (Duncan and Seed, 1966). This leads to higher residual strength values. Such problems could be worst in the case of loading sequence tests and it is highly expected that such errors are not significant at higher normal stress.

Minimum values of $\phi_r'$ for London clay obtained from both ring shear and shear box are almost the same which is about $8.0^\circ$ with a maximum difference of $0.3^\circ$.

Shearing of London clay shows a clearer sliding mode than the Lias clay as their failure surfaces were more shiny, the latter produced with low strength shear surfaces of strongly orientated particles. This hints that the sliding mode clays are less affected by experimental errors.
Figure 7.1  Variation of $\tan^{-1} \phi'$ vs Normal Stress
7.2.3  Baghdad Clay

Baghdad clay was tested using the shear box samples and ring shear apparatus for both clay samples and composite samples of clay and concrete.

7.2.3.1  Reversible shear box tests

100 x 100 mm shear box pre-cut clay samples and composite samples of clay and concrete were tested using the recommended method of testing the Lias clay. Table 7.4 summarises the results obtained.

<table>
<thead>
<tr>
<th>Type of Sample</th>
<th>Minimum $\phi'_r$ deg</th>
<th>Maximum $\phi'_r$ deg</th>
<th>Maximum $\Delta\phi'_r$ deg</th>
<th>$P_{cr}$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay samples</td>
<td>26.0</td>
<td>32.0</td>
<td>6.0</td>
<td>200</td>
</tr>
<tr>
<td>Composite samples</td>
<td>27.0</td>
<td>34.0</td>
<td>7.0</td>
<td>150</td>
</tr>
</tbody>
</table>

Maximum values of $\phi'_r$ were obtained at a normal stress of 20 kPa. These values were not considered as a realistic residual strength parameter for the same reason discussed with the Lias clay and London clay.

Tests on composite samples revealed lower $P_{cr}$ values due to the role of concrete.
Generally, the difference in the residual strength values are referred to experimental error only.

Test results showed that the residual strength values were higher than the other two British soils and the drop from peak values towards the residual values was relatively small. This is due to the presence of coarse soil particles as will be discussed later in the mineralogical analysis. Visual inspection on completion of the tests revealed that the failure surfaces were of a non-slickenside type as shown on Plate 7.3.

Few water content tests indicated that the water content on the failure surfaces was higher than the other sides of the sample by 7-10%. The difference is due to the same reason discussed for Lias clay.

For this type of soil there is no other residual strength data published yet. Baghdad clay is a sedimentary soil with an overconsolidation ratio ranging between 3 to 10. No figures or slip surfaces exist in the soil deposit and therefore slope stability problems are not usually controlled by the residual strength of the soil. Typical deep foundation problems are those relating to piles or pipe jacking. As these problems involve large displacement shearing, it was decided to conduct tests on composite samples of clay and concrete to understand their behaviour on large displacement shearing.

Figures 6.55 to 6.58 show the shear stress-displacement relationships of the clay samples and Figures 6.61 to 6.63 show the shear stress-displacement relationships of the composite samples. There is no distinguishable difference in the displacement required to develop the residual strength of the two types of samples tested at the same
PLATE 7.3: 100 x 100 mm SHEAR BOX COMPOSITE SAMPLE OF BAGHDAD CLAY AND CONCRETE
normal stress. However, the residual envelope of the composite samples (Figure 6.64) shows lower curvature than the residual envelope of the clay samples (Figure 6.59).

Therefore the conclusion drawn by Kanji and Wolle (1977) which states that the concrete surface plays an important role in helping the clay particles to orientate themselves in the direction of shearing, is not valid here. Such a conclusion should be considered with caution when it is applied to different types of soil; it could be fully valid for soils exhibiting sliding mode of shearing only.

7.2.3.2 Ring Shear Tests

Baghdad clay was also tested using the ring shear apparatus for both clay samples and composite samples. Table 7.5 summarises the residual strength results obtained.

<table>
<thead>
<tr>
<th>Type of Sample and Test</th>
<th>Minimum $\phi_r'$ deg</th>
<th>Maximum $\phi_r'$ deg</th>
<th>Maximum $\Delta\phi_r'$ deg</th>
<th>$P_{cr}$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay samples</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>loading sequence</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>24.0</td>
<td>28.80</td>
<td>4.8</td>
<td>200</td>
</tr>
<tr>
<td>Composite samples</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>loading sequence</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>24.7</td>
<td>29.0</td>
<td>4.3</td>
<td>200</td>
</tr>
<tr>
<td>Composite samples</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>unloading sequence</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>27.4</td>
<td>28.0</td>
<td>0.6</td>
<td>150</td>
</tr>
</tbody>
</table>
Residual strength values obtained from the ring shear were lower than those obtained from the shear box for the main reason discussed. Minimum values obtained beyond $P_{cr}$, where maximum values obtained at normal stresses ranging between 26-52 kPa. Both types of samples show almost the same results with a slight difference due to the experimental errors, while tests of composite samples on unloading sequence show a marked increase in the residual strength values. Visual inspection of the failure surface revealed that the failure surface was undulant as shown on plate 7.4. It is believed that the main reason behind that was the nature of the concrete surface which had some small cracks produced by subsequent loading and unloading. More tests with better quality concrete are required to confirm this observation. In any case test results showed a curved residual failure envelope (Figure 6.78) with a lower curvature than that which was obtained with the same condition by the shear box, the latter the affected by the stress concentration imposed by the edges of the concrete.
PLATE 7.4: PIECE OF THE FAILURE SURFACE OF BAGHDAD CLAY SHEARED AGAINST CONCRETE ON UNLOADING SEQUENCE
7.3 MAIN CONCLUSIONS ON THE LABORATORY MEASUREMENTS

1. The shear box and ring shear do not produce identical residual failure envelopes. Values of residual strength obtained by the ring shear are always lower than those obtained by the shear box. This is due to the experimental problems associated with the shear box, such as an excessive soil extrusion which leads to an appreciable amount of side friction and the undesirable tilting produced during long displacement. This can be reduced to some extent by using larger shear box samples.

2. The use of fast forward and backward shearing to generate a failure surface followed by slow shearing is the best technique of measuring the residual strength of reconstituted shear box samples. This is due to the saving in time and to the reduced possibility of producing an undulating shear surface, which may lead to a locally greater shear resistance.

3. The complete residual failure envelope can be better determined by the modified Bromhead ring shear apparatus and in a faster time than by the shear box for all types of soil. This is due to the nature of the failure surface obtained by the apparatus which is almost flat and within the mid-height of the sample, and that the problem of side friction, which can develop with other types of ring shear apparatus has been reduced significantly. Experience with the apparatus has proved that there is a minimum amount of soil being extruded during shearing, which permits the same sample to be tested under different normal stresses. The residual strength mobilised along soil-structure interfaces could also be determined easily by this apparatus using a composite
sample of clay and the material under consideration. The modified apparatus can be used by any practising engineer or technician without difficulty due to its relative simplicity.

4. Residual strength parameters are not a soil property. The whole failure envelope should be taken into consideration rather than dependency on single parameters. The shape and degree of curvature are governed by the mineralogy of the soil under consideration, other soil parameters such as liquid limit or plastic limit should only be used as a guide.

5. Residual strength envelopes of all types of clays employed were curved on both loading and unloading conditions, with a lower degree of curvature on unloading condition. The cause of curvature in the case of the unloading condition is referred entirely to the experimental errors associated with the test at normal stresses below 100 to 150 kPa.

6. For practical purposes, it is not recommended to investigate the residual strength at normal stresses below 30 kPa in either the ring shear apparatus or shear box, as the residual strength cannot develop even for a displacement of 500 mm. This is mainly due to the physical characteristic of the clay particles as they do not orientate parallel to the failure plane at such normal stresses. Furthermore, the experimental errors usually associated with such stresses are high.

Values of C' obtained are very small or sometimes equal to zero, such values should be ignored as they are also related to the experimental errors.
7. Rate of shearing of 0.178 mm/min to 0.0178 mm/min or similar range has no effect on the results. For practical purposes it should not cause too much soil extrusion. The latter depends entirely on the type of soil and should be determined by some experimental trials.

The initial water content, as found by many other investigators, has no effect but it should be within a range so that it does not lead to a high initial soil extrusion.
7.4 RESIDUAL STRENGTH AT LOW NORMAL STRESSES

The residual strength of clay at low normal stresses was the main objective of this work. Many investigators have referred to this aspect of residual strength of clay as many landslides or natural slopes occur in nature at a relatively shallow depth.

Figures 7.2 and 7.3 show the residual strength envelope and the variation of \( \tan^{-1} \phi' \) with normal stress respectively for the three types of soil under examination. All show that there is a variation of \( \tan^{-1} \phi' \) with normal stress up to a normal stress of 250 kPa. However as presented before in Chapter 2, Figure 2.3, such a variation could occur up to a value of 500 kPa with some soils. Therefore, this problem should be investigated for all types of soils at any depth. In other words, the whole residual failure envelope should be determined.

To understand the mechanism of such behaviour, the current research programme was divided into three parts which are:

1. Laboratory tests
2. Theoretical background and explanation.

Final conclusions were drawn from the outcome of these three parts.
Ring Shear Tests, Loading Sequences
Rate of Shearing: 0.0178 mm/min

Figure 7.2 Residual Failure Envelopes for Three Clays
Figure 7.3 Variation of $\tan^{-1} \phi_r$ vs Normal Stress for Three Clays
7.4.1 Laboratory Tests Evidence

The extensive laboratory investigations carried out in this research project proved that all soils exhibit a non-linear residual failure envelope on both loading and unloading. The degree of curvature and the critical normal stress at which the shape of the curve changed to a linear relationship is also dependent on the type of soil. The critical normal stress $P_{cr}$ ranged between 150 to 250 kPa.

Part of the curvature is due to experimental errors as proved on unloading condition, while the major part is related to the parallelism of the clay particles in the shear zone. However it is extremely difficult to separate between them.

The laboratory tests gave the shape and amount of the variation of residual strength, where the actual cause cannot be fully determined. However, the sliding mode failure surfaces which were produced at high normal stresses such as 300 kPa showed a slickenside nature which is an indication of perfect parallelism. The latter was found for London clay and Lias clay, while Baghdad clay did not show such indications.

7.4.2 Theoretical Explanation of the Curved Residual Failure Envelope

Generally the shear strength of clay is governed by the equation:

$$\tau = \sigma_n \tan \phi'$$  \hspace{1cm} 7.1

assuming $C'$ equal to zero in the absence of cohesion forces.

The friction angle used above includes sliding of grains and particle rearrangement, which are both related to the original fabric of the soil.
Terzaghi (1920), as mentioned by Mitchell (1977), hypothesised that the normal load $N$, acting between two bodies in contact causes yielding at asperities where the actual interbody solid contact develops. The actual contact area $A_c$ is given by

$$A_c = \frac{N}{\sigma_y} \quad 7.2$$

where $\sigma_y$ is the yield strength of the material.

The maximum shearing force $T$ that can be withstood by the contact is then:

$$T = A_c \tau \quad 7.3$$

As the coefficient of friction is given by $T/N$,

$$\mu = \frac{T}{N} = \frac{\tau}{\sigma_y} = \tan \phi \quad 7.4$$

The basis of the adhesion theory of friction is presented in equation 7.4, namely the tangential force that causes sliding depends on the solid contact and the shear strength of the contact.

To gain a better understanding of the mechanism of the minerals contact, it seems very useful to demonstrate the mechanism of the mineral junction in terms of elastic and plastic behaviour as presented by Mitchell (1976).
1. **Elastic Junction**

For two spheres in contact, application of Hertz theory leads to:

\[ d = (\delta NR)^{1/3} \]

where \( d \) is the diameter of the plane circle area of the contact, \( \delta \) is a function of geometry, Poisson's ratio, and Young's modulus, and \( R \) is the sphere radius. On this basis Mitchell was able to correlate \( \tan \phi' \) to \( \sigma_n' \) as follows:

\[ \tan \phi' = \tau_i K (\sigma_n')^{-1/3} \]

where \( \tau_i \) is the shear strength of the contact and \( K = \pi (4\delta)^{2/3}/4 \).

On this basis the coefficient of friction should decrease with increasing \( \sigma_n' \), but it should be independent of sphere radius or particle size.

2. **Plastic Junction**

If asperities yield and deform plastically, then the contact area is proportional to the normal load on the asperity.

![Diagram](image)

**FIGURE 7.4:** PLASTIC JUNCTION BETWEEN ASPERITIES WITH ADSORBED SURFACE FILMS
If the contaminated film strength is $\tau_C$, the mineral contact $\tau_m$, then the strength of the contact will be

$$T = A_c [\delta \tau_m + (1-\delta) \tau_C]$$  \hspace{1cm} 7.6

However equation 7.6 cannot be applied in practice as $\delta, \tau_C$ are difficult to determine.

In conclusion, plastic and or elastic deformations are controlling the contact area at asperities of the clay particles and as it reaches the plastic condition, soil attains a constant residual strength. This in turn is related to the mineralogy of the clay itself, so if there are higher percentages of platy clay particles, they will behave plastically while if there are high percentages of bulky shaped clay particles, the clay will behave elastically and will not cause too much drop from the peak value towards its residual value as found with Baghdad clay.

Some investigators have suggested that $\tan^{-1} \phi' \tau$ is related to $\sigma^{-1/3}$. The latter resembles the purely elastic condition, so such relation has no significant practical nature.
7.4.3 Soil Fabric Studies

This research was carried out on natural soils in order to give the research a practical character. The main difficulty in carrying out such research was the mineralogical investigation which by itself could be an individual research project.

X-ray diffraction tests and SEM photographs for different failure surfaces at different normal stresses using two different methods of drying, as described in Chapter 5, are presented in Appendices A and B respectively.

Lack of personal experience with the X-ray diffraction tests was one of the problems encountered in this research, since it is extremely difficult to evaluate numerically the percentage of clay minerals existing in each type of clay as it needs a very long time and skill. However the dominant clay minerals for each type of clay were determined as shown in Figures A.1 to A.3 (Appendix A).

Kenney (1977) found that the numerical accuracy of the mineral-composition determinations of natural soils needs great amounts of time and effort. His interpretations were based on approximate and indirect correlations.

Table 7.6 presents mineralogical and geological information for different types of massive and clay minerals. It is presented here for reference.

As found from laboratory test results, Baghdad clay has a relatively higher residual strength value than the other types of clay. The main reason behind this is the presence of a relatively high percentage of illite/mica group as shown in Figure A.3. These particles have a sheety shape and high $\phi_r'$. value.
<table>
<thead>
<tr>
<th>Mineral</th>
<th>Mode of Cleavage</th>
<th>Bonding along Cleavage Planes</th>
<th>$\phi'_{r}$</th>
<th>Particle Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>No definite cleavage along (110) plane</td>
<td>Si-O-Si, weak</td>
<td>35 degrees</td>
<td>Bulky</td>
</tr>
<tr>
<td>Attapulgite</td>
<td></td>
<td></td>
<td>30 degrees</td>
<td>Fibrous and needle-shaped</td>
</tr>
<tr>
<td>Mica</td>
<td>Good vasal (001)</td>
<td>Secondary valence (0.5 to 5 kcal/mole) + K - linkages</td>
<td>17 to 24 degrees</td>
<td>Sheet</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>Basal (001)</td>
<td>Secondary valence (0.5 to 5 kcal/mole) + H-bonds (5-10 kcal/mole)</td>
<td>12 degrees</td>
<td>Platy</td>
</tr>
<tr>
<td>Illite</td>
<td>Basal (001)</td>
<td>Secondary valence (0.5 to 5 kcal/mole) + K linkages</td>
<td>10.2 degrees</td>
<td>Platy</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>Excellent basal (001)</td>
<td>Secondary valence (0.5 to 5 kcal/mole) + exchangeable ion linkages</td>
<td>4 to 10 degrees</td>
<td>Platy-filmy</td>
</tr>
<tr>
<td>Talc</td>
<td>Basal (001)</td>
<td>Secondary valence (0.5 to 5 kcal/mole)</td>
<td>6 degrees</td>
<td>Platy</td>
</tr>
<tr>
<td>Graphite</td>
<td>Basal (001)</td>
<td>van der Waal's</td>
<td>3 to 6 degrees</td>
<td>Sheet</td>
</tr>
<tr>
<td>MoS$_2$</td>
<td>Basal (001)</td>
<td>Weak interlayer</td>
<td>2 degrees</td>
<td>Sheet</td>
</tr>
</tbody>
</table>

Adapted from Chattopadhyay (1972)
However, it should be noted that during sample preparation, it was difficult to obtain pure clay minerals unless a perfect sedimentation process was performed. Therefore the test results were highly influenced by the presence of other massive minerals. X-ray diffraction tests for the natural Baghdad (Figure A.4) soil showed that the original soil contained some quartz minerals which have a bulky shape with a $\phi_r$ of 35°. It is the belief of the author that the presence of such massive minerals on the failure surface had an influence on the residual strength values and the shape of residual envelope obtained.

The other British clays have a lower illite/mica group and higher kaolinite minerals (Figures A.1 and A.2). This accounts for the difference between them and Baghdad clay. Kenney (1977) claimed that in the case of soils and mineral mixtures in which the dominant clay mineral is hydrous mica, unexplained anomalies exist among the residual strength results.

Different (SEM) photographs are presented in Appendix B of this thesis. The photographs were taken for samples tested under different normal stresses at their failure surfaces and on the opposite unsheared face. Three different magnifications were employed which were 1K, 10K and 20K or the nearest possible.

The accuracy of the photographs was influenced by many technical defects in addition to those mentioned in Chapter 5 concerning the methods of sample preparation, adjusting of the sample position and the surface disturbance were the main obstructions in focusing the exact shearing paths throughout the failure surface.
In Appendix B there are some distinguishing photographs which show the shearing path or the orientation of clay particles for example:

1. Plates B.3: shows a clear shearing path in a circular direction produced by testing London clay in the ring shear apparatus at a normal stress of 182 kPa.

2. Plates B.4: shows a clear orientation of a kaolinite particle in the shear zone for a sample of London clay tested in the ring shear apparatus at a normal load of 208 kPa.


4. Plates B.17: shows some development of the clay particle orientation in the shear zone for a sample of Lias clay tested in the ring shear at a normal stress of 208 kPa.

5. Plates B.21 and B.23: show the partial development of the clay particles orientation in the shear zone for samples of Baghdad clay tested in the shear box at normal stresses of 200 and 300 kPa respectively.

From what is described before, it seems that the (SEM) photographs gave some evidence of perfect orientation of clay particles in the shear zone beyond the critical normal stress, $P_{cr}$, especially for clays exhibiting sliding mode of shearing.
7.5 MAIN CONCLUSION

1. The residual failure envelope for all types of soils employed in this research was curved up to a certain critical normal stress beyond which it becomes a straight line. The shape and degree of curvature is entirely controlled by the shape of the dominant clay mineral. With soils of more platy particle shape, both the degree of curvature and \( P_{cr} \) are higher than the other type of soil; also they show more brittle stress-strain relationships.

2. The residual failure envelope becomes a straight line when the clay particles in the shear zone orientate themselves in a horizontal direction and attain a complete plastic junction at contact.

3. Measurements of residual strength at low normal stresses are more influenced by experimental error than those beyond the critical normal stress. Ring shear apparatus can give more realistic results than the shear box.

4. No empirical relationship can govern the shape and degree of curvature of the residual failure envelope, however there is a general trend that \( \tan \phi_r' \) could relate to \( \sigma_r^{-1/3} \) for purely elastic particle contents. For practical purposes, the whole residual failure envelope should be taken into consideration in a condition similar to that expected to occur in nature.

5. There is no unique displacement at which the residual strength of clay is produced at different normal stresses. Thus for a natural general failure zone, it is highly cautioned to adopt a
certain displacement at which the residual strength is expected to produce. The required displacement should be determined from laboratory tests at the corresponding normal stress.

6. Shearing against hard surfaces such as concrete have no significant role in reducing the shape and degree of curvature for clay having no platy particle shape. It is expected to do better for platy shape particles. The validity of the latter statement was proved later at Loughborough by Rouaiguia (1990).

7. SEM photographs are a useful guide in examining the failure surface, however they need a lot of skill and experience to interpret and are often inconclusive.
CHAPTER 8
SUGGESTIONS FOR FURTHER WORK

8.1 INTRODUCTION

The various aspects of residual shear strength have been extensively investigated over the last 26 years by many researchers, as mentioned in the literature survey (Chapter 2).

In the main, these researchers have been concerned with methods for determining the residual strength in the laboratory and the study of the factors that affect it, such as the shear rate, the stress history and so on. Comparison between values obtained in the laboratory and in the field, and correlations between the residual strength of soil and other soil parameters, such as clay fraction and plastic limit, have also been presented.

In order to improve the understanding of the subject, four areas of further research are suggested here. It is the belief of the author that the first two (subsections 8.2.1 and 8.2.2) will fill in gaps in the research carried out so far. The other two (subsection 8.2.3 and 8.2.4) arose out of the present research on low normal stresses.

8.2 SUGGESTED FURTHER INVESTIGATIONS

8.2.1 Effect of Temperature on the Residual Strength of Clay
The literature shows that very little investigation has been carried out regarding the effect of temperature on the drained residual strength of clay soil. The main exception is the work of Bucher (1975), who claimed that within a temperature range of 10°C to 60°C
the measured residual strength is unaffected. This observation seems to be inadequate as its findings cannot be generalised to a higher temperature range similar to that produced during very fast landslides, as in the special case of air-entrapped landslides which may reach 300 km/hr (Perry, 1985). Heat is inevitably produced along such landslide slip zones due to the friction produced by the huge soil mass sliding.

Generally, the increase in temperature has two opposing effects on the shear strength of clay, which are:

1. The clay particles have a tendency toward flocculation on temperature increase (Lambe and Whitman, 1969), therefore the shear strength will increase due to the increase in the resistance between the clay particles in such a particle assembly.

2. As the temperature increases, salt concentration increases due to water evaporation. The effect is to increase the degree of parallelism between adjacent particles since the attraction between the particles is of the secondary valence type. This in turn leads to a reduction in the shear strength due to the parallelism of clay particles.

Therefore without carrying out an actual test at the required temperature, it is quite difficult to forecast the results. To do so, it is first required to determine the actual field temperature and second to select the proper method of testing. Therefore the following is suggested.
1. The field temperature can be estimated by using the thermal infra-red technique (Moffat, 1989). It uses sophisticated aerial photographs to estimate the degree of temperature of the sliding surface depending upon the degree of brightness of red spots appearing in the photographs. However, this technique could be expensive and complicated.

2. Residual strength measurements are to be carried out on 100mm diameter pre-cut triaxial samples in order to run the test under controlled temperature. Heat could be supplied by using a submerged electrical coil. However, the required temperature could be maintained by balancing the supplied temperature and the cell pressure. Special load cells, rubber membranes, etc should be manufactured to sustain the high temperature. In any case, it may not be possible to reach temperatures as high as that produced in the field. Doing so, it is expected to get residual strength results at a temperature range higher than that recorded by Bucher (1975).

8.2.2 Shear Strength Improvements Along Residual Failure Surfaces

Many slope stability problems involve failure surfaces at their residual state such as old landslides, faults, etc. The use of supporting structures such as retaining walls and ground anchors are usually considered as the best available solution to a geotechnical problem. Such solutions are usually expensive and complicated.

To get an alternative better solution, it is required to improve the shear resistance along such failure surfaces in order to prevent further movement, therefore it seems beneficial to carry out a laboratory investigation involving improvements of the residual strength along the failure surfaces.
Here are two suggested methods of shear strength improvements.

1. **Cation Replacement**
   It is well known from the soil mechanics literature that in this electro-mechanical method soil water tends to flow away from the anode and towards the cathode, thereby helping to reduce pore water content and consolidate the soil. Perry (1985) mentioned that Gedney and Weber (1978) found that cation replacement along a failure surface can increase the soil strength by up to 300%.

2. **Stabilising Methods**
   Using stabilisers such as lime or other injected chemicals along the pre-determined failure surface, shear strength will be highly improved.

Adopting these methods of shear strength improvement, a laboratory measurement of residual strength could be carried out which involves measurements of residual strength on the pre-determined failure surface developed in the shear box before and after improvement. The amount of shear strength improvement is an indication of their applicability.

The cost of each method should also be taken into consideration. Finally, the best selected method is the cheapest and most adequate method.
8.2.3 Measurements of Shear Strength of Clay Adjacent to the Interface of Piles, Pipe Jacking and Other Ground Construction

As it has been shown before in this research and other previous research, there is a reduction in the angle of shearing resistance of clay particles in any clay when it undergoes large displacements. As a limit it reaches the residual value. Therefore it is expected that the shear strength of clay which is adjacent to piles and pipe jacking construction will reach its residual value as shearing in such cases is relatively high. In addition, hard surfaces such as concrete and steel encourage the particles to arrange themselves parallel to the plane of shearing, Kanji and Wolle (1977). Anderson et al (1985), Anderson (1988) and Coop and Wroth (1989), have found that shear planes are created in the clay adjacent to the interface of piles and subsequently only small movements are required to mobilise the residual shearing resistance of the soil.

Therefore, to investigate this effect tests of clay shearing against hard surfaces are suggested. Both shear box and ring shear could be used for this purpose as discussed before in Chapter 3. Similar loading conditions and rate of shearing should be applied during testing.

Another set of shear box tests should be performed on clay samples taken from the interface of piles or pipe jacking laboratory model. Comparisons between the results of the two types of tests will point to the reliability of using the conventional apparatus for this purpose.
8.2.4 Measurement of Field Residual Strength at Low Normal Stresses

As the current research involves only laboratory investigation of residual strength of clay at low normal stresses, it will be very beneficial to extend this research to the field investigation of residual strength characteristics on existing slip surfaces at shallow depths and to re-create those characteristics in the simple ring shear apparatus. A few shear box samples can be cut from larger ones taken from sea cliffs at shallow depths after erosion and subsequent drying.

Different shear box sizes should be tried, it could be very worthwhile to use a very large shear box, such as the 300 x 300 mm box, in order to investigate the effect of sample size on the measured values of residual strength. Mineralogical studies are essential for such research.
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Marsh, A.D. (1972). Determination of the residual shear strength of clay by modified shear box method, TRRL Report LR 515, Transport and Road Research Laboratory, Crowthorne, Berks.

Matthews, M.C. (1987). The engineering application of direct and simple shear testing. A report of the discussion organised by the British Geotechnical Society and held at the Royal Institution of Chartered Surveyors.


APPENDIX A

X-RAY DIFFRACTION TEST RESULTS
FIGURE A.1: X-RAY DIFFRACTION PATTERN OF LIAS CLAY
(≤2 µm clay fraction)
FIGURE A.2: X-RAY DIFFRACTION PATTERN OF LONDON CLAY
(<2 μm clay fraction)
FIGURE A.3: X-RAY DIFFRACTION PATTERN OF BAGHDAD CLAY
(<2 μm clay fraction)
APPENDIX B

SCANNING ELECTRON MICROSCOPE PHOTOGRAPHS
Type of soil: London clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 104 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 2000 times)
Plate 2: Unsheared surface (magnified 10,000 times)
Plate 3: Sheared surface (magnified 2000 times)
Plate 4: Sheared surface (magnified 10,000 times)

PLATES B.1: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 104 kPa)
PLATES B.1
<table>
<thead>
<tr>
<th>Type of soil:</th>
<th>London clay</th>
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</thead>
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<tr>
<td>Sample preparation:</td>
<td>Air drying method</td>
</tr>
<tr>
<td>Normal stress:</td>
<td>156 kPa</td>
</tr>
<tr>
<td>Rate of shearing:</td>
<td>0.0178 mm/min</td>
</tr>
<tr>
<td>Type of shearing:</td>
<td>Soil to soil</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plate 1</th>
<th>Unsheared surface (magnified 2000 times)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate 2</td>
<td>Unsheared surface (magnified 10,000 times)</td>
</tr>
<tr>
<td>Plate 3</td>
<td>Sheared surface (magnified 2000 times)</td>
</tr>
<tr>
<td>Plate 4</td>
<td>Sheared surface (magnified 10,000 times)</td>
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</tbody>
</table>

PLATES B.2: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 156 kPa)
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<thead>
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<tr>
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</tr>
<tr>
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</tr>
<tr>
<td>Type of shearing:</td>
<td>Soil to soil</td>
</tr>
</tbody>
</table>

| Plate 1 | Unsheared surface (magnified 1000 times) |
| Plate 2 | Unsheared surface (magnified 10,000 times) |
| Plate 3 | Unsheared surface (magnified 20,000 times) |
| Plate 4 | Sheared surface (magnified 1100 times) |
| Plate 5 | Sheared surface (magnified 11,000 times) |
| Plate 6 | Sheared surface (magnified 22,000 times) |

PLATES B.3: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 182 kPa)
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<thead>
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<th>Type of soil</th>
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<tbody>
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</tr>
<tr>
<td>Sample preparation</td>
<td>Air drying method</td>
</tr>
<tr>
<td>Normal stress</td>
<td>208 kPa</td>
</tr>
<tr>
<td>Rate of shearing</td>
<td>0.0178 mm/min</td>
</tr>
<tr>
<td>Type of shearing</td>
<td>Soil to soil</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Plate 1</th>
<th>Unsheared surface (magnified 2000 times)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate 2</td>
<td>Unsheared surface (magnified 10,000 times)</td>
</tr>
<tr>
<td>Plate 3</td>
<td>Sheared surface (magnified 2000 times)</td>
</tr>
<tr>
<td>Plate 4</td>
<td>Sheared surface (magnified 10,000 times)</td>
</tr>
</tbody>
</table>

PLATES B.4: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 208 kPa)
Type of soil: London clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 260 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1  Unsheared surface (magnified 1200 times)
Plate 2  Unsheared surface (magnified 12,000 times)
Plate 3  Unsheared surface (magnified 24,000 times)
Plate 4  Sheared surface (magnified 1200 times)
Plate 5  Sheared surface (magnified 12,000 times)
Plate 6  Sheared surface (magnified 24,000 times)

PLATES B.5: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 260 kPa)
Type of soil: London clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 416 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1
Unsheared surface (magnified 2000 times)
Plate 2
Unsheared surface (magnified 10,000 times)
Plate 3
Sheared surface (magnified 2000 times)
Plate 4
Sheared surface (magnified 12,000 times)

PLATES B.6: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 416 kPa)
Type of soil: London clay
Type of test: Ring shear
Sample preparation: Freeze drying method
Normal stress: 156 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1
Unsheared surface (magnified 2000 times)
Plate 2
Unsheared surface (magnified 10,000 times)
Plate 3
Sheared surface (magnified 2000 times)
Plate 4
Sheared surface (magnified 10,000 times)

PLATES B.7: SEM PHOTOGRAPHS OF LONDON CLAY (RING SHEAR 156 kPa)
Type of soil: London clay
Type of test: Ring shear
Sample preparation: Freeze drying method
Normal stress: 208 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1  Unsheared surface (magnified 2000 times)
Plate 2  Unsheared surface (magnified 10,000 times)
Plate 3  Sheared surface (magnified 2000 times)
Plate 4  Sheared surface (magnified 10,000 times)

PLATES B.8: SEM PHOTOGRAFHS OF LONDON CLAY (RING SHEAR 208 kPa)
Type of soil: London clay
Type of test: Shear box
Sample preparation: Air drying method
Normal stress: 40 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1
Unsheared surface (magnified 2000 times)
Plate 2
Unsheared surface (magnified 10,000 times)
Plate 3
Sheared surface (magnified 2000 times)
Plate 4
Sheared surface (magnified 10,000 times)

PLATES B.9: SEM PHOTOGRAPHS OF LONDON CLAY (SHEAR BOX, 40 kPa)
Type of soil: London clay
Type of test: Shear box
Sample preparation: Air drying method
Normal stress: 340 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1 . Unsheared surface (magnified 1200 times)
Plate 2 Unsheared surface (magnified 12,000 times)
Plate 3 Unsheared surface (magnified 24,000 times)
Plate 4 Sheared surface (magnified 1200 times)
Plate 5 Sheared surface (magnified 12,000 times)
Plate 6 Sheared surface (magnified 24,000 times)

PLATES B.10: SEM PHOTOGRAPHS OF LONDON CLAY (SHEAR BOX, 340 kPa)
PLATES B.10
Type of soil: London clay
Type of test: Shear box
Sample preparation: Air drying method
Normal stress: 340 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1
Unsheared surface (magnified 2000 times)

Plate 2
Unsheared surface (magnified 10,000 times)

Plate 3
Sheared surface (magnified 2000 times)

Plate 4
Sheared surface (magnified 10,000 times)

PLATES B.11: SEM PHOTOGRAPHS OF LONDON CLAY (SHEAR BOX, 340 kPa)
Type of soil: London clay
Type of test: Shear box
Sample preparation: Freeze drying method
Normal stress: 60 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 1300 times)
Plate 2: Unsheared surface (magnified 13,500 times)
Plate 3: Unsheared surface (magnified 27,000 times)
Plate 4: Sheared surface (magnified 1300 times)
Plate 5: Sheared surface (magnified 13,500 times)
Plate 6: Sheared surface (magnified 23,000 times)

PLATES B.12: SEM PHOTOGRAPHS OF LONDON CLAY (SHEAR BOX, 60 kPa)
Type of soil: London clay
Type of test: Shear box
Sample preparation: Freeze drying method
Normal stress: 140 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1  Unsheared surface (magnified 2000 times)
Plate 2  Unsheared surface (magnified 10,000 times)
Plate 3  Sheared surface (magnified 2000 times)
Plate 4  Sheared surface (magnified 11,000 times)

PLATES B.13: SEM PHOTOGRAPHS OF LONDON CLAY (SHEAR BOX, 140 kPa)
Type of soil: London clay
Type of test: Shear box
Sample preparation: Freeze drying method
Normal stress: 240 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 2000 times)
Plate 2: Unsheared surface (magnified 10,000 times)
Plate 3: Sheared surface (magnified 2000 times)
Plate 4: Sheared surface (magnified 10,000 times)

PLATES B.14: SEM PHOTOGRAPHS OF LONDON CLAY (SHEAR BOX, 240 kPa)
Type of soil: Lias clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 52 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1
Unsheared surface (magnified 2000 times)
Plate 2
Unsheared surface (magnified 10,000 times)
Plate 3
Unsheared surface (magnified 20,000 times)
Plate 4
Sheared surface (magnified 1000 times)
Plate 5
Sheared surface (magnified 10,000 times)
Plate 6
Sheared surface (magnified 20,000 times)

PLATES B.15: SEM PHOTOGRAPHS OF LIAS CLAY (RING SHEAR, 52 kPa)
Type of soil: Lias clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 104 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 1000 times)
Plate 2: Unsheared surface (magnified 10,000 times)
Plate 3: Unsheared surface (magnified 20,000 times)
Plate 4: Sheared surface (magnified 1000 times)
Plate 5: Sheared surface (magnified 10,000 times)
Plate 6: Sheared surface (magnified 20,000 times)

PLATES B.16: SEM PHOTOGRAPHS OF LIAS CLAY (RING SHEAR, 104 kPa)
Type of soil: Lias clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 208 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 1000 times)
Plate 2: Unsheared surface (magnified 10,000 times)
Plate 3: Unsheared surface (magnified 20,000 times)
Plate 4: Sheared surface (magnified 1000 times)
Plate 5: Sheared surface (magnified 10,000 times)
Plate 6: Sheared surface (magnified 20,000 times)

PLATES B.17: SEM PHOTOGRAHS OF LIAS CLAY (SHEAR BOX, 208 kPa)
Type of soil: Lias clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 260 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 1200 times)
Plate 2: Unsheared surface (magnified 12,000 times)
Plate 3: Unsheared surface (magnified 24,000 times)
Plate 4: Sheared surface (magnified 1200 times)
Plate 5: Sheared surface (magnified 12,000 times)
Plate 6: Sheared surface (magnified 24,000 times)

PLATES B.18: SEM PHOTOGRAPHS OF LIAS CLAY (RING SHEAR, 260 kPa)
Type of soil: Baghdad clay
Type of test: Ring shear
Sample preparation: Air drying method
Normal stress: 100 kPa
Rate of shearing: 0.0178 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 1000 times)
Plate 2: Unsheared surface (magnified 10,000 times)
Plate 3: Unsheared surface (magnified 20,000 times)
Plate 4: Sheared surface (magnified 1000 times)
Plate 5: Sheared surface (magnified 10,000 times)
Plate 6: Sheared surface (magnified 20,000 times)

PLATES B.19: SEM PHOTOGRAPHS OF BAGHDAD CLAY (RING SHEAR, 100 kPa)
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<td>Normal stress:</td>
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</tr>
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<td>Soil to soil</td>
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<thead>
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<tr>
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</tr>
<tr>
<td>Plate 3</td>
<td>Unsheared surface (magnified 20,000 times)</td>
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<tr>
<td>Plate 4</td>
<td>Sheared surface (magnified 1000 times)</td>
</tr>
<tr>
<td>Plate 5</td>
<td>Sheared surface (magnified 10,000 times)</td>
</tr>
<tr>
<td>Plate 6</td>
<td>Sheared surface (magnified 20,000 times)</td>
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</tbody>
</table>

PLATES B.20: SEM PHOTOGRAPHS OF BAGHDAD CLAY (SHEAR BOX, 200 kPa)
Type of soil: Baghdad clay
Type of test: Shear box
Sample preparation: Air drying method
Normal stress: 100 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1
Unsheared surface (magnified 1000 times)
Plate 2
Unsheared surface (magnified 11,000 times)
Plate 3
Unsheared surface (magnified 20,000 times)
Plate 4
Sheared surface (magnified 1000 times)
Plate 5
Sheared surface (magnified 10,000 times)
Plate 6
Sheared surface (magnified 21,000 times)

PLATES B.21: SEM PHOTOGRAPHS OF BAGHDAD CLAY (SHEAR BOX, 100 kPa)
Type of soil: Baghdad clay
Type of test: Shear box
Sample preparation: Air drying method
Normal stress: 200 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1  Unsheared surface (magnified 1000 times)
Plate 2  Unsheared surface (magnified 10,000 times)
Plate 3  Unsheared surface (magnified 20,000 times)
Plate 4  Sheared surface (magnified 1000 times)
Plate 5  Sheared surface (magnified 10,000 times)
Plate 6  Sheared surface (magnified 20,000 times)

PLATES B.22: SEM PHOTOGRAPHS OF BAGHDAD CLAY (SHEAR BOX, 200 kPa)
Type of soil: Baghdad clay
Type of test: Shear box
Sample preparation: Air drying method
Normal stress: 300 kPa
Rate of shearing: 0.0160 mm/min
Type of shearing: Soil to soil

Plate 1: Unsheared surface (magnified 1200 times)
Plate 2: Unsheared surface (magnified 12,000 times)
Plate 3: Unsheared surface (magnified 24,000 times)
Plate 4: Sheared surface (magnified 1200 times)
Plate 5: Sheared surface (magnified 10,000 times)
Plate 6: Sheared surface (magnified 20,000 times)

PLATES B.23: SEM PHOTOGRAPHS OF BAGHDAD CLAY (SHEAR BOX, 300 kPa)