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THE UBC RING SHEAR DEVICE

by

BRUCE W. BOSDET

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Department of Civil Engineering

The University of British Columbia
2075 Wesbrook Place
Vancouver, Canada
V6T 1W5

Date October 10, 1980

ABSTRACT

The purpose of this thesis has been to design and develop a practical apparatus for determining the residual strength of clay soils. To provide background for the study, current knowledge regarding residual strength is reviewed, including the following points.

1. Residual strength, defined as the lowest drained strength a soil can exhibit, is attained at large shear displacements.
2. Residual conditions result when particles located in shear bands within the failure zone become aligned in the direction of shear.
3. Residual strength, derived from interparticle bonding, is influenced by crystal structure and, in active clay minerals, by pore water chemistry.

The ring shear test, performed by applying a torsional shear load to an annular shaped specimen, is particularly suited for residual strength determinations because of unlimited uni-directional strain capabilities. The UBC Ring Shear Device was designed to combine versatility with uncomplicated operation. Features of the design are as follows.

1. Variable sample height up to 0.75 inches.
2. Smoothly variable normal stress up to 200 psi delivered through an air piston.
3. Smoothly variable rate of shear from 3.2 inches per year to 9 inches per hour.

4. A non-tilting loading platen which reduces required machine tolerances and improves control of sample losses during testing.

5. Automatic data acquisition.

6. A simple method of sample placement.

Residual strength determinations obtained with the UBC Ring Shear Device demonstrate its efficient and effective operation. Minimal supervision is required and test results are easily interpreted. Multi-reversal direct shear tests for residual strength were undertaken for comparison with the ring shear results, but no satisfactory results were obtained due to excess pore pressures within the test specimens. Recommendations for improvements to both ring shear and direct shear devices are given.

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Many people have contributed to the completion of this project. Fred Zurkirchen, a machinist at U.B.C., patiently advised me in the practical aspects of the equipment design and constructed the ring shear device. Golder Associates, Vancouver, and Sydney Hillis, a consulting engineer in Vancouver, provided interesting soil specimens for the testing program. Friends and family provided the needed encouragement to complete the written work when the testing was done. However, this thesis would never have been completed without the aid of my supervisor, R.G. Campanella, Department of Civil Engineering, U.B.C., who suggested the project and provided support and guidance throughout.

CHAPTER I

INTRODUCTION

A soil subjected to shear displacement under drained conditions will normally exhibit a rise in shearing resistance with increasing displacement until a maximum resistance, the peak strength, is reached. With continued displacement, the shearing resistance of the soil will decrease until a minimum resistance, unaffected by further displacements, is attained. The minimum resistance, termed residual strength, is the lowest shearing resistance the soil can exhibit under drained conditions.

This general relationship between shear displacement and the drained shear strength of soil has long been recognized. In particular, peak strength and the behaviour of soils at small strains has been studied in great detail. However, the field of residual strength and soil behaviour at large strains has been largely unexplored until recent years. The relative inactivity in this field can be attributed to practical difficulties in testing to large strains and to an under-assessment of the importance of residual strength to the stability of earth structures.

Early studies into the residual strength behaviour of soils were carried out in the 1930's. The inability of conventional shear testing techniques to model field conditions by producing large uni-directional shear strains led the early researchers to devise large displacement rotational shear devices of varying design. Several of these devices performed torsional shear tests on annular soil specimens and, as such, were the first ring shear devices.

The initial interest in residual strength apparently stemmed from the recognition that residual strength, which could be as little as 20 percent of peak strength, governs the stability of previously failed structures (Hvorslev, 1939). This interest was shortly lost as attention turned to the study of peak strength behaviour and its broader application to unfailed slopes. However, attention returned to residual strength in the mid-1960's when residual strength was clearly linked to the instability of certain soils which had not previously failed.

In the Fourth Rankine Lecture, Skempton (1964) presented evidence to show that the long-term stability of slopes in overconsolidated fissured clays is governed, at least in the lower bound, by residual strength through the mechanism of progressive failure. A progressive failure may occur if the load on a small section of a large soil mass surpasses the peak soil strength. As the peak strength is exceeded, the shearing resistance of the soil falls toward the residual value and the loads previously carried by the small section of soil are transferred to adjacent areas, causing the stresses in these areas to exceed the peak strength also. In this manner, a failure may progress throughout a large soil mass without ever causing a large scale movement. This is particularly true in heavily overconsolidated clays which generally pass the peak strength at relatively small strains. Bjerrum (1967) has indicated that an overconsolidated clay need not contain fissures for its long-term stability to be governed by residual strength.

The application of residual strength to the stability of unfailed slopes, as outlined by Skempton, sparked new research into the changes that occur in soil during the transition from peak to residual strength and led to the development of new rotational devices for determining the residual strength of soils. The UBC ring shear apparatus is one of these new devices.

The purpose of this thesis project has been to design, construct and test a practical apparatus for determining the residual strength of soil. During the course of the study, available literature was reviewed to develop an understanding of residual strength behaviour and the historical development of the residual strength apparatus. Chapter II describes residual strength behaviour and reviews current knowledge and theory regarding the causes of this behaviour. Chapter III briefly reviews and evaluates the commonly used methods for determining residual strength and traces the historical development of ring shear devices. The design of the UBC Ring Shear Device is described in detail in Chapter IV. Chapter V presents the results of the testing program undertaken to evaluate the new ring shear device and to explore testing methods and procedures. The results of repeated direct shear tests, conducted for purposes of comparison, are also presented and discussed. A summary of the work and an evaluation of the equipment is provided in Chapter VI, together with recommendations for improvements to the apparatus.

CHAPTER II

RESIDUAL STRENGTH BEHAVIOUR OF CLAYS

2.1 INTRODUCTION

Considerable advances have been made since the mid-1960's in our understanding of soil behaviour at large strains. It has been shown that granular and clayey soils exhibit similar macroscopic behaviour in shear, but that only the clayey soils, and to a lesser extent dense sands, exhibit the significant drop from peak to residual strength which results in important engineering applications. For this reason, clays are the focus of attention in the study of residual strength.

This chapter provides a review of clay behaviour in shear and the microscopic mechanisms which influence the residual strength of these soils.

2.2 SHEAR BEHAVIOUR OF CLAY

All soils subjected to shear displacement under drained conditions will eventually arrive at a shear strength and water content that are functions of the soil composition and applied stresses only, regardless of the initial state of the soil. The strength of the soil is then the lowest that can be achieved and is termed the ultimate or residual strength. Further shearing will not affect the strength of the soil once the residual condition is established. Because residual strength is unaffected by the initial structure or stress history of the soils, it is sometimes considered to be a fundamental soil parameter, an unchanging characteristic which identifies the soil.

The stress-strain and volume change characteristics exhibited by a clay before reaching the residual condition are functions of the stress history of the soil. Heavily overconsolidated clays have high peak strengths and generally exhibit a large decrease from peak to residual strength. The reduction in strength is accompanied by an increase in void ratio and water content. Normally consolidated clays generally have lower peak strengths than overconsolidated clays and exhibit a smaller decrease from peak to residual strength. This decrease in strength is accompanied by a reduction in void ratio and water content. This typical soil behaviour is illustrated in Figure 2.1.

As with peak strength behaviour, the residual strength of a clay is described in terms of a friction angle, ϕ'_r, and a cohesion intercept, c'_r, as follows:

$$\tau_r = c'_r + \sigma' \tan \phi'_r$$

$$= c'_r + (\sigma - u) \tan \phi'_r$$

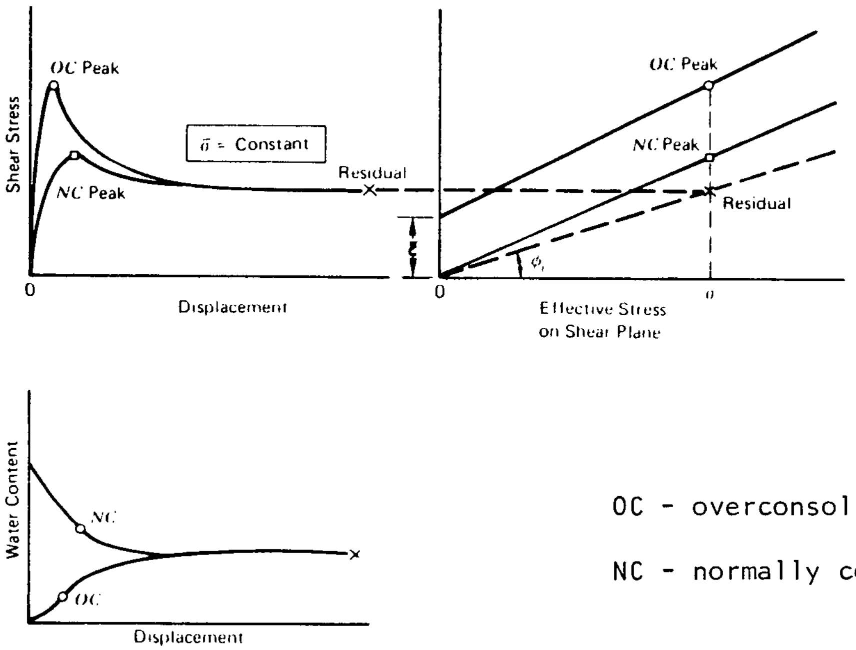
where τ_r = residual shear stress

σ = total normal stress on the shear plane

u = pore water pressure

$\sigma' = \sigma - u$ = effective normal stress

For most natural soils, the residual strength cohesion intercept is close or equal to zero, and the friction angle is less than the peak friction angle. The residual strength behaviour is commonly described by the shearing resistance ratio, $\tau/\sigma' = \tan \phi'_r$ (for $c'_r = 0$)



OC - overconsolidated

NC - normally consolidated

FIGURE 2.1 Shear Behaviour of Clays Under Fully Drained Conditions

(from Mitchell, 1976)

2.3 FAILURE PLANE STRUCTURE

The physical process by which a failure plane develops in clay subjected to shear displacement has been studied by Morgenstern and Tchalenko (1967). Their microscopic observations of kaolin clay indicate that clay deforms in simple shear up to the displacement at peak strength. However, beyond the peak strength, the soil gradually develops a complex system of shear structures and particle orientations which lead to the establishment of a shear zone. At large displacements, as the clay approaches the residual condition, the shear zone consists of two thin shear bands, in which the microscopic plate-shaped clay particles are oriented in the direction of shear displacement, enclosing a region of compression textures, as shown in Figure 2.2. The compression textures result from the parallel alignment of clay particles in the regions between shear bands such that the particles are oriented with the basal crystal planes approximately perpendicular to the major principal stress.

The dominant mechanism of deformation at large strains is by slip between the basal cleavage planes of adjacent parallel aligned particles within the two thin shear bands. All deformations are accounted for by basal plane slip or particle rotation. Shear failure through particles does not appear to occur.

The study by Morgenstern and Tchalenko also showed that the particles must be in virtually perfect parallel alignment before the residual strength is obtained. Small local particle disorientations significantly increase the measured friction angle. Tests carried out during the study indicate that the initial orientation of samples composed of particles with a strong preferred parallel alignment

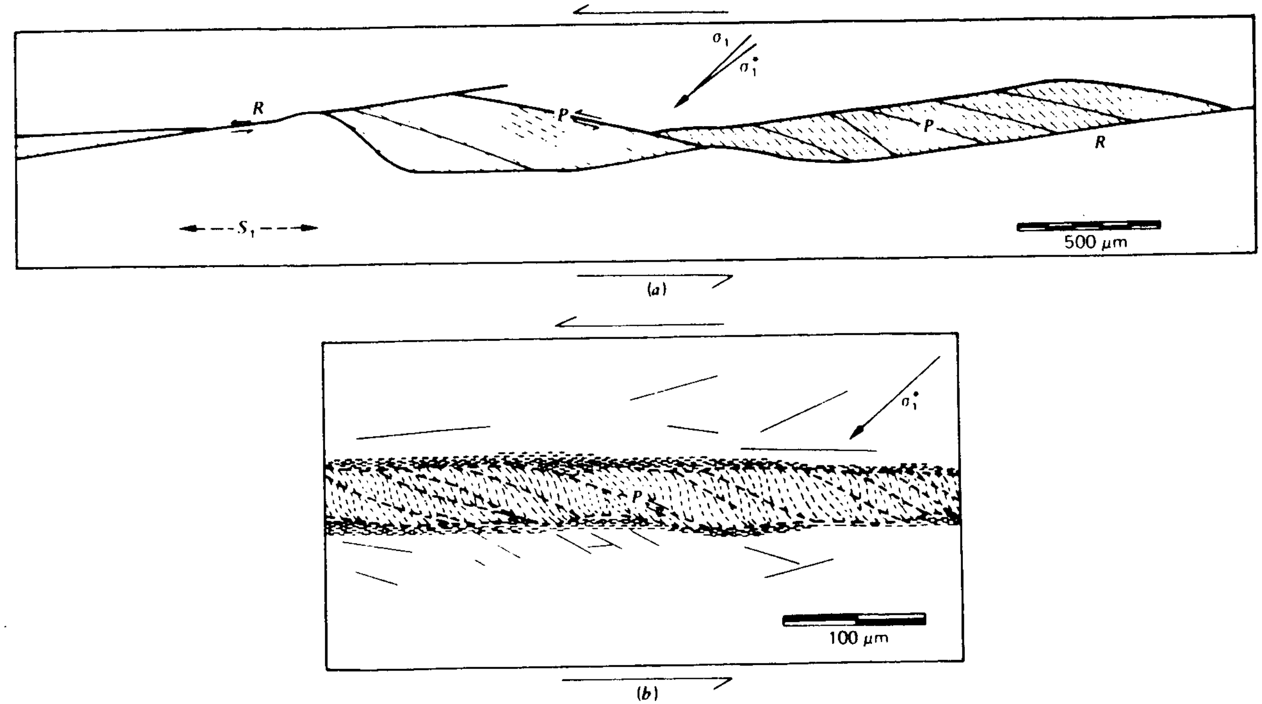


FIGURE 2.2 Shear Zone Textures and Fabric at the Residual Strength State

(from Mitchell, 1976; after Tchalenko, 1968)

(a) Structures formed in direct shear test
(b) Principal displacement shears in direct shear test

S_1, initial fabric attitude
σ_1, σ_1^*, major principal stress directions at peak and residual strengths, respectively
P, thrust shears
R, Riedel shears
hatchings, particle orientation in compression texture
white areas, particles in initial fabric attitude

does not affect the displacement required to obtain the residual condition. However, pre-cutting the shear plane prior to shearing greatly reduces the required displacement, presumably by improving clay particle alignment.

2.4 CLEAVAGE

The residual strength of clay soils depends, in part, on the crystal cleavage mode. Although, as discussed in the previous section, slip along the basal cleavage planes is the dominant mechanism of deformation at large strains, some clay minerals have no easy slip planes and exhibit higher residual strengths. For example, attapulgite, a fibrous needle-shaped clay mineral which shears preferentially along a stepped cleavage plane, has a very high residual angle of friction (see Table 2.1). The high shear strength of attapulgite may also be related to the intermeshing of particles which occurs even at large strains (Chattopadhyay, 1972, p. 234). Kaolinite, illite and montmorillonite, with a strongly preferred basal cleavage, have low residual strengths.

With the possible exception of montmorillonite, slip does not appear to occur on cleavage planes within the mineral crystals, but between cleavage planes of adjacent particles (Chattopadhyay, 1972, p. 61).

TABLE 2.1 Cleavage and Residual Friction Angle of Some Clay Minerals

Mineral	Cleavage Mode	Bonding Along Cleavage Planes	Residual Friction Angle ϕ'_r (degrees)
Montmorillonite	Easy (001) basal cleavage	Secondary valence plus exchangeable ion link	8.5
Kaolinite	Easy (001) basal cleavage	Secondary valence plus hydrogen bonding	11
Attapulgite	Staircase like (110) cleavage	Si - O - Si weak link	30

(after Chattopadhyay, 1972)

2.5 BONDING

Based on studies of soil creep as a rate process, Mitchell, Singh and Campanella (1969) developed a hypothesis concerning the relationship between bonding, effective stress and strength of soil. Chattopadhyay (1972) reached substantially similar conclusions based on his study of the influence of pore water chemistry on the residual strength of clays.

Mitchellet al (1969) postulate that the only significant mechanism by which effective normal and shear stresses are transmitted through a soil mass is by intergranular contact. The intergranular contact is essentially solid-to-solid, although adsorbed water layers may sometimes behave as part of the crystal structure. The shearing resistance of the soil is derived from inter-atomic and inter-molecular bonding across the contact zone between adjacent particles.

Direct physical evidence of solid-to-solid interparticle contacts in clays has been obtained in the form of scanning electron microscope photographs of clay particles (Matsui, Ito, Mitchell and Abe, 1980). Microphotographs of kaolin clay particles obtained after shearing show occasional scratches on particle surfaces which are interpreted as tracks formed by interparticle friction. No such tracks were observed on the particles prior to shearing. For further evidence of solid-to-solid contacts in clays Matsui et al (1980) point to the work of Koerner, Lord and McCabe (1977), who recorded the acoustic emissions of cohesive soils during shear. It is suggested that the acoustic emissions occur as a result of solid-to-solid interparticle contacts and that such emissions should not be expected if the contacts occurred between "soft" adsorbed water layers.

The shear strength of clay is governed by the strength and concentration of bonds in the plane of deformation (Chattopadhyay, 1972). As shown by Morgenstern and Tchalenko (1967), beyond the peak strength displacement, deformation occurs by slip between mineral cleavage planes of adjacent parallel oriented mineral crystals in the two thin shear bands. The slippage involves the continuous rupture and formation of bonds in the interparticle contact areas. Between minerals with the same cleavage mode, the mineral with the lesser bond energy per unit of contact area will have the lower residual strength. The general relationship between bonding and residual strength is illustrated by the data tabulated in Table 2.2.

For normally consolidated clays, the number of bonds formed across an interparticle contact zone appears to be proportional to the effective normal force at the contact (Mitchell et al, 1969). This effect may be explained by the Terzaghi-Bowden and Tabor adhesion theory of friction which states that the interparticle contact area is proportional to the effective normal force. An increase in effective normal force results in a proportionate increase in the size of the contact zone, the number of bonds per unit of contact area remaining constant. However, in the absence of shear deformation, reducing the consolidation pressure applied to a clay does not result in a proportionate reduction in the number of bonds, thus accounting for the higher shear strength of overconsolidated clays.

Water does not appear to affect the strength of bonds, but does alter the effective (particle-to-particle) stress and, therefore, the number of bonds which form.

TABLE 2.2 Bonding Along Cleavage Planes and Residual Strength

Mineral	Mode of Cleavage	Bonding Along Cleavage Planes	Residual Friction Angle, ϕ'_r	Particle Shape
Quartz	No definite cleavage		35 degrees	Bulky
Attapulgite	Along (110) plane	Si-O-Si, weak	30 degrees	Fibrous and needle-shaped
Mica	Good basal (001)	Secondary valence (0.5 to 5 kcal/mole) + K- linkages	17 to 24 degrees	Sheet
Kaolinite	Basal (001)	Secondary valence (0.5 to 5 kcal/mole) + H bonds	12 degrees	Platy
Illite	Basal (001)	Secondary valence (0.5 to 5 kcal/mole) + K linkages	10.2 degrees	Platy
Montmorillonite	Excellent basal (001)	Secondary valence (0.5 to 5 kcal/mole) + exchangeable ion linkages	4 to 10 degrees	Platy-filmy
Talc	Basal (001)	Secondary valence (0.5 to 5 kal/mole)	6 degrees	Platy
Graphite	Basal (001)	van der Waal's	3 to 6 degrees	Sheet
MoS_2	Basal (001)	Weak interlayer	2 degrees	Sheet

From Mitchell, 1976; as adapted from Chattopadhyay, 1972.

The true effective normal force by which contacts are made and bonds are formed is composed of externally applied forces (including the self-weight of soil) and/or internally generated physico-chemical forces. In general, the physico-chemical forces are relatively small, the true effective stress is virtually equal to the apparent effective stress, and the strength of clay soils appears purely frictional in character. However, in very active clay minerals, such as sodium montmorillonite, large physico-chemical forces carry part of the normal stress. As the pore water chemistry greatly influences the magnitude of the physico-chemical forces in active clays, changes in pore water chemistry cause changes in the apparent residual angle of friction, as illustrated in Figure 2.3.

Studies by Chattopadhyay (1972, p. 326) indicate that, if the physico-chemical forces are taken into account, the behaviour of sodium montmorillonite is purely frictional, similar to the behaviour of less active clays. He concludes that every mineral has an unchanging true, or intrinsic, residual angle of friction. Electrolytic concentration may influence water content and physico-chemical stresses, thereby altering the true effective stress, but it has no effect on the strength generating mechanism or the true residual angle of friction.

In practical applications, however, the apparent residual angle of friction measured is the macroscopic or mass soil property, no allowance being made for the influence of physico-chemical forces.

A detailed theoretical treatment of this microscopic model of shear strength in clay soils is provided by Matsui et al (1980), including consideration of the intrinsic or true angle of friction and

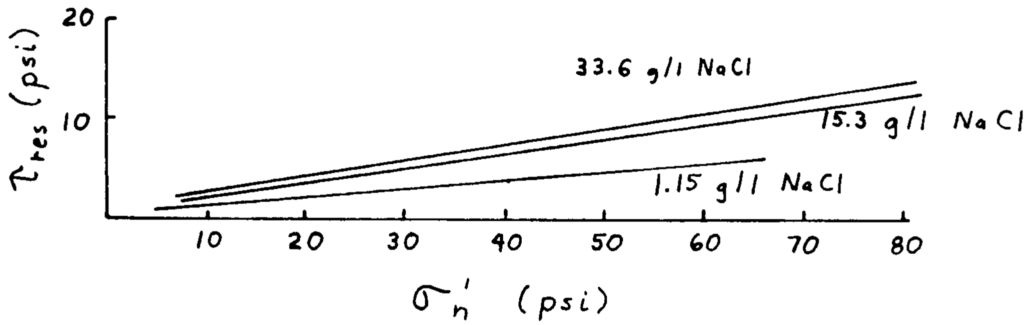


FIGURE 2.3 Residual Shear Strength Versus Apparent Effective Stress for Sodium Montmorillonite at Various Pore Fluid Salt (NaCl) Concentrations

(after Chattopadhyay, 1972)

the effects of physico-chemical forces, dilatency, cementation, suction and stress history.

2.6 STRESS DEPENDENT BEHAVIOUR

Studies have shown that, for many clays, the apparent residual friction angle is greater at normal stresses less than 30 psi than at higher stresses, as shown in Figure 2.4. As recorded by Mitchell (1976, p.315), two possible explanations for this curvature of the residual strength envelope in the low normal stress region are as follows.

1. The work required to attain a perfect particle alignment in the failure plane under low normal stresses exceeds the work required to shear the soil without such alignment. Therefore, the particles are less than perfectly aligned under low normal stresses and the shear strength is correspondingly increased. Under higher normal stresses, less work is required if the particles are perfectly aligned and the lower residual strength results.

2. Under low normal stresses the particle contact areas behave elastically. By elastic junction theory, it can be shown that the real contact area increases proportionally with $(\sigma'_n)^{2/3}$ at the elastic contact. The shearing resistance, $\tan \phi'_r = \tau/\sigma'$, must therefore vary as $(\sigma'_n)^{-1/3}$. At higher normal stresses the particle contact areas behave plastically and the solid-to-solid contact areas are directly proportional

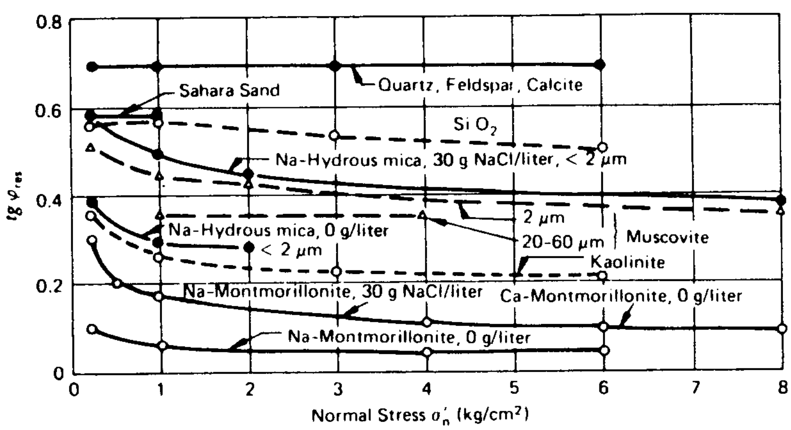


FIGURE 2.4 Relationship Between Shearing Resistance and Normal Stress for Different Minerals

(From Mitchell, 1976; after Kenny, 1967) to the effective normal force. This concept is supported by the plot of tan ϕ'_r versus $(\sigma'_n)^{-1/3}$ for various clays shown in Figure 2.5.

Mitchell suggests that either of the preceding hypotheses is plausible and that further research will be required to fully determine the cause of the stress dependent behaviour of frictional resistance.

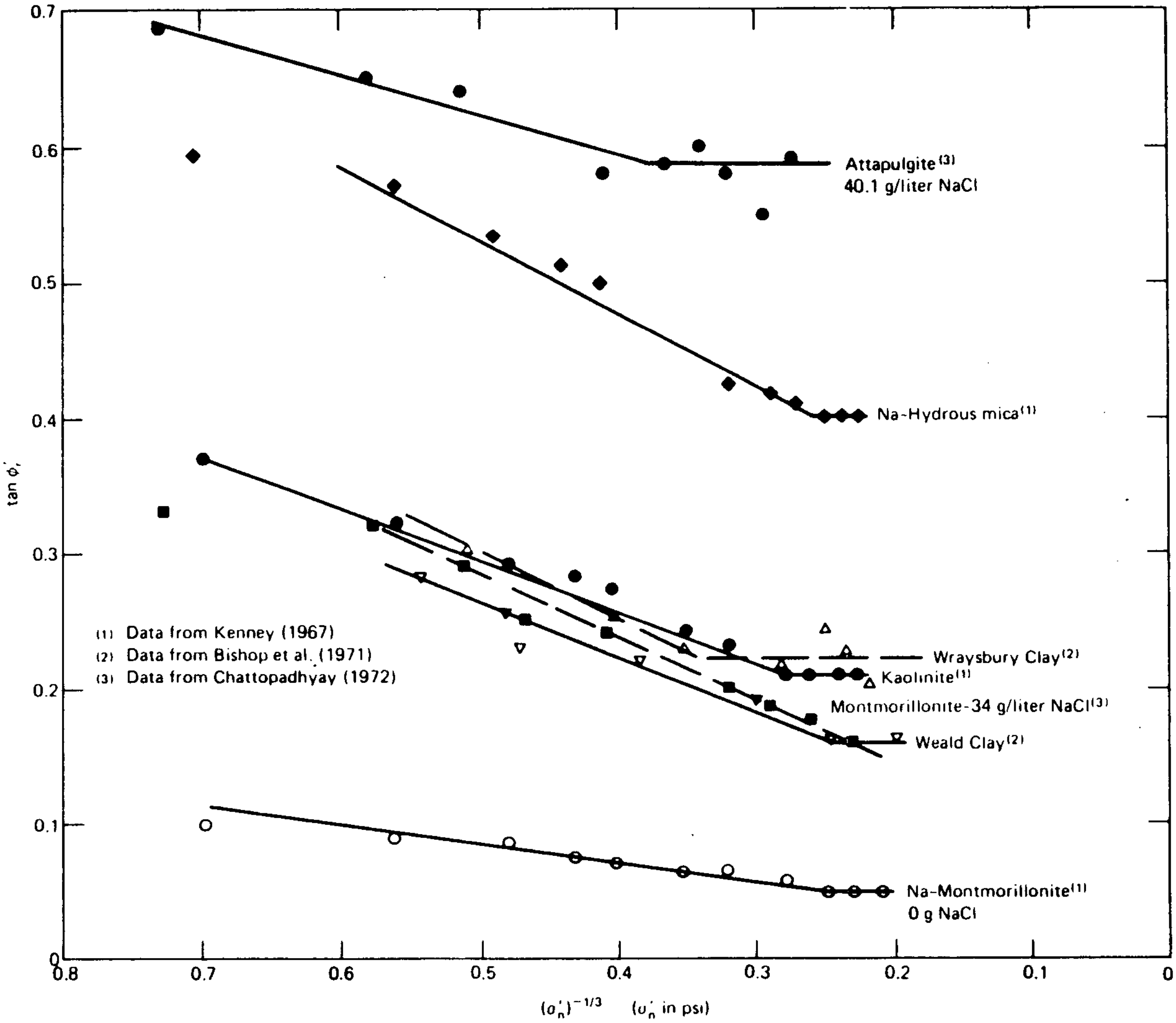


FIGURE 2.5 Shearing Resistance Versus Normal Effective Stress on the Shear Plane Raised to the Minus 1/3 Power

(From Mitchell, 1976)

CHAPTER III

RESIDUAL STRENGTH TESTING METHODS

3.1 INTRODUCTION

The major methods which have been used for residual strength determination are the direct shear, triaxial and ring shear tests. The direct shear and triaxial apparatus, originally developed for the study of peak strength behaviour, are small displacement devices that have been adapted for use in determining residual strength. The ring shear apparatus was designed specifically for residual strength testing.

Many of the features desirable in a residual strength device are also generally required for any drained shear apparatus. For efficiency, the soil specimens should be simple to prepare and the test device should be easy to operate with a minimum of supervision. The device should be versatile, offering a wide selection of applied strain rates and normal loads. To facilitate pore pressure dissipation in low permeable soils (fully drained conditions), the ability to test thin samples at very low strain rates would be necessary. Uniform stress and strain conditions along the shear plane are desirable for reliable interpretation of results.

The unique design requirement of a residual strength device is the large shear displacement necessary to ensure that the shear plane is fully developed and the residual condition has been achieved. The shear displacement should be uni-directional to model field conditions and to minimize disturbances on the shear plane which might increase the shearing resistance of the soil.

3.2 DIRECT SHEAR TEST

The direct shear test is the most common method used for determining residual strength. Little or no modification of the equipment is required. Due to the short length of travel of the shear box, large strains are accumulated either by repeated reversals of the direction of travel or by repeated uni-directional shears, repositioning the shear box at the end of each travel. Some researchers cut the shear plane prior to shearing in an attempt to accelerate development of the residual condition.

The major advantages of the common shear box method applied to residual strength testing would appear to be the ready availability of suitable equipment and the simple method of operation. Disadvantages include the inconvenience imposed by repeated repositionings or reversals of the shear box, the possible influence on results of fluctuating stress conditions in the sample, and difficulties encountered in interpreting the test results. The stress fluctuations occur in the sample during testing due to the changing cross-sectional area of the shear plane and due to the alternating direction of applied shear stress when the direction of travel is reversed. Considering that the compression textures within the shear zone are oriented relative to the major principal stress, as described in Chapter II, and that a major change in the direction of the major principal stress occurs during a reversal of travel, shear box reversals would be expected to cause some physical disruption of the shear plane, possibly delaying the development of residual conditions. The difficulties in interpreting test results appear to arise from curvature or inclination of the shear plane, as encountered by Hermann and Wolfskill (1966, p. 118) in direct shear tests on weak clay shales, or from premature termination of testing. Based on a review of published data, La Gatta (1970) suggests that termination of testing before the residual condition has been adequately established is a common failing of residual strength studies employing the direct shear test method. Considerations which can result in early termination of testing include excessive sample loss through the loading frame gap or time limitations. Some of the problems encountered in direct shear testing for residual strength during the course of this thesis study are discussed in detail in Chapter V.

3.3 TRIAXIAL TEST

The triaxial test is more difficult to adapt for residual testing than the direct shear and, for this reason, has not been widely used. The major difficulty encountered in triaxial testing for residual strength is the inability to obtain large shear strains without severely distorting the sample. Other disadvantages include the corrections which must be made for the constraining effect of the rubber membrane surrounding the sample, for the changing cross-sectional area of the failure plane, and for the horizontal component of the ram load if the loading cap is incapable of lateral movement.

In triaxial testing for residual strength, because only limited displacements can be achieved, methods must be employed to promote shear plane development at small strains. Some success is claimed for triaxial tests on samples with pre-cut shear planes or pre-existing fissures oriented at an angle of $45 + \phi_r'/2$ to the horizontal, where ϕ_r' is the estimated angle of residual friction (Chandler, 1966; Petley, 1966; Webb, 1969). Petley (1966) found that the displacement to the residual condition could be significantly reduced by smoothing the pre-cut shear surfaces with a piece of glass drawn over the surfaces in the direction of shear. However, complicated test procedures and the uncertainty of residual strength determinations obtained at small shear displacements make the triaxial test unsuited for normal practical use in this application.

3.4 RING SHEAR TEST

The ring shear test was developed specifically for the purpose of determining residual strength. The test is performed on an annular disc-shaped specimen with the normal load applied in the axial direction and the shear load applied tangentially. The soil is sheared in a plane perpendicular to the axis by rotating the top of the specimen with respect to the base.

The ring shear test has the facility for unlimited uni-directional strains (all strains are tangential in this torsional test) with a constant soil cross-section. The most serious disadvantage of the test, other than the relatively complicated construction of the apparatus, would appear to be the variation in strain rate across the shear plane which can affect pore pressure dissipation. However, the variation can be minimized by choosing appropriate sample dimensions and tests indicate that residual shear strength is insensitive to small changes in strain rate (Townsend and Gilbert, 1974, p. 12).

A variant form of the ring shear test is carried out on solid disc-shaped specimens. Despite theoretical objections to the large variation in strain rate across the sample which exists in this type of test, some reasonable results have been achieved using this method (La Gatta, 1970).

Ring shear devices were initially developed in the early 1930's by a number of independent researchers including Tiedemann, Gruner and Haefeli, Cooling and Smith, and Hvorslev (Hvorslev, 1939). The more successful devices incorporated upper and lower pairs of confining rings positioned to cause failure to occur within the sample body away from the end platens. All of the early devices were stress-controlled. In 1947, Hvorslev directed the development of a ring shear machine that could be operated in either a stress-controlled or strain-controlled mode (La Gatta, 1970, p. 10). With the exception of this work by Hvorslev, the evolution of the ring shear apparatus virtually ceased until interest in residual strength was rekindled in the 1960's.

Of several devices developed since the early 1960's, the more significant in terms of design advancement and innovation are the device developed at Harvard University (La Gatta, 1970) and the device developed jointly by the Imperial College and the Norwegian Geotechnical Institute (Bishop et al, 1971). The I.C.-N.G.I. machine tests thick (3/4 inch) annular specimens using split confining rings. Sample loss through squeezing between the confining rings is regulated by a precise gap control mechanism. The Harvard machine can test annular or solid disc-shaped specimens with either solid or split confining rings. The sample height is variable from 1 to 25 mm, but thin samples (2 to 3 mm thick) are normally used. Both the Harvard and the I.C.-N.G.I. devices are strain-controlled.

Many of the early devices are described in detail by Hvorslev (1939). The advantages and disadvantages of most ring shear devices constructed prior to 1971 are reviewed by Bishop et al (1971).

CHAPTER IV

THE UBC RING SHEAR DEVICE

4.1 INTRODUCTION

The UBC ring shear machine results from the author's thesis project to design and develop a practical residual strength apparatus. After considering various options and alternatives, the ring shear configuration was accepted as the most advantageous approach to residual strength determination. The preliminary design was begun in May 1976, and construction was completed in January of the following year.

The UBC ring shear device operates on the same basic principles as early devices such as that designed by Hvorslev (1939) and more modern devices such as that developed jointly by the Norwegian Geotechnical Institute and the Imperial College (Bishop et al, 1971). However, it is hoped that the emphasis on operational simplicity and practicality in the UBC design will prove a useful contribution to the evolution of residual strength test devices.

4.2 GENERAL DESCRIPTION

A photograph of the UBC ring shear device is provided in Figure 4.1. A schematic section of the device identifying major components is presented in Figure 4.2. As shown in the section, the annular soil sample is contained at the sides between upper and lower pairs of confining rings and at the top and bottom between porous stainless steel platens. The lower confining rings and porous platen are fastened to a turntable which is rotated at a chosen rate. The upper confining rings and porous

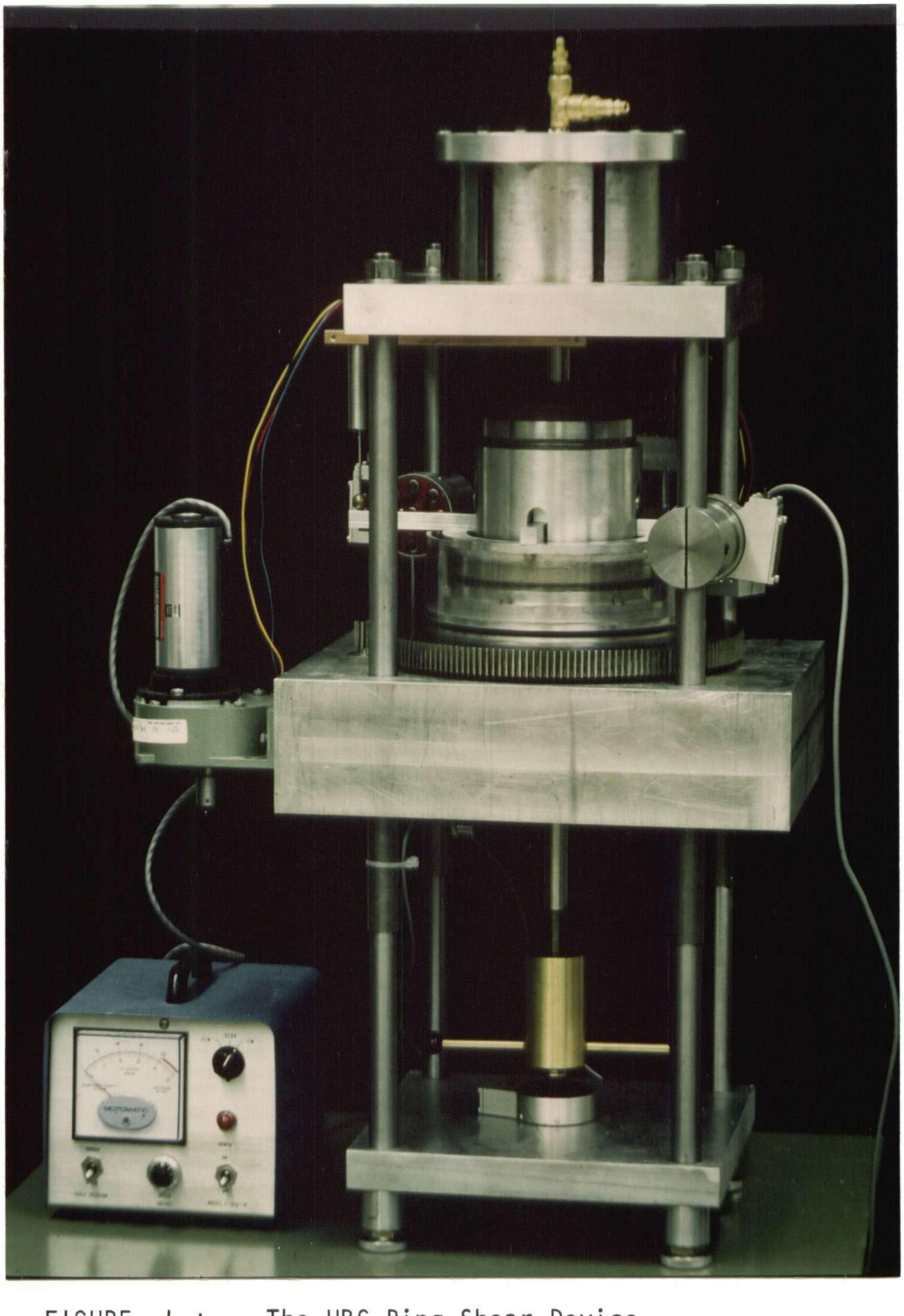
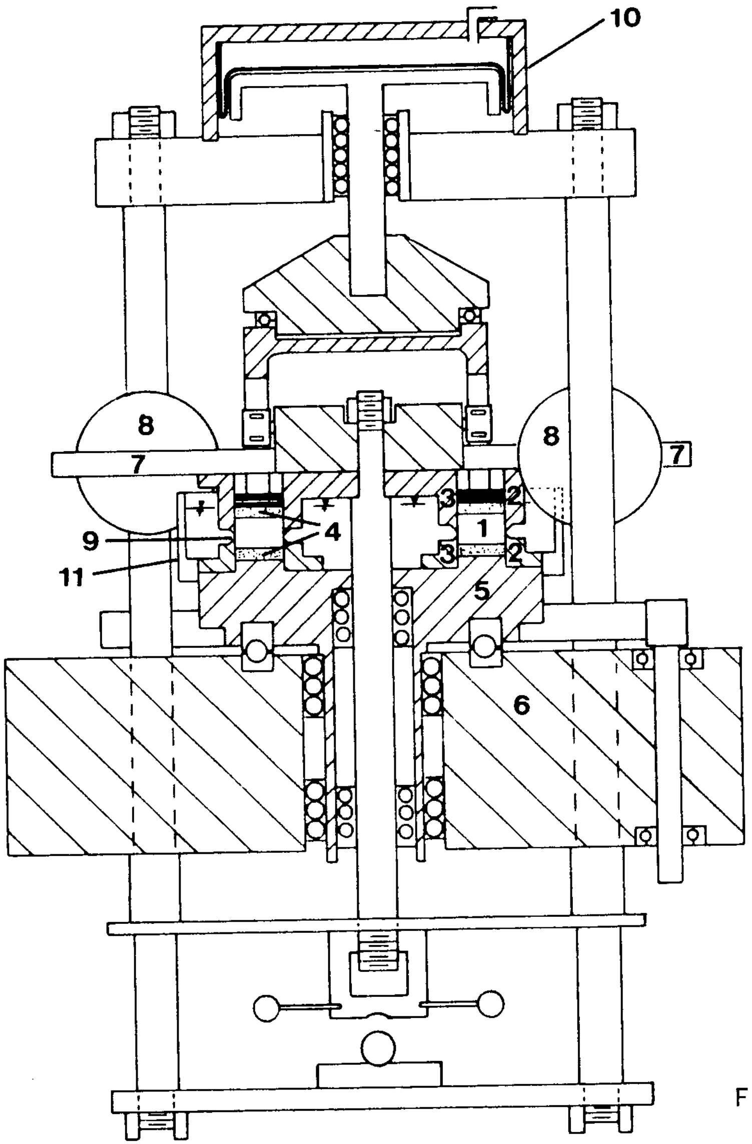


FIGURE 4.1 The UBC Ring Shear Device



MAJOR COMPONENTS

1 Annular Soil Sample

2 Upper and lower outside confining rings

3 Upper and lower inside confining rings

4 Porous stainless steel platens

5 Turntable

6 Turntable base

7 Moment transfer arms

8 Moment-measuring force transducers

9 Confining ring gap

10 Air piston

11 Water Reservoir

FIGURE 4.2 Major Components of UBC Ring Shear Device platen are restrained from rotating by moment-transfer arms which abut against two moment-measuring force transducers. The soil sample is thereby compelled to shear in the horizontal plane intersecting a small gap maintained between the upper and lower confining rings. The normal load, which is applied through the upper porous platen, is derived from and regulated by air pressure supplied to an air piston. Sample drainage is provided through the porous platens which are connected to a water reservoir encompassing the confining rings.

The major machine parameters are as follows:

Soil Sample:	Outside diameter	5.5 inches
	Inside diameter	3.5 inches
	Initial height	Variable up to 0.75 inches
Basal Drainage Path:	Variable up to 0.25 inches	
Normal Stress:	Variable up to 200 psi with 100 psi house line air pressure	

Rate of Shear (at center of shear plane): Smoothly variable

1.5×10^{-1} to 1.5×10^{-4} inches/minute or

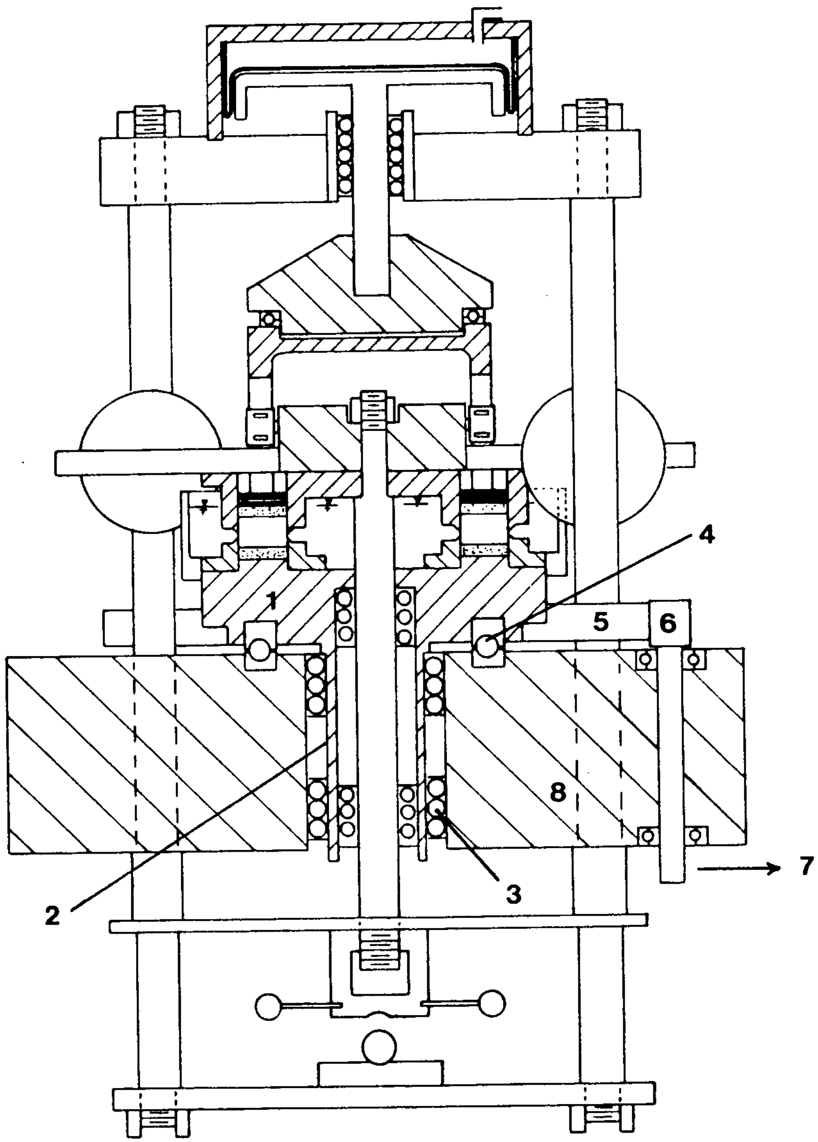
6.1×10^{-3} to 6.1×10^{-6} inches/minute

with an optional gearbox

4.3 DETAILED DESCRIPTION

4.3.1 The Drive System

Components of the turntable drive system, other than the motor and an optional gearbox, are identified in a schematic section of the UBC ring shear device in Figure 4.3. The turntable is driven by a Motomatic E-550 MGHD d.c. servo motor with a heavy duty gearhead (manufactured by Electro-Craft Corporation, Hopkins, Minnesota).



DRIVE SYSTEM

1 Turntable

2 Turntable alignment shaft

3 Rotary bearings providing lateral alignment

4 Thrust bearings providing vertical support

5 Spur gear

6 Pinion gear

7 Chain drive to motor

Other Components

8 Turntable base

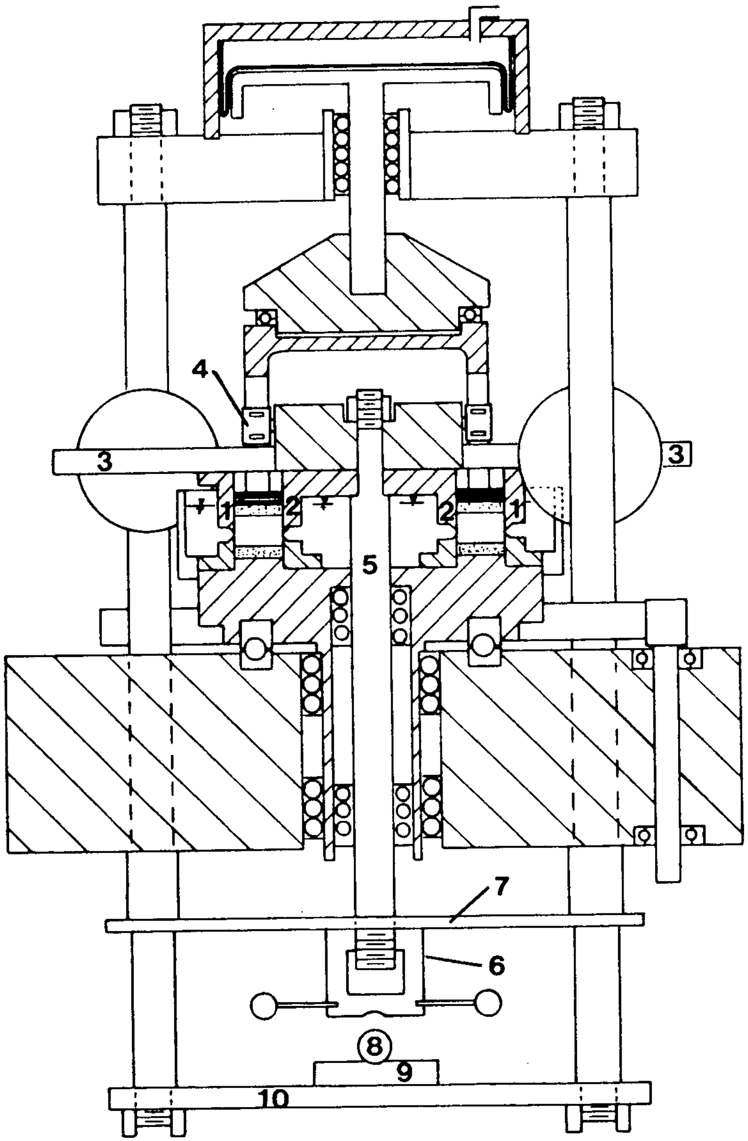
FIGURE 4.3 Drive System

With a master control unit, the motor-gearhead combination provides a smoothly variable output from 0.001 to 1 rpm. The motor-gearhead is connected by a chain drive to the pinion which directly drives the large spur gear of the turntable. When further speed reductions are desired, a small optional gearbox is used to reduce the output of the motor-gearhead. The turntable is supported vertically by a single large thrust bearing. Lateral alignment is provided along the hollow turntable alignment shaft by two rotary bearings set in the turntable base.

As previously mentioned, the drive system provides a smoothly variable rate of shear at the center of the sample from 1.5×10^{-1} to 1.5×10^{-4} inches/minute, or from 6.1×10^{-3} to 6.1×10^{-6} inches/minute with the optional gearbox. A performance test of the drive system revealed small, gentle fluctuations in turntable strain rate of $\pm 2.5\%$. As the period of tooth engagement equals the period of the strain rate fluctuations, the behaviour is attributed to slight imperfections in the tooth design of the spur and pinion gears. The magnitude of the fluctuations, however, is considered too small to influence residual strength measurements.

4.3.2 The Upper Ring Assembly

A schematic section of the UBC ring shear device identifying components of the upper ring assembly and suspension system is presented in Figure 4.4. The upper ring assembly consists of both the inside and outside upper confining rings and the moment-transfer arms to which they are attached. The assembly is supported and aligned by the



UPPER RING ASSEMBLY

1 Outside upper confining ring

2 Inside upper confining ring

3 Confining ring support / moment transfer arms

4 Roller bearings

SUSPENSION SYSTEM

5 Center support shaft

6 Threaded cylinder for adjusting length of shaft and width of confining ring gap

7 Removable metal bar for closing confining ring gap

8 Loading ball

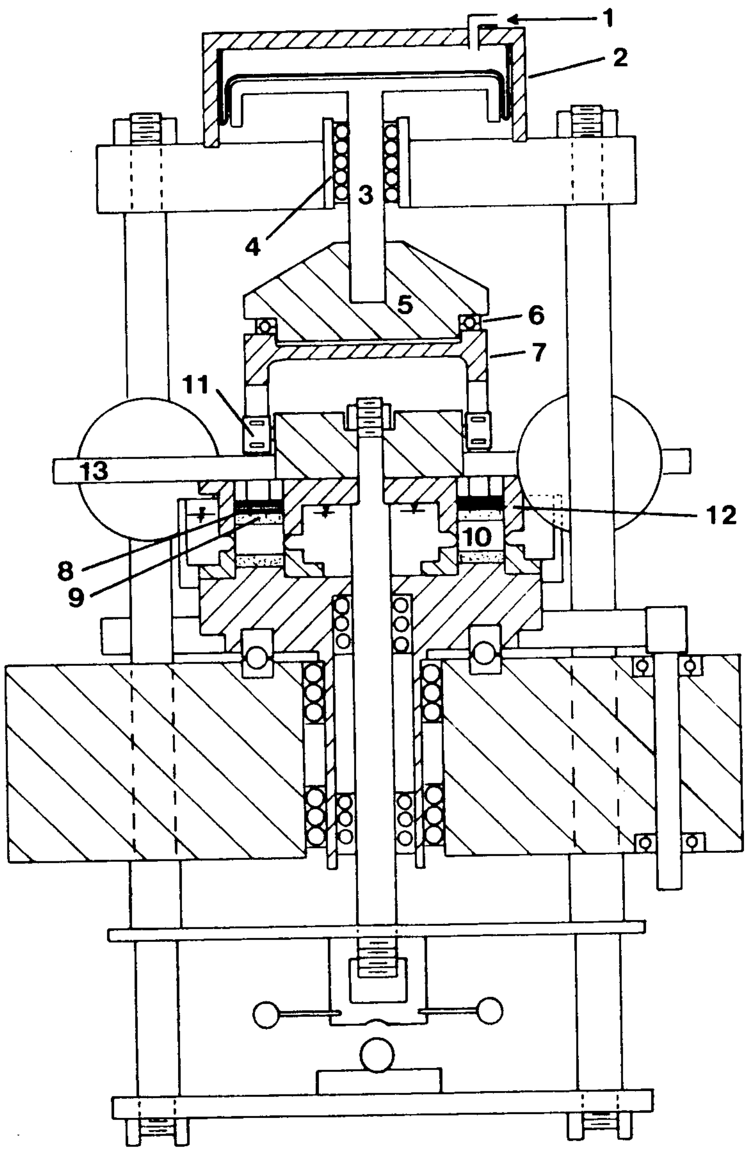
9 Load cell to measure vertical (side) friction on upper confining rings

10 Bottom plate

FIGURE 4.4 Upper Ring Assembly and Suspension System center shaft . A threaded cylinder at the base of the center shaft is used to shorten or lengthen the shaft, thereby closing or opening the gap between the upper and lower confining rings. During sample consolidation the threaded cylinder is tightened against a removable metal bar, tensioning the center shaft and firmly closing the confining ring gap. This configuration is shown in Figure 4.4. During testing the threaded cylinder is tightened against the loading ball of the load cell resting on the bottom plate of the ring shear machine until the confining ring gap is jacked open the desired amount. The gap width during testing is typically set at 0.002 to 0.004 inches. The load cell on the bottom plate measures side friction of the soil sample and the upper porous platen on the upper confining rings plus the weight of the upper ring assembly.

4.3.3 The Loading System

A schematic section of the UBC ring shear device identifying components of the loading system is provided in Figure 4.5. The normal load is derived from air pressure supplied to a Bellofram rolling-diaphragm air piston. The piston shaft, aligned by a linear ball bushing, transfers the load through a conical adapter, a loading cylinder and the upper porous platen to the soil sample. The loading cylinder is slotted at 90 degree intervals to accommodate four support arms for the outside upper confining ring. Two of the confining ring support arms, shown in Figure 4.5, double as moment-transfer arms. The only contact between the support arms and the loading cylinder is through two roller bearings which allow frictionless differential movement



LOADING SYSTEM

1 Air supply

2 Rolling diaphragm air piston

3 Piston shaft

4 Linear ball bushing

5 Conical adapter

6 Thrust bearing

7 Loading cylinder

8 Loading plate - a stiff backing for porous platen

9 Upper stainless steel porous platen

10 Annular soil sample

Other Components

11 Roller bearings

12 Outside upper confining ring

13 Confining ring support / moment transfer arms

FIGURE 4.5 Loading System

between the components.

Each component of the loading system from the air piston to the upper porous platen is aligned by the preceding component to produce a rigid, non-tilting unit. The purpose of the careful alignment is to prevent tilting of the upper porous platen in the event of uneven soil compression. The non-tilting platen reduces the tolerance required between the upper platen and confining rings to allow for smooth differential movement of components. If the sample compresses unevenly, the non-tilting platen will cause a non-uniform stress distribution in the sample. In such a case, the highly remolded soil in the failure zone will tend to flow plastically under the stress gradient until near-uniform stress conditions are re-established.

4.3.4 Measurement of Normal Loads

The normal load on the failure plane is calculated by subtracting the side friction on the upper confining rings from the total applied normal force. The total applied normal force is a pre-determined function of the air pressure supplied to the air piston as measured by a pressure transducer. The side friction is calculated by subtracting the weight of the upper ring assembly and suspension system from the force on the load cell at the base of the center shaft.

4.3.5 Measurement of Shear Forces

Shear forces developed across the failure plane during testing are transferred through the upper confining rings and loading platen to the moment-transfer arms. Shear forces transferred to the upper confining rings are passed directly to the moment-transfer arms to which the rings are attached. Shear forces transferred to the loading platen are passed through the loading cylinder to roller bearings attached to the moment-transfer arms. A thrust bearing between the conical adapter and the loading cylinder prevents the transfer of moments to the loading piston. A pair of force transducers measure the moment carried by the moment transfer arms.

During testing, the loads on the moment-measuring force transducers are maintained in balance, equal in magnitude but opposite in direction. In this configuration, the forces form a near-perfect force couple, reducing side-loads on the center shaft to a minimum. Minimizing the shaft side-load helps maintain alignments and reduces loads on bearings and bushings. The loads on the force transducers are maintained in balance by occasional minor adjustments to the screw-mounted loading tips of the transducers. The adjustments do not interrupt testing or the collection of data.

The residual friction angle is calculated from the moment and normal load on the shear plane, both measured quantities, by the following equation, derived in Appendix I:

$$\tan \phi_r' = \frac{3M\,(r_1 + r_2)}{2W\,(r_1^2 + r_1 r_2 + r_2^2)}$$

where M = sum of moments on shear plane

W = total normal load on shear plane

r_1 = inside radius of soil specimen

r_2 = outside radius of soil specimen

A study by the I. C. - N.G.I. group (Bishop et al, 1971) has shown that the above equation will not be in serious error for any reasonable distribution of shear stresses in the sample.

4.3.6 Displacement Measurements

Displacements are measured using linear variable displacement transformers (LVDT'S). Angular displacements of the turntable are converted to vertical displacements using a spur gear and screw arrangement, the resulting vertical displacements being measured by an LVDT. Consolidation, sample loss and the width of the confining ring gap are directly measured by a single LVDT suspended between the shaft of the air piston and the moment transfer arm. These vertical displacements can be identified by the character and timing of the motion. Sudden changes in vertical displacement result from adjustments made to the confining ring gap. Very gradual changes in vertical displacement typically occur from the gradual loss of soil through the confining ring gap. Vertical displacements that are neither sudden nor very gradual, and which follow a change in confining pressure, are the result of sample consolidation or swell.

4.4 SAMPLE PLACEMENT

Undisturbed samples are prepared by cutting a maximum 3/4 inch thick slab of soil and trimming it to the approximate dimensions of the annular sample desired. The final trimming is done with a cutting unit that is pushed into the soil. The cutting unit, shown in Figure 4.6, consists of the upper confining rings, moment-transfer arms, loading cylinder and upper porous platen. The confining rings

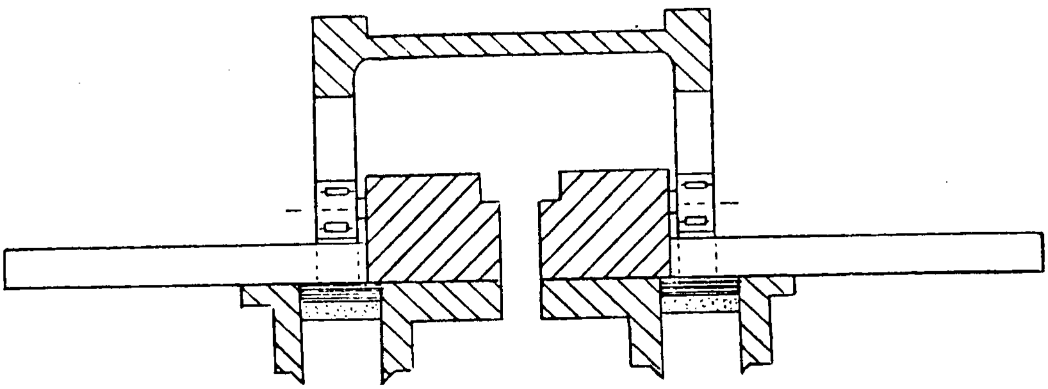


FIGURE 4.6 Sample Cutting Unit

This unit, composed of the upper confining rings, moment-transfer arms and loading cylinder, is used for final trimming of undisturbed samples or as a mold for remolded samples. The unit, complete with soil sample, is then mounted into the ring shear device for testing.

have sharp tips and behave well as cutters provided the sample has been carefully trimmed to prevent excessive build-up of soil at the cutting tips. After excess soil is removed, the cutting unit is mounted in the ring shear machine and the confining ring gap is firmly closed. The soil sample is then gently pushed into position in the lower confining rings by applying a small downward force through the air piston loading system.

Remolded samples are placed in a similar manner except that the cutting unit is used as a mold into which the remolded material is pushed or spread.

With minor adjustments, such as placing a spacer beneath the lower porous platen, the minimum drainage path from the shear plane to the lower platen can be shortened from a maximum of 1/4 inch to any practical minimum. Shortened drainage paths are preferred for clays with low permeability to induce a more rapid dissipation of pore water pressures in the shear plane region.

For samples which contain sand sized particles, the shear plane to platen separation should be sufficient to prevent sand grains lodged against the porous platen from interfering with natural shear plane development.

The total sample height can also be varied from the maximum height of 3/4 inch to any practical minimum. Thick samples are usually preferred for clays and clayey silts with relatively high permeabilities because the greater sample mass allows continued testing for periods of weeks or months despite minor sample losses by squeezing through the confining ring gap. Thick samples are also required when porous platens with ribs are used. Although tests indicate that friction of the soil against the platens and confining rings is normally sufficient to ensure shearing will occur in the body of the sample, ribbed porous platens are used when soil slippage adjacent to the platens is a concern. The ribbed platens contain 16 equally-spaced ribs which protrude 1/16 inch into the sample over its entire width.

4.5 DATA COLLECTION

The UBC ring shear machine is designed for use with an automatic electronic data collection system and can be easily adapted for computerized data reduction. Although detailed supervision is not required, it has proved prudent to check the various electronic and mechanical systems every day or two to ensure smooth operation.

CHAPTER V

TESTING PROGRAM

5.1 INTRODUCTION

A testing program was adopted early in this thesis study to provide a means of determining appropriate testing techniques and procedures and a means of evaluating the UBC Ring Shear Device. The program began in July 1976 with a series of 5 single stage multi-reversal direct shear tests undertaken to provide data for comparison and possible correlation with ring shear test results. These tests, which varied from 0.5 to 3 months in duration, were completed in the course of one year. Due to the need for detailed test supervision, the direct shear equipment was operated only part-time and cumulative shear displacements did not exceed 6 inches for any of the 5 tests.

A description of the direct shear apparatus used in this study is provided in Appendix II.

Ring shear testing began in mid-January 1977 following completion of the new device. A series of 11 ring shear tests were performed over an 8 month period. The series was comprised of 2 tests for machine performance and 9 tests for residual strength, including several multi-stage tests in which residual strength determinations were made at 2 or more normal loads. In view of the experimental nature of the apparatus, all test results, whether successful or inconclusive, are presented and discussed in this chapter.

Three soils having widely varying properties were included in the test program. Two of the soils, Haney Clay and Hat Creek Clay, were tested in both the ring shear and direct shear apparatus. The third soil, Iranian Clay, was tested in the ring shear apparatus only.

This chapter presents the results of residual strength testing on these three soils. To provide an orderly presentation of data, tests performed on each type of soil are discussed in separate sections. In each case, the discussion of test results is preceded by a detailed description of the soil tested. Plots and tables of test results are also presented. The ring shear and direct shear testing programs are summarized in Tables 5.1 and 5.2, respectively. The tests in each program are numbered according to the chronological order of testing.

Prior to the presentation and discussion of residual strength test results, there is a discussion of machine performance tests carried out early in the ring shear testing program.

5.2 RING SHEAR PERFORMANCE TESTS

Two ring shear tests, RS 1 (Ring Shear Test 1) and RS 3, were conducted to investigate machine performance only. RS 1, the initial test following construction of the new device, established that the mechanical systems were functioning as planned. The test also provided an opportunity to investigate details of sample preparation and the practical techniques of testing. However, the electronic measurement systems were not fully installed at the time of testing and no useful soil strength data was recorded.

TABLE 5.1 Schedule of Ring Shear Tests

Test	Soil	Duration (days) Consolidation	Shearing	End of Test	No. of Stages	Total Displacement (inches)	Comments
RS 1	H.C.	3	1	1/18/77	1	-	Machine Performance Test
RS 2	H.C.	-	22	2/12/77	5	23.5	Pre-cut failure plane
RS 3	H.C.	-	8	3/18/77	1	12.7	Machine Performance Test Soil marked with modelling clay wedges
RS 4	H.C.	14	7	4/11/77	1	11.1	Test ended due to leaking O-ring seal
RS 5	H.C.C.	1	5	5/15/77	1	6.8	Test ended due to high side friction on confining rings
RS 6	H.C.C.	1	20	6/7/77	1	44.0	
RS 7	H.C.C.	2	7	6/16/77	1	24.8	
RS 8	H.C.	1	28	7/16/77	10	28.1	
RS 9	I.C.	1	4	7/22/77	1	3.1	Test ended due to high side friction
RS 10	I.C.	-	27	8/18/77	2	20.1	Test ended due to faulty air regulator
RS 11	I.C.	-	14	9/18/77	1	50.7	

H.C. - Haney Clay
H.C.C. - Hat Creek Clay
I.C. - Iranian Clay

TABLE 5.2 Schedule of Direct Shear Tests

Test	Soil	Test Duration (days)	End of Test	No. of Stages	Total Displacement (inches)	Comments
DS 1	H.C.C.	17	8/14/76	1	3.3	
DS 2	H.C.C.	55	11/9/76	1	5.4	
DS 3	H.C.	87	2/28/77	1	5.3	
DS 4	H.C.C.	79	5/20/77	1	3.1	Remolded, sieved and reconsolidated sample
DS 5	H.C.	50	7/19/77	1	5.3	Pre-cut failure plane

H.C.C. - Hat Creek Clay

H.C. - Haney Clay

The soil used in this first test was remolded Haney Clay. Following a short 3 day test period, the sample was removed and examined. As expected, a well-developed shear plane was located within the body of the sample as defined by the confining ring gap. As a result of the successful operation and performance of the equipment in RS1, residual strength testing was begun immediately thereafter.

RS3 was an investigation to determine if any sample slippage was occurring adjacent to the porous platens. A sample of undisturbed Haney Clay marked by 2 thin slices of coloured oil-base modelling clay was tested. The initial position of the modelling clay wedges was clearly identified on both upper and lower outside confining rings prior to testing. The soil was sheared to a total displacement of about 12.5 inches. The normal load was maintained at about 75 psi, except for a 16 hour period during which the house line air pressure supplying the loading piston was lost. During that period the normal load dropped to about 2 psi for a displacement of 2.6 inches. The test results are plotted in Figure 5.1.

At the end of the test it was observed that no slippage had occurred in the upper confining rings, but that a limited slippage of about 0.5 inches had occurred in the lower rings. This slippage is believed to have resulted from the distribution of effective normal stresses within the sample following the drop in applied normal load.

Due to the low permeability of Haney Clay, the sudden reduction in applied normal load created negative pore water pressures within the body of the sample approximately equal in magnitude to the

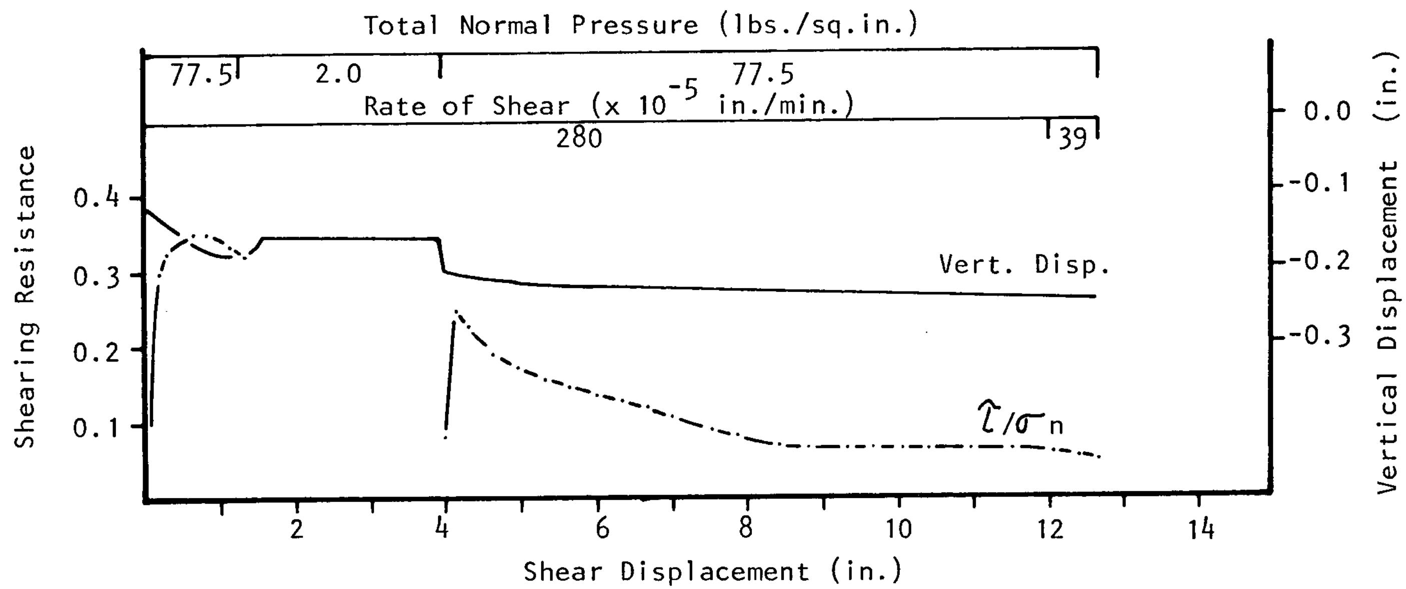


FIGURE 5.1 Shearing Resistance vs. Displacement - RS 3

Specimen: Haney Clay marked with wedges of modelling clay.

drop in normal stress. Therefore, the effective stress within the body of the sample remained virtually unchanged, falling only gradually as the excess negative pore pressures dissipated. However, due to the high permeability of the platens, the negative pore pressures at the soil-to-platen contact dissipated almost instantly resulting in a sudden drop in effective normal stress. Hence, the sample was compelled to shear or 'slip' at the platen-to-soil interface where the effective normal stress and the related shearing resistance were very low. The slippage lasted only until the negative pore pressures in the body of the sample dissipated sufficiently to reverse the balance of shearing resistance between the platen and the established failure plane within the sample. Slippage naturally occurred in the lower confining rings because the upper confining rings are deeper with a correspondingly greater contact area and frictional resistance to slippage.

Soil slippage adjacent to the porous platens is not considered a serious problem with the UBC Ring Shear Device. However, as mentioned in a previous section, porous platens fitted with metal ribs to prevent soil slippage are available for use should there be any doubt.

A plot of the shearing resistance versus cumulative shear displacement for RS3 is presented in Figure 5.1. The residual shearing resistance of about 0.05, indicated on the plot, corresponds to a residual friction angle of 3 degrees. However, a systematic experimental error, described in the discussion of test results for Haney Clay, effectively increased the measured normal stress by a constant value. The values of residual shearing resistance and friction angle, as corrected for this systematic error, are 0.18 and 10.1 degrees, respectively.

This low friction angle is apparently that of the modelling clay used to mark the position of the sample.

An examination of the sample after removal from the ring shear apparatus revealed the shear plane was completely coated with a thin translucent layer of modelling clay. This result may be indicative of how thin seams of weak material can significantly affect the strength behaviour of more competent soils.

5.3 RESIDUAL STRENGTH TESTS ON HANEY CLAY

5.3.1 Soil Description

Haney Clay is a dark-grey, sensitive marine-deposited silty clay. The undisturbed clay has a flocculated structure believed to result from the interaction of the soil particles with the salt solutions of its marine depositional environment. However, this flocculated structure is metastable because the clay has been subsequently uplifted above sea level and leached of salt by fresh water infiltration (Vaid and Campanella, 1977). Due to the reduced salt concentration, the clay particles take up a more stable dispersed structure upon remolding. The change from a flocculated to a dispersed structure is accompanied by a significant loss in soil strength, as evidenced by the sensitivity of Haney Clay (Campanella and Gupta, 1969). For detailed information regarding the structure and behaviour of sensitive clays, the reader is referred to Mitchell and Houston (1969) and Houston and Mitchell (1969).

The soil was block sampled from a deposit near the town of Haney, British Columbia, some 20 miles east of Vancouver. Due to the uniformity and availability of the material, Haney Clay has been studied in numerous experimental works conducted at the University of British Columbia over a period in excess of 15 years. Typical characteristics and properties for Haney Clay are as follows (Byrne and Aoki, 1969; Bosdet, Lum and Negussey, 1976):

Specific gravity	2.8
Liquid limit	44%
Plasticity index	18%
Natural water content	41%
Percent finer than 2 microns	46
Unconfined compressive strength	1550 psf
Sensitivity	12
Maximum past pressure	7000 psf
Permeability	2×10^{-8} cm/s

5.3.2. Ring Shear Tests

TEST PROCEDURES AND RESULTS

Three ring shear tests for the purpose of determining residual strength were performed on Haney Clay: RS 2, RS 4, and RS 8. In the case of RS 2, the shear plane was cut prior to shearing. RS 2 and RS 8 were multi-stage tests in which residual strength determinations were obtained at 4 and 10 different normal loads, respectively. RS 4 was terminated prematurely following the first stage of loading when a faulty o-ring seal caused water to leak from the reservoir into the center column of the ring shear apparatus.

The test data is presented on plots of shearing resistance and vertical displacement versus cumulative shear displacement in Figures 5.2, 5.3 and 5.4. Vertical displacement data was not recorded during RS 2, hence, this plot is omitted in Figure 5.2.

The tabulated results of each stage of testing are presented in Tables 5.3 and 5.4. The residual strength results are also summarized on a Mohr plot in Figure 5.5.

DISCUSSION AND INTERPRETATION OF RESULTS

The peak strength behaviour of Haney Clay during ring shear testing was markedly different in RS 2 than in RS 4 and RS 8, as shown in Figures 5.2 to 5.4. RS 2 reached a peak shearing resistance of less than 0.5 at a displacement of 0.03 inches, corresponding to about 4 percent strain at the center of the sample. In contrast, RS 4 and RS 8 reached a peak shearing resistance of about 0.54 at a displacement and strain of about 0.4 inches and 50 percent, respectively. The reduced peak strength and peak strength displacement in RS 2 is attributed to the effect of pre-cutting the shear plane before testing.

Beyond the peak strength displacement, the measured or apparent shear strength dropped temporarily below the final recorded residual strength in RS 2 and RS 8, and to a lesser degree in RS 4. Because the residual strength is the lowest drained strength of a soil, it is evident that the soil was not fully drained during this period and that excess pore pressures were influencing shear strength measurements.

The development of excess pore pressures during the initial stages of testing is believed to be related to the sensitivity of Haney Clay. During initial shearing, the metastable flocculated structure

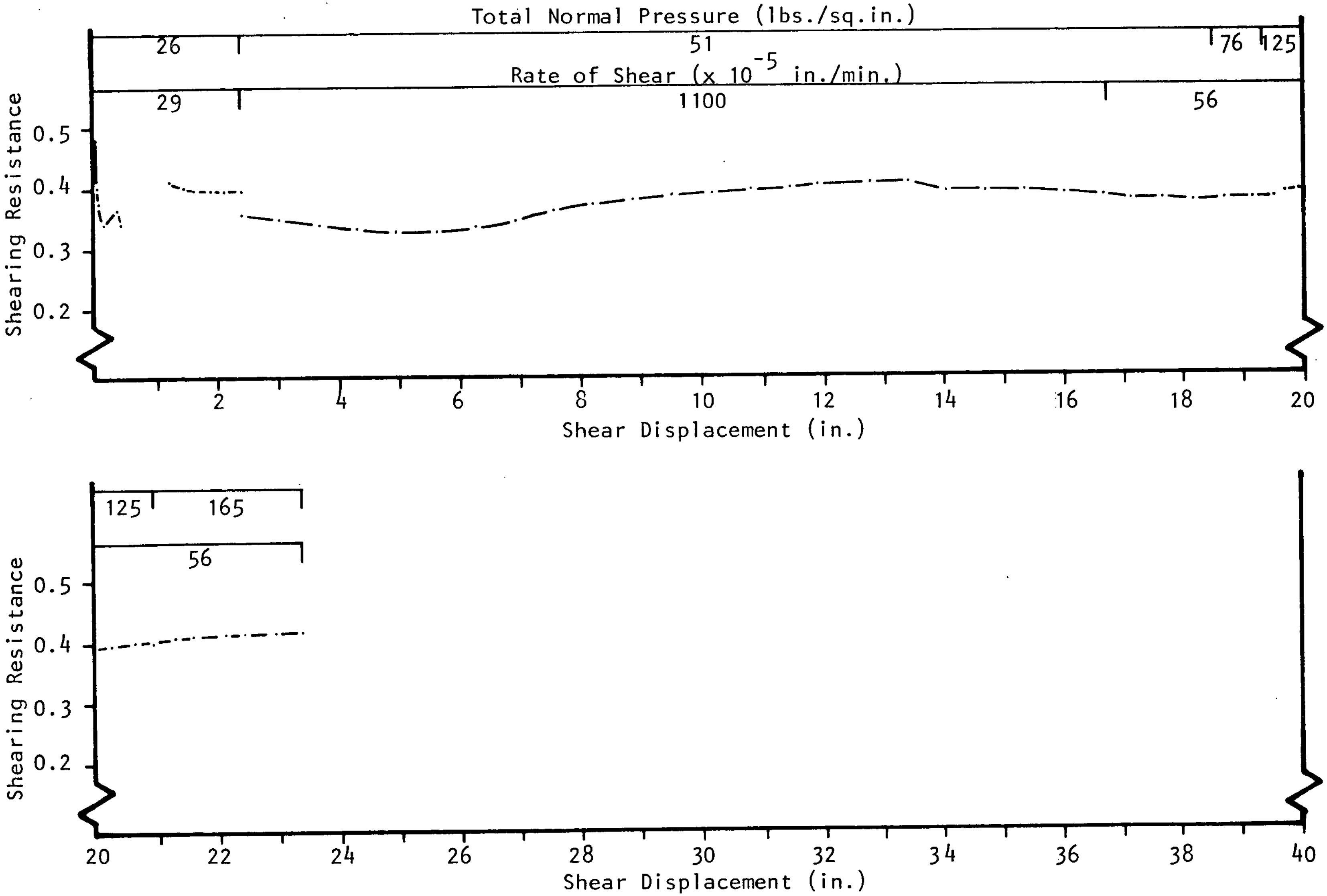


FIGURE 5.2 Shearing Resistance vs. Displacement - RS 2
Specimen: Undisturbed Haney Clay with Pre-cut Shear Plane

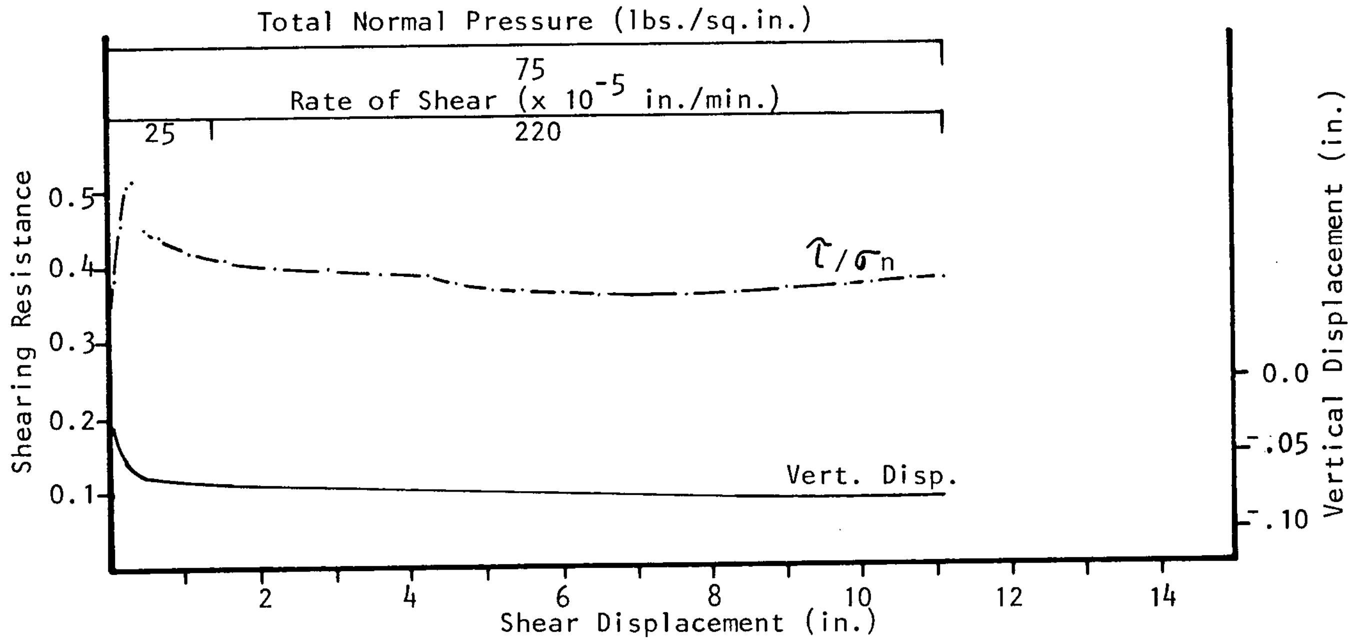


FIGURE 5.3 Shearing Resistance vs. Displacement - RS 4

Specimen: Undisturbed Haney Clay

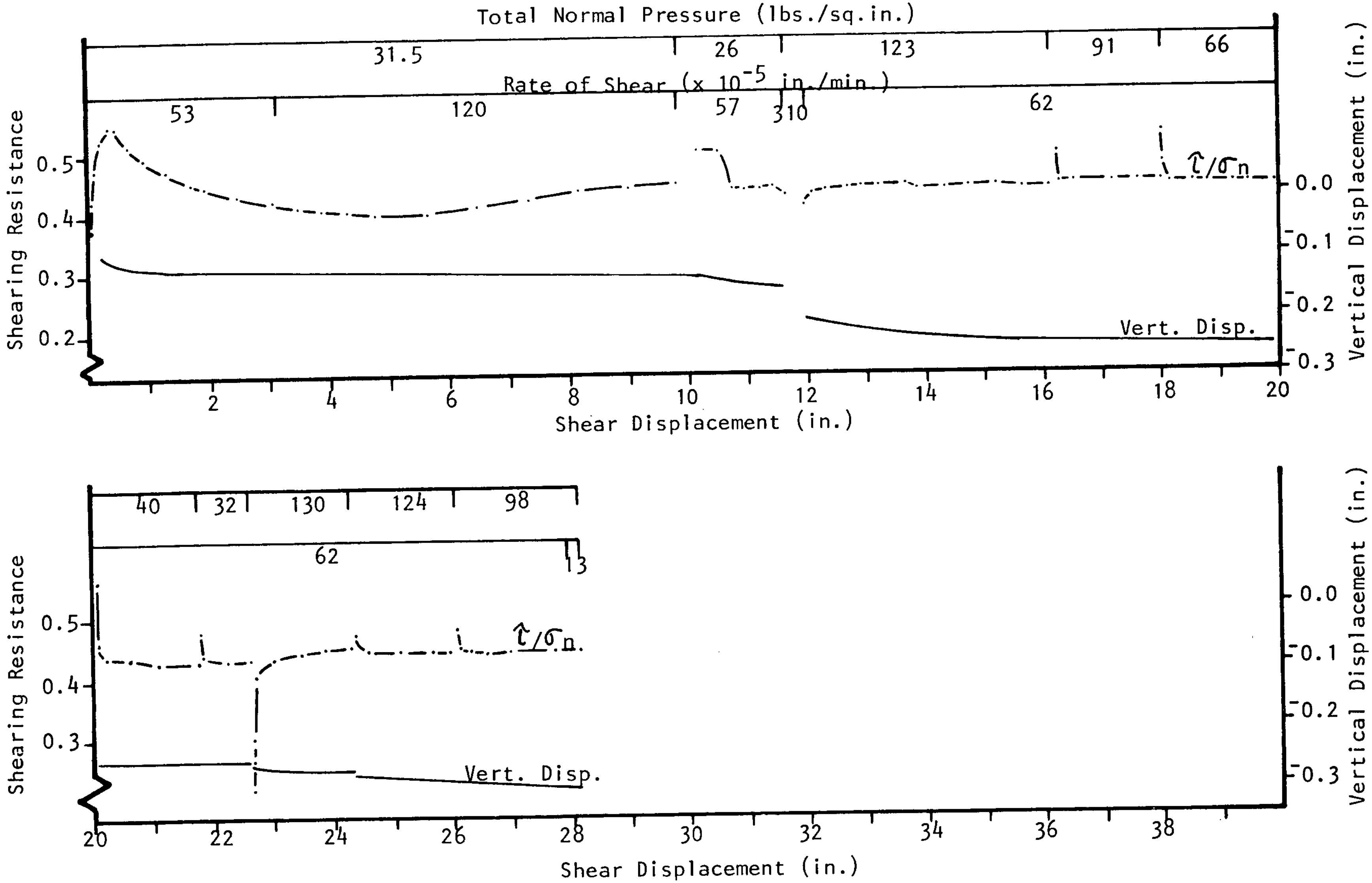


FIGURE 5.4 Shearing Resistance vs. Displacement - RS 8
Specimen: Undisturbed Haney Clay

TABLE 5.3 Test Results: RS 2 and RS 4 on Haney Clay

Test	Stage	Normal Stress	Shearing Resistance	Ø'r	Shearing Rate	Cumulative Displacement
RS 2	1	26 psi	insufficient displacement*		2.9×10^{-4} in/min	2.38 inches
	2	51 "	.370	20.3°	5.6×10^{-4} "	18.53 "
	3	76 "	.376	20.6°	5.6×10^{-4} "	19.33 "
	4	125 "	.400	21.8°	5.6×10^{-4} "	21.02 "
	5	165 "	.418	22.7°	5.6×10^{-4} "	23.51 "
			Average Ø'r = 21.3°			
RS 4	1	75 "	.379	20.8°	2.2×10^{-3} "	11.08 "

* insufficient displacement to achieve residual condition at this load.

By linear regression analysis for RS 2 and 4, Ø'r = 23.8°, $C_r' = -4.4$ psi

TABLE 5.4 Test Results: RS 8 on Haney Clay

Stage	Normal Stress	Shearing Resistance	Ø'r	Shearing Rate	Cumulative Displacement
1	31.5 psi	.446	24.04^{o}	1.2×10^{-3} in/min	9.81 inches
2	26 "	.438	23.65^{o}	5.7×10^{-4} "	11.59 "
3	123 "	.435	23.51^{o}	6.2×10^{-4} "	16.07 "
4	91 "	.445	23.99^{o}	6.2×10^{-4} "	17.98 "
5	66 "	.440	23.75^{o}	6.2×10^{-4} "	20.07 "
6	40 "	.429	23.22^{o}	6.2×10^{-4} "	21.73 "
7	32 "	.432	23.36^{o}	6.2×10^{-4} "	22.60 "
8	130 "	.451	24.28^{o}	6.2×10^{-4} "	24.29 "
9	124 "	.442	23.85^{o}	6.2×10^{-4} "	26.06 "
10	98 "	.447	24.08^{o}	6.2×10^{-4} "	28.11 "

Average Ø'r = 23.8^{o}

By linear regression analysis for RS 8, Ø'r = 24.1^{o}, C_r' = -0.7 psi

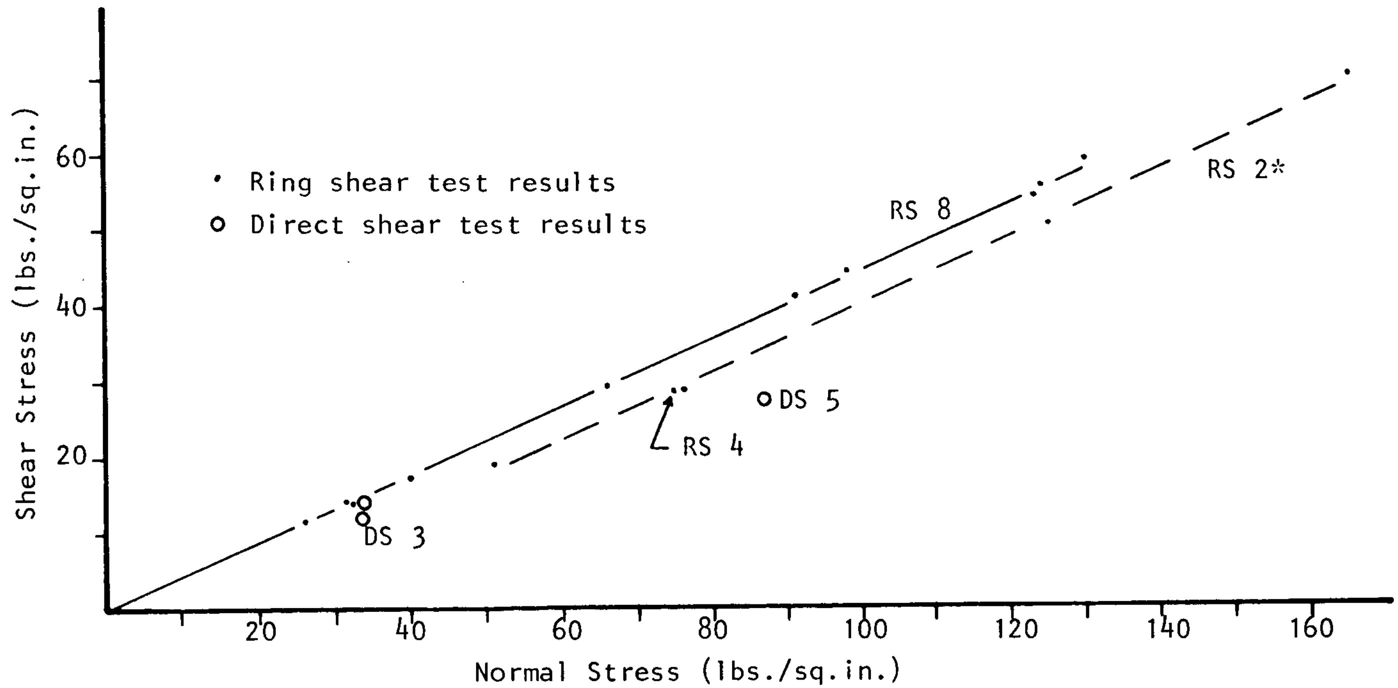


FIGURE 5.5 Mohr Rupture Diagram: Summary of Results of Residual Strength Tests on Haney Clay

* It is considered that the results of RS 2 and 4, if corrected for a calibration error, would be coincident with the results of RS 8.

of the sensitive clay is broken down within the shear plane by remolding and is replaced by a dispersed structure with greater parallel orientation of clay particles. Under constant volume conditions, the new dispersed structure is less capable of supporting loads and part of the normal stress must be transferred to the pore water, creating excess pore pressures (Mitchell and Houston, 1969). However, once the structural change is complete within the shear plane, the natural dissipation of excess pore pressures re-establishes drained conditions throughout the sample.

Excess pore pressures can also be generated after the shear plane is fully established if the sample is sheared too rapidly to allow full dissipation of pore pressures. This effect was observed in RS 8 where, at a displacement of about 12 inches (see Figure 5.4), the rate of shear was increased 6 fold producing a rapid drop in measured shear strength due to the build-up of pore pressures. A subsequent reduction in the rate of shear brought about quick dissipation of the excess pressures and return to equilibrium.

Following establishment of the shear plane and dissipation of excess pore pressures, the shearing resistance recorded in RS 8 became virtually constant regardless of normal load, as shown in Figure 5.4. This is the anticipated result based on previous experience, as described in Chapter 2. However, the results of RS 2, tabulated in Table 5.3, record an increase in apparent shearing resistance with normal stress. The single residual strength determination made in RS 4 is in close agreement with one of the determinations made in RS 2.

The true relationship between the results of these ring shear tests is best illustrated by the Mohr plot, Figure 5.5. As shown in the figure, the results of RS 8 indicate that Haney Clay has a residual angle of friction of 23.8 degrees and no residual cohesion. The results of RS 2 and RS 4 exhibit the same angle of friction, but the data is shifted to indicate a negative cohesion intercept. The shift in cohesion intercept is evidence of a constant and systematic experimental error. The error ended following recalibration of the transducers at the end of RS 4, indicating that the source of the problem was a zero-shift error in transducer calibration. When corrected for this error, the results of RS 2 and RS 4 would be coincident with the results of RS 8, indicating that Haney Clay has a residual angle of friction of 23.8 degrees, with no residual cohesion.

5.3.3 Direct Shear Tests

TEST PROCEDURES AND RESULTS

Two single stage direct shear tests for the purpose of determining residual strength were performed on Haney Clay: DS 3 and DS 5, carried out at applied normal stresses of 34 and 87 psi, respectively. Both samples were placed as undisturbed specimens, but the shear plane was cut prior to testing in DS 5.

The tests consisted of a series of alternating forward and reverse traverses of the shear box, the length of each traverse not usually exceeding 0.3 inches. The shear stress versus displacement behaviour of the soil in the individual traverses consisted of variations of the typical shear behaviour of a normally consolidated clay, illustrated in Figure 2.1. The test results have been summarized in plots of the ultimate shearing resistance determined in each traverse of the shear box versus cumulative shear displacement, presented in Figures 5.6 and 5.7. Vertical displacement data is also shown on these figures. The residual strength results as interpreted from this data are presented on the Mohr plot, Figure 5.5. Tabulated results are presented in Table 5.5.

DISCUSSION AND INTERPRETATION OF RESULTS

The results of direct shear tests on Haney Clay indicate that there is a reduced displacement to peak strength when the failure plane is cut prior to shearing, also demonstrated in ring shear tests. DS 3, carried out on undisturbed soil, and DS 5, carried out on a pre-cut specimen, attained peak strength at cumulative displacements of 0.6 and 0.1 inches, respectively.

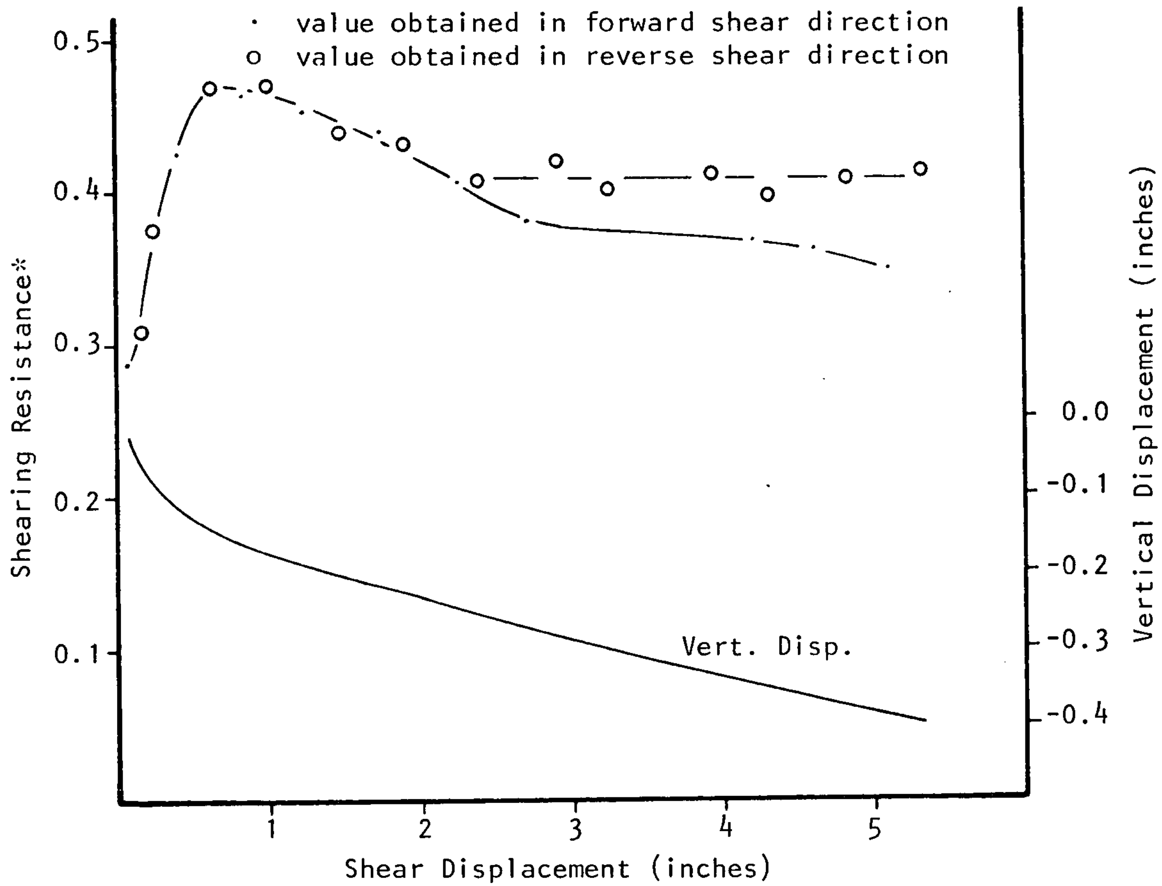


FIGURE 5.6 Shearing Resistance vs. Cumulative Displacement - DS 3
Specimen: Haney Clay

Total Normal Pressure = 33.9 psi
Typical Rate of Shear = 10×10^{-5} in./min.
* Ultimate value in each recorded traverse

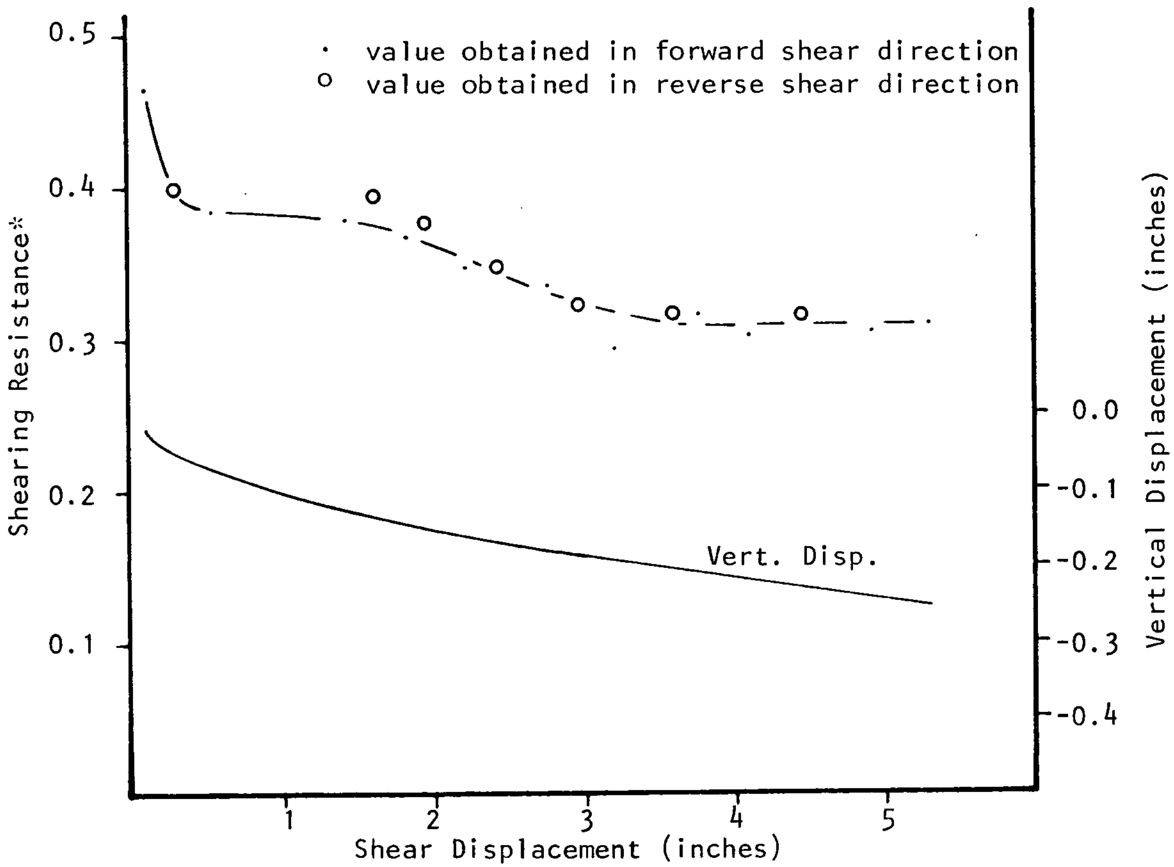


FIGURE 5.7 Shearing Resistance vs. Cumulative Displacement - DS 5
Specimen: Haney Clay

Total Normal Pressure: 87.3 psi
Typical Rate of Shear: 7×10^{-5} in./min.
* Ultimate value of each recorded traverse

TABLE 5.5 Test Results: Direct Shear Tests on Haney Clay

Test	Normal Stress	Shearing Resistance	Ø'r apparent	Shearing Rate	Cumulative Displacement
DS 3	33.85 psi	*F .350	F 19.3°	1×10^{-4} in/min	5.3 inches
		R .412	R 22.4°	1×10^{-4} "	
DS 5	87.3 "	.310	17.2°	7×10^{-5} "	5.3 "

* F - value obtained in forward shear direction

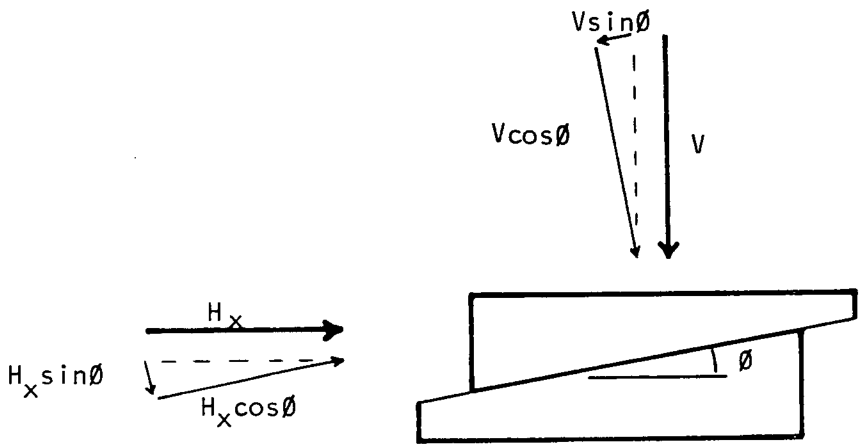
R - value obtained in reverse shear direction

DS 3 exhibited a gradual fall in shearing resistance beyond peak strength displacement until, at a displacement of about 2.5 inches, the shearing resistances measured in the forward and reverse directions diverged, with the higher resistance being registered in the reverse direction. This dichotomy in measured resistance was maintained throughout the remaining test period. As indicated by the lack of divergence in the initial test period and confirmed by subsequent examination, the divergence in resistance was not caused by an error in the calibration of the proving ring used to measure shear force.

The diverging resistance phenomenon, also observed in tests on Hat Creek Clay, is apparently the result of an inclined failure surface. An inclined failure surface is mechanically permissible in direct shear testing because the freedom of movement of the upper loading frame includes vertical and, to a lesser degree, rotational motion.

To analyze the effect of an inclined plane of failure, the vertical and horizontal applied loads have been broken down into components acting parallel and perpendicular to the inclined failure surface in Figure 5.8. The resolved normal and shear forces as related by the friction angle can be described by two equations, one for each direction of travel, in two unknowns. The two unknowns, which can be determined from these equations are the angle of shear plane inclination and the angle of friction of the soil. The angle of friction is the residual angle provided drained residual conditions have been established within the shear plane. The equations resulting from the preceding analysis are presented in Figure 5.8.

Toward the end of testing, DS 3 exhibited a gradual decrease in the angle of friction measured in the forward direction and a gradual



ø	inclination of shear plane
θ	angle of friction for soil
V	applied vertical force
H_1	applied horizontal force in forward direction
H_2	applied horizontal force when shear direction is reversed
N	resolved forces normal to failure plane
S	resolved forces parallel to failure plane

In terms of forces, the equation of soil strength is as follows.

$$S = N \tan \theta \qquad (1)$$

Resolve the applied forces shown above and substitute in equation (1) to obtain:

$$(H_1\cos \phi - V\sin\phi) = (V\cos\phi + H_1\sin\phi)\tan\theta \qquad (2)$$

Reverse the direction of applied horizontal force, resolve and substitute in equation (1) to obtain:

$$(H_2\cos\phi + V\sin\phi) = (V\cos\phi - H_2\sin\phi)\tan\theta \qquad (3)$$

Combine equations (2) and (3), collect and simplify terms, to determine the inclination of the shear plane.

$$\tan 2\phi = \frac{V(H_1 - H_2)}{V^2 + H_1H_2} \qquad (4)$$

The soil friction angle is obtained by entering the determined angle of inclination in equations (2) or (3).

FIGURE 5.8 Analysis of Direct Shear Results Influenced by an Inclined Failure Surface increase in the friction angle measured in the reverse direction. The final recorded values were about 19.3 and 22.4 degrees in the forward and reverse directions, respectively. By applying the analysis described above, it is calculated that the true angle of friction at the end of testing was 20.8 degrees, the diverging recorded values resulting from a shear plane inclination of 1.5 degrees.

At the end of DS 3, the sample was removed and photographed. As seen from a view of the upper loading frame in Figure 5.9, the shear plane is well established, highly polished, and slickensided. The shear plane is not flat but rounded, having protruded into the lower loading frame. The shape of the failure plane may result from the gradual loss of remolded soil through the gap between the loading frames. Lost soil is replaced by fresh material from the upper loading frame only, resulting in a gradual erosion of soil from the lower loading frame. The shear plane develops a rounded shape due to edge constraints which prevent uniform soil erosion.

Herrman and Wolfskill (1966, p. 120) encountered similar shaped shear planes in repeated direct shear tests on weak clay-shales, noting that the apparent shearing resistance was increased by the inclination of the failure surface. However, as measurements were made in only one traverse direction, they did not record the corresponding decline in apparent shearing resistance in the opposite direction of travel.

Beyond peak strength displacement in DS 5, the shearing resistance fell to a temporary plateau early in testing, followed by a further drop to a final plateau. At the end of testing, DS 5 exhibited an angle of friction of about 17.2 degrees with some minor scatter of data.

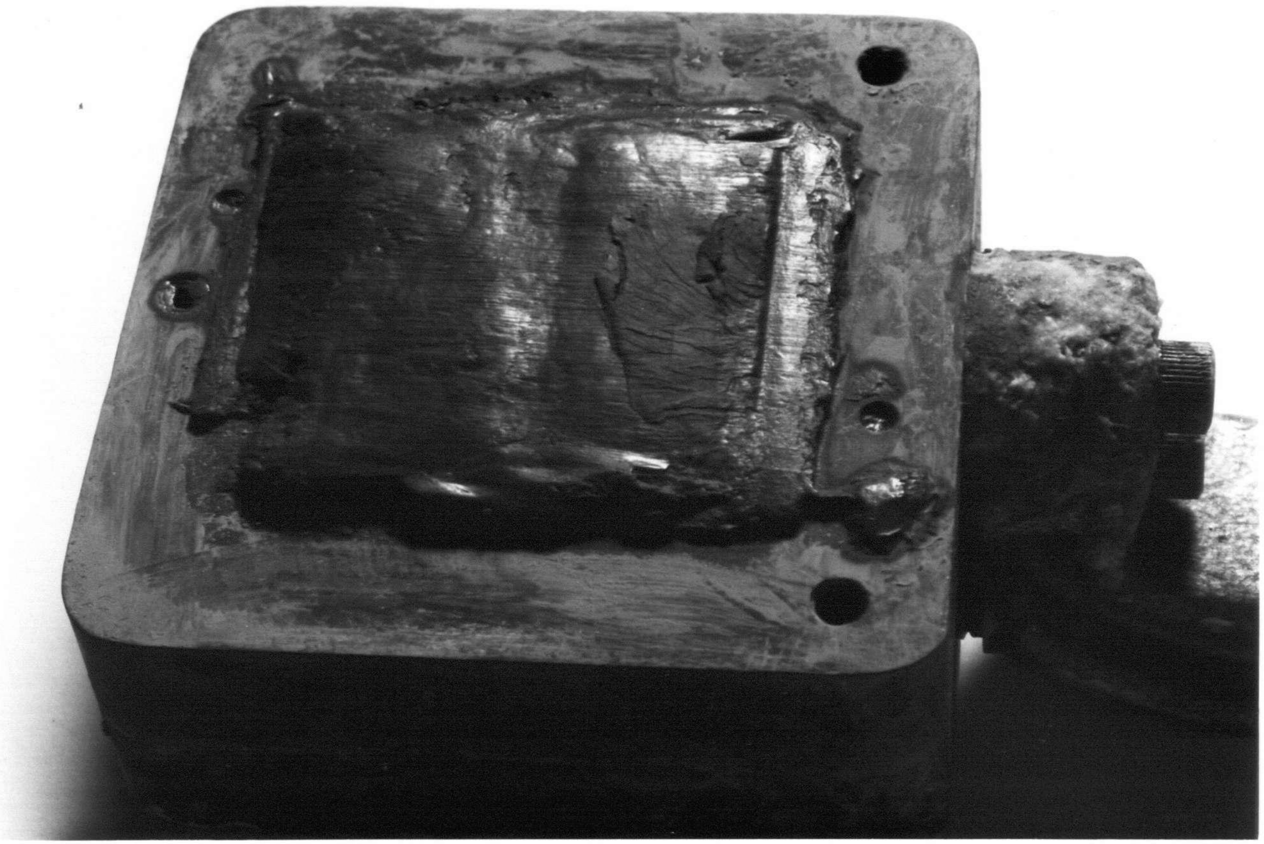


FIGURE 5.9 Shear Plane at End of Testing - DS 3

View of upper loading frame. Note the highly polished, slickensided surface. The curvature of the shear plane may alter the apparent shearing resistance of the soil.

5.3.4 Review and Evaluation of Results

The results of ring shear tests on three samples of Haney Clay indicate that this soil has a residual angle of friction of 24 degrees. The uniformity of the data obtained over a wide range of normal loads provides a strong degree of confidence in this determination of residual friction angle. A zero-shift calibration error during RS 2 and RS 4 produced data which, while falsely indicating a negative cohesion intercept, is nevertheless in full agreement with the determined angle of residual friction.

The results of the direct shear tests carried out on Haney Clay, DS 3 and DS 5, indicate that Haney Clay has a residual friction angle of 21 and 17 degrees, respectively. A divergence in shearing resistance recorded in the forward and reverse traverse shear directions observed in DS 5, is believed to result from the inclination of the shear surface. As the shear plane inclination is small, in the order of 1 to 2 degrees, the vertical motion caused by the inclination is masked by the downward movement due to sample losses which, in this series of tests, is 1.5 to 3.0 times greater. The diverging resistance phenomena does not occur in ring shear testing because the confining rings are not capable of vertical movement or rotational motion about a horizontal axis. These constraints make shear plane inclination mechanically impermissible.

For a variety of reasons the degree of confidence in the residual strength determinations made using the direct shear device is quite limited. Factors influencing this judgement include:

1. Significantly reduced total displacements in relation to the large displacements obtained in the ring shear apparatus.

2. Moderately scattered data.
3. Limited number of residual strength determinations (two) having conflicting results.

The strong degree of confidence in the results of ring shear tests and the uniform nature of Haney Clay would suggest that the true residual friction angle is 24 degrees, and that the values obtained in direct shear tests are too low. Barring errors in measurement, the source of the lowered values must be excess pore pressures within the soil samples.

The most probable source of the excess pore pressures would appear to be ingestion of water into the shear plane during testing. At the end of each traverse, part of the sample is extended into the reservoir. This material, exposed to water and carrying no normal load, is free to swell. As shear displacement continues, the swelled soil is drawn back into the failure plane and again subjected to normal loading, generating excess pore pressures within the swelled soil during the reconsolidation period. Repeated exposure of the ends of the failure plane to water might be expected to foster a gradual build-up of excess pore pressure in the sample.

Other factors which might influence the pore pressures in the sample include the rate of shear, the rate of soil loss from the loading frame gap, and the length of pause between traverses - a longer pause allowing greater dissipation of pore pressures and greater swelling of exposed soils. However, the author has found no apparent correlation in the available data between these parameters and the measured shear strength.

5.4 RESIDUAL STRENGTH TESTS ON HAT CREEK CLAY

5.4.1 Soil Description

Hat Creek Clay is a heavily overconsolidated, brittle, highly plastic, dark blue clay. The soil was core sampled from extensive clay deposits associated with coal measures in the Hat Creek Valley of British Columbia. A study performed at the University of Western Ontario shows that the soil contains a high percentage of sodium montmorillonite with variable amounts of carbonate, quartz and feldspar (Quigley, 1976). The same study presented the following typical properties for the clay:

Liquid limit	109 to 178%
Plastic limit	34 to 39%
Natural water content	35%
Percent carbonate	1 to 68

The samples tested at UBC, containing a relatively high percentage of silt and sand sized particles, had the following characteristics:

Liquid limit	84 to 119%
Plastic limit	27 to 28%
Plasticity index	68 to 92%
Natural water content	29%
Percent greater than 2 microns	40 to 50%

Based on the results of a strain-controlled consolidation test, Hat Creek Clay has a permeability in the order of 10^{-10} cm per second.

5.4.2 Ring Shear Tests

TEST PROCEDURES AND RESULTS

Three single stage ring shear tests were performed on Hat Creek Clay: RS 5, RS 6 and RS 7, carried out at applied normal stresses of 26.5, 60 and 27 psi, respectively. The test data is presented in plots of shearing resistance versus displacement in Figures 5.10 to 5.12. The final results, tabulated in Table 5.6, are plotted on a Mohr diagram in Figure 5.13.

To facilitate sample placement, the hard and brittle clay was crushed by mortar and pestle before being placed for testing. The soil sample for RS 5 was crushed and placed at the natural water content. For RS 6, the soil was crushed at the natural water content, but lightly wetted as it was placed. The soil for RS 7 was crushed, wetted and remolded until it exhibited a plastic consistency before being placed.

To reduce the time required for pore pressure dissipation at the shear plane, the shortest drainage path to the failure plane was reduced to 1/8 inch for this test series by placing a spacer beneath the lower porous platen. Due to the large sand fraction in the soil, further reductions in the drainage path would have increased the probability that sand grains lodged against the platen might protrude through the failure plane, disrupting failure plane development and increasing the apparent shearing resistance of the soil.

During testing of Hat Creek Clay, sand particles from the soil sample had a tendency to become wedged between the confining rings, hampering adjustment of the gap. The particles were dislodged

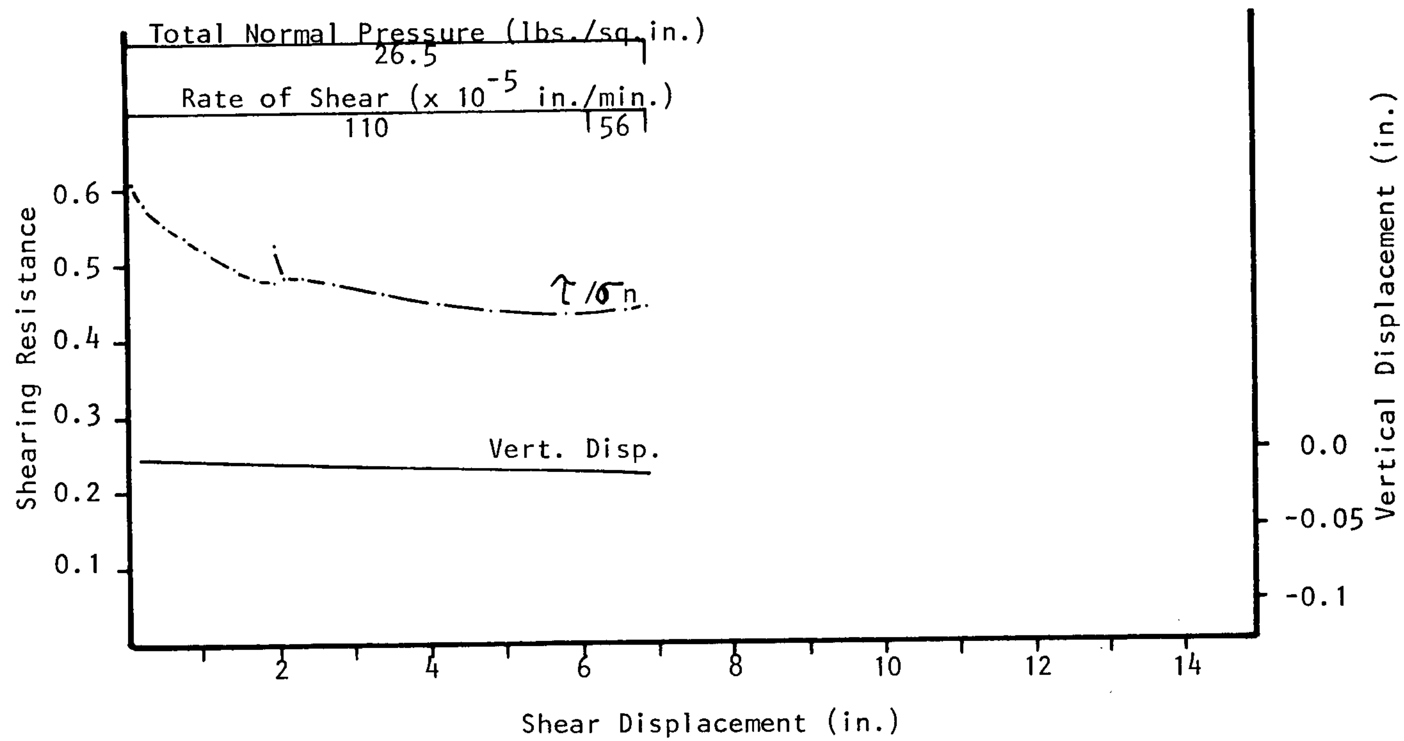


FIGURE 5.10 Shearing Resistance vs. Displacement - RS 5

Specimen: Hat Creek Clay

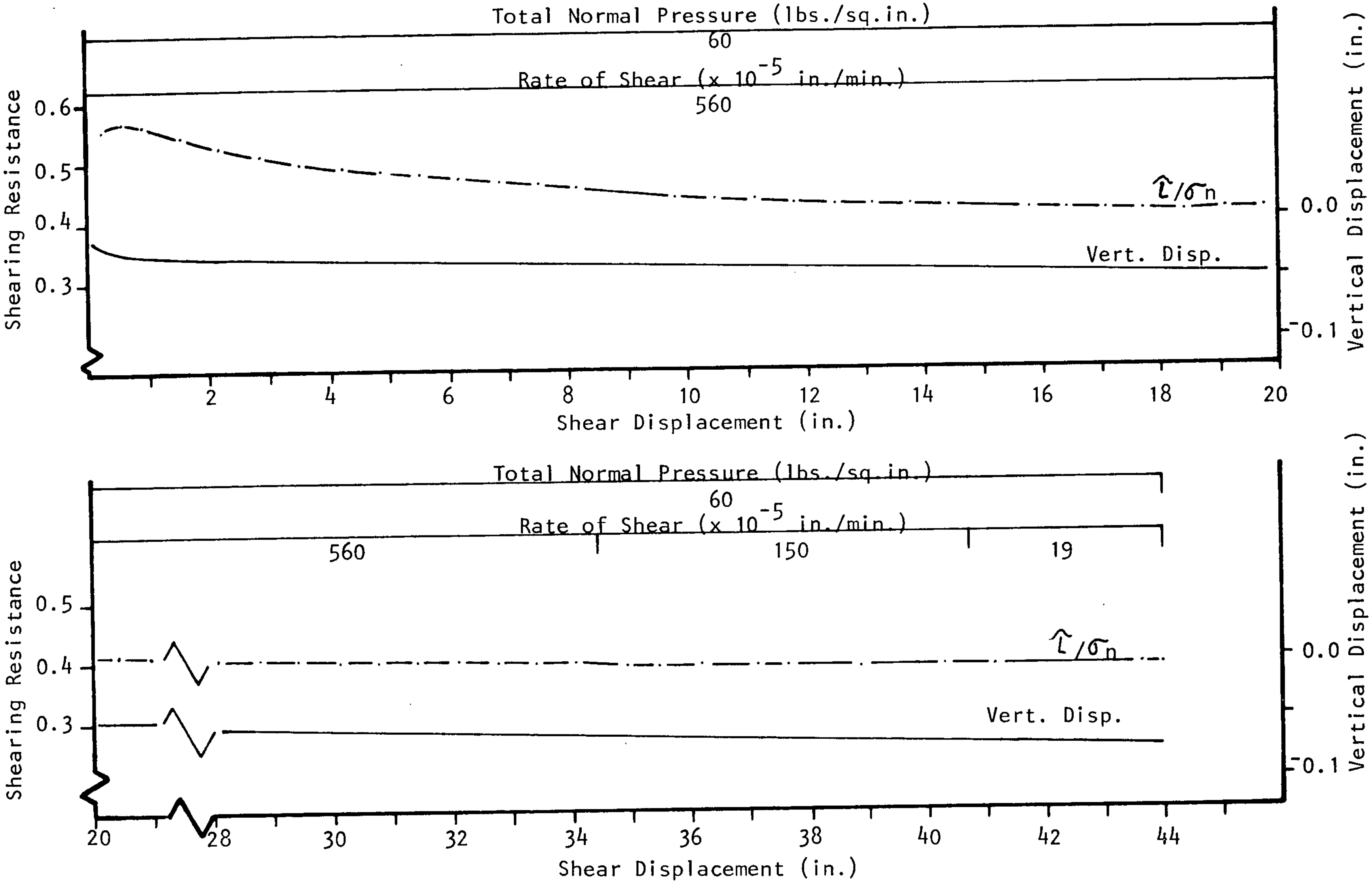


FIGURE 5.11 Shearing Resistance vs. Displacement - RS 6
Specimen: Hat Creek Clay

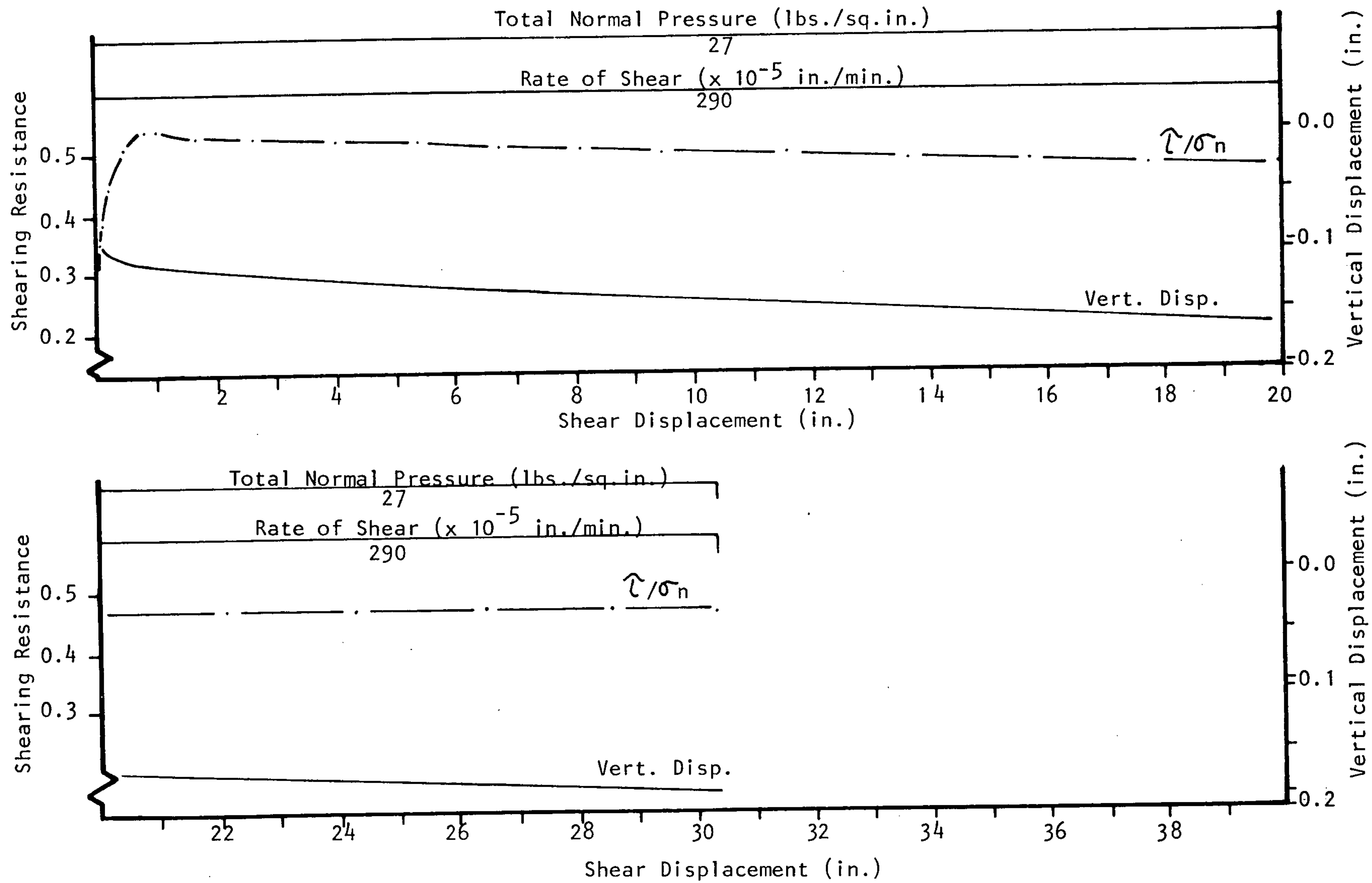


FIGURE 5.12 Shearing Resistance vs. Displacement - RS 7
Specimen: Hat Creek Clay

TABLE 5.6 Test Results: Ring Shear Tests on Hat Creek Clay

Test	Stage	Normal Stress	Shearing Resistance	Ø'r	Shearing Rate	Cumulative Displacement
RS 5	1	26.5 psi	.445	24.0°	5.6×10^{-4} in/min	6.84 inches
RS 6	1	60 "	.382	20.9°	1.9×10^{-4} "	43.98 "
RS 7	1	27 "	.461	24.8°	2.9×10^{-3} "	30.33 "

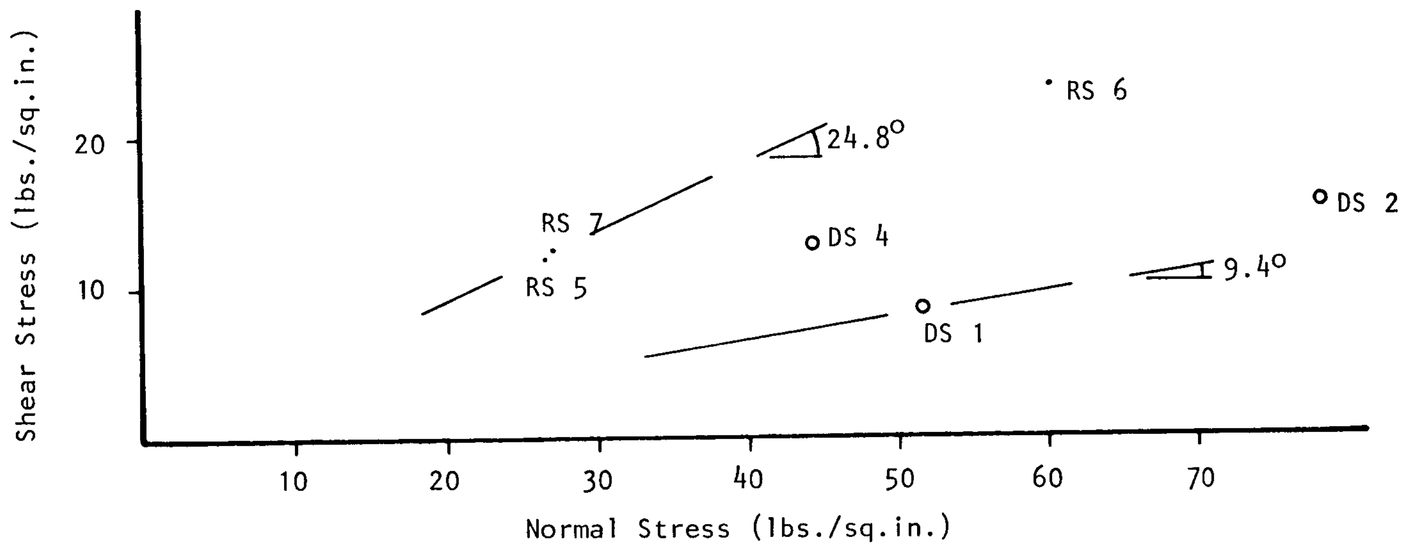


FIGURE 5.13 Mohr Rupture Diagram: Summary of Results of Residual Strength Tests on Hat Creek Clay

by clamping the rings shut while shearing the sample. The problem was eliminated by maintaining a very narrow confining ring gap.

Due to the swelling nature of Hat Creek Clay, material lost through the confining ring gap increased in volume producing the appearance of excessive sample losses. However, the actual rate of sample loss as measured by the change in sample thickness was minor.

DISCUSSION AND INTERPRETATION OF RESULTS

At the end of the first ring shear test on Hat Creek Clay, RS 5, the apparent friction angle (neglecting possible excess pore pressures) was 24 degrees. As shown in Figure 5.10, the shearing resistance was stabilizing when the test was ended such that significant changes in shearing resistance would not anticipated if testing had been extended to any reasonable displacement and duration. Considering that the accepted values of residual friction angle for montmorillonite clay ranges from 4 to 10 degrees (see Table 2.2), the shearing resistance encountered in testing was higher than expected.

Similar results were obtained during RS 6 and RS 7 which, at the end of testing, exhibited apparent residual friction angles of 20.9 and 24.8 degrees, respectively.

The high values of soil shearing resistance are attributed to shear plane disruptions caused by sand particles in the soil. The disruptions to shear plane development may have been intensified by the shortened drainage path should the 1/8 inch separation between porous platen and shear plane have been insufficient to prevent interaction between sand particles lodged against the platen and the shear plane. The influence of sand and silt content on the shearing resistance of several different clays is illustrated in Figure 5.14. As shown in the figure, the curves for montmorillonite clay indicate that Hat Creek Clay, having a sand and silt fraction of 40 to 50 percent, would be expected to exhibit a shearing resistance similar to that of the pure clay. However, the continual loss of fines through the confining ring gap during testing may have concentrated the coarser particles in the shear plane, increasing the shearing resistance significantly.

The samples appear to have been well-drained at the end of testing, showing little or no variation in measured shearing resistance with increasing displacement or with changes in rate of shear. Further evidence that the samples were well-drained is provided by the similarity of results obtained from RS 5, where the soil was placed at its natural water content, and RS 7, where the soil was wetted and remolded prior to placement.

Very high values of side friction (frictional forces of the loading platen and the soil against the upper confining rings) were encountered in RS 5. In previous tests, the measured side friction had been in the order of 10 to 20 percent of the total applied load. In RS 5, the side friction began at a very low value but steadily increased during the test. When the side friction exceeded 30 percent of the applied load, the test was terminated to prevent possible damage to the equipment. The high values of side friction are attributed to corrosion welts on the upper confining rings, but could also result from misalignment of machine components during assembly or from granular soil particles wedged between the confining rings and the upper loading platen.

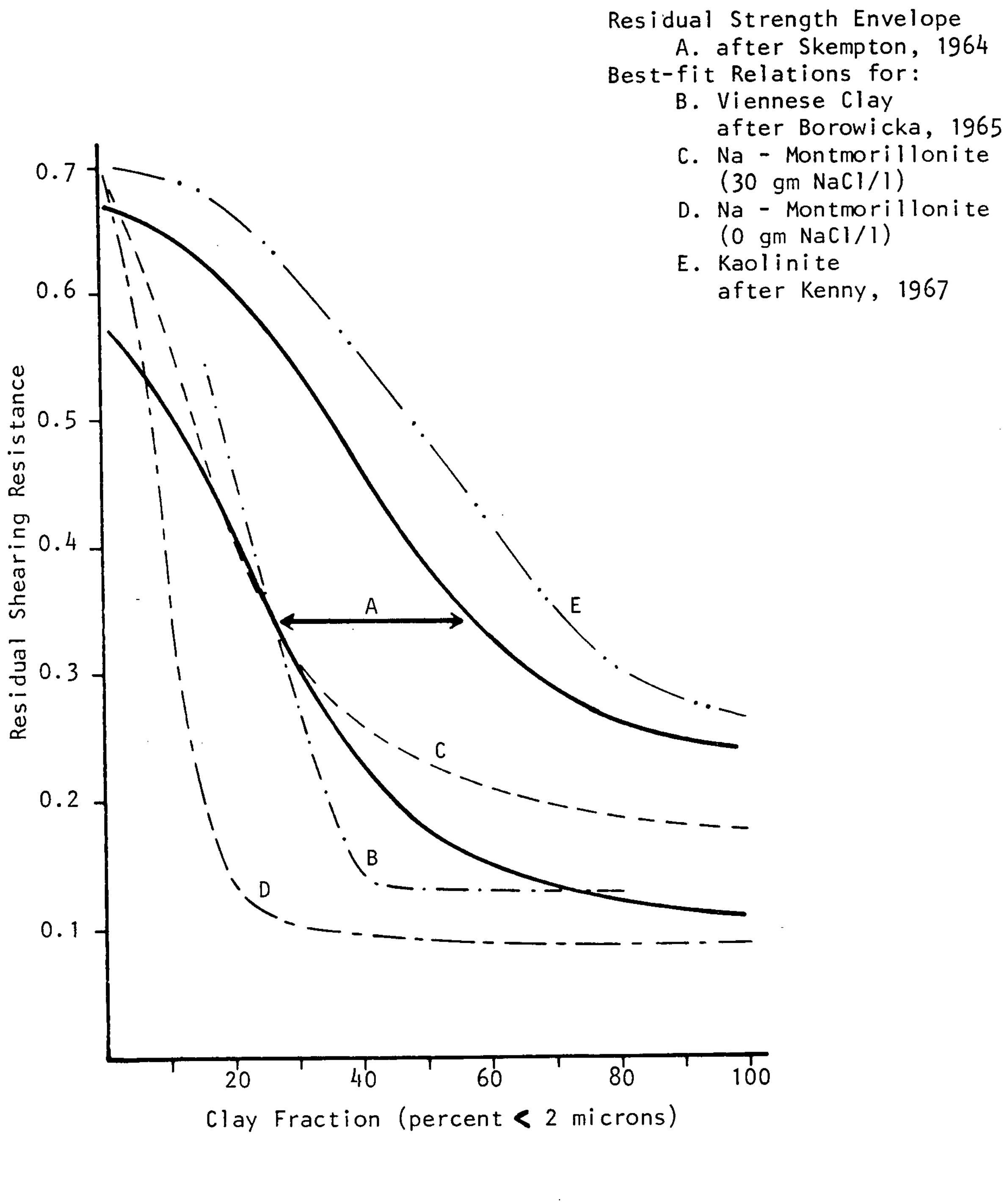


FIGURE 5.14 Relation Between Clay Content and Residual Shearing Resistance for Various Soils

5.4.3 Direct Shear Tests

TEST PROCEDURES AND RESULTS

Three single stage direct shear tests for the purpose of determining residual strength were performed on Hat Creek Clay: DS 1, DS 2, and DS 4, carried out at applied normal stresses of 52, 78.5 and 44.5 psi, respectively.

To prepare the sample for DS 1, the soil was broken and ground with mortar and pestle, and subsequently placed in lifts in the direct shear box. After each lift was placed, the loading block was laid on top and hammered firmly to compact the soil The sample was made as deep as possible to allow for later consolidation and loss of sample by squeezing through the loading frame gap.

The sample for DS 2 was placed in a similar manner, except that each lift of soil was sprinkled with distilled water to moisten the sample.

The sample for DS 4 was wetted, remolded, and sieved to remove the medium to coarse sand fraction. The remaining soil was reconsolidated under strain-controlled conditions, trimmed and placed in the shear box.

The test results have been summarized in plots of the ultimate shearing resistance determined in each traverse of the shear box versus cumulative displacement, Figures 5.15 to 5.17. Vertical displacement data is also shown on these figures. The residual strength results interpreted from the data are presented on a Mohr plot in Figure 5.13. Tabulated results are presented on Table 5.7.

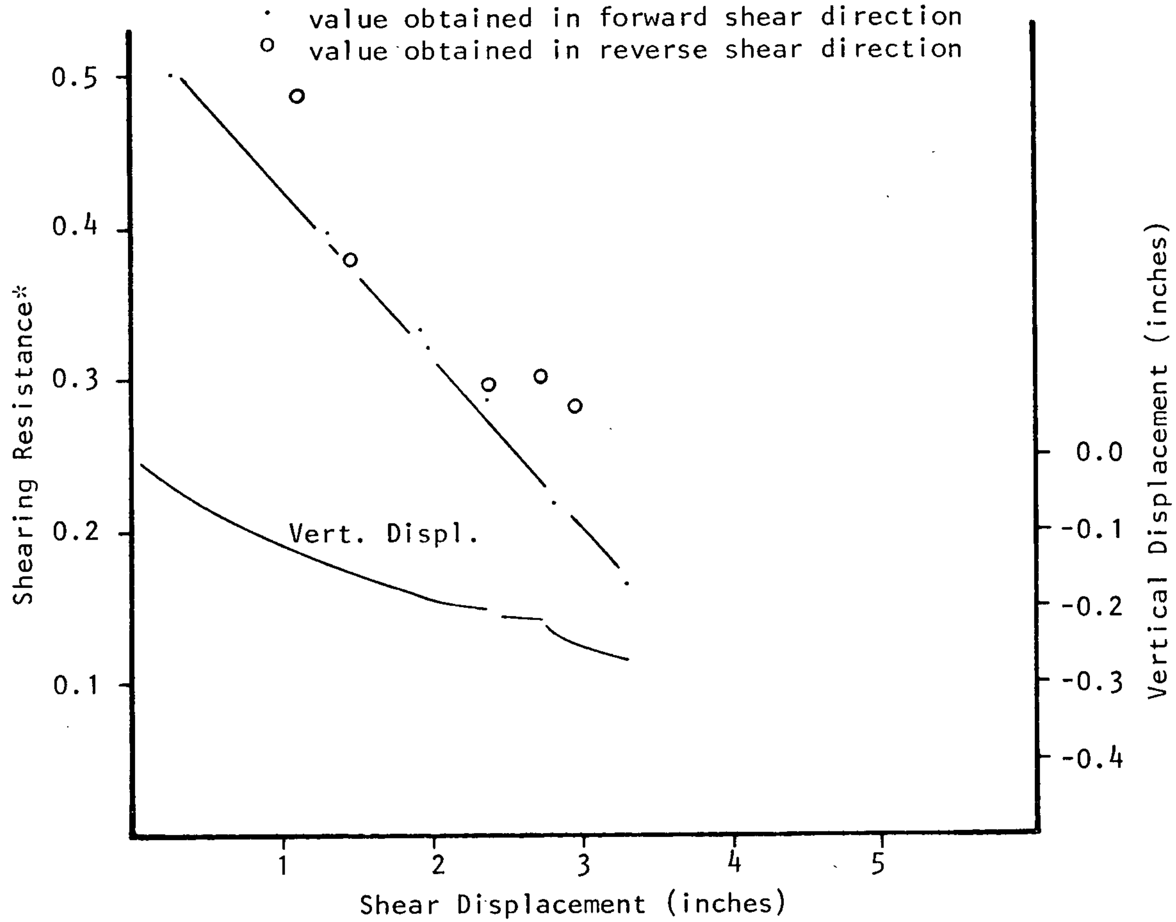


FIGURE 5.15 Shearing Resistance vs. Cumulative Displacement - DS 1

Specimen: Hat Creek Clay

Total Normal Pressure: 51.8 psi

Typical Rate of Shear: 50×10^{-5} in./min.

* Ultimate value in each recorded traverse

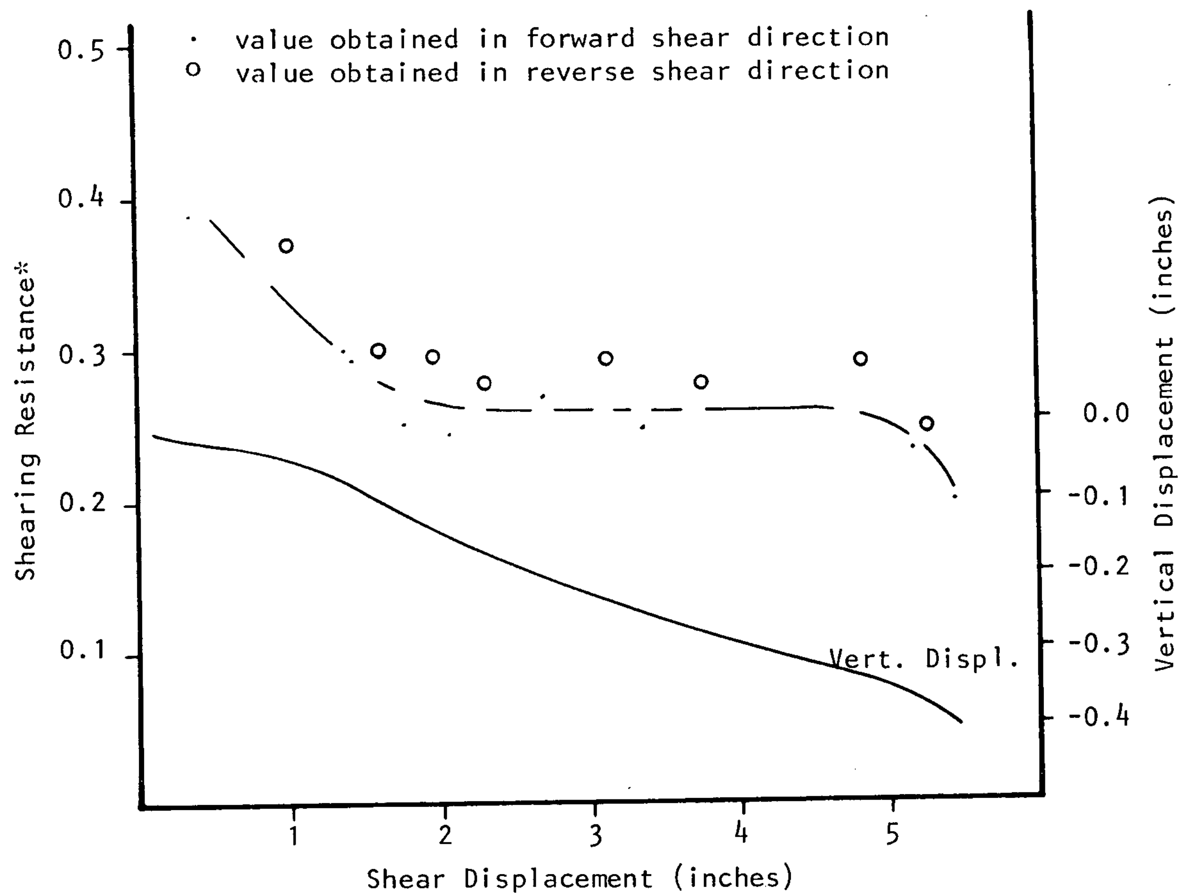


FIGURE 5.16 Shearing Resistance vs. Cumulative Displacement - DS 2

Specimen: Hat Creek Clay

Total Normal Pressure: 78.4 psi

Typical Rate of Shear: 25×10^{-5} in./min.

* Ultimate value in each recorded traverse

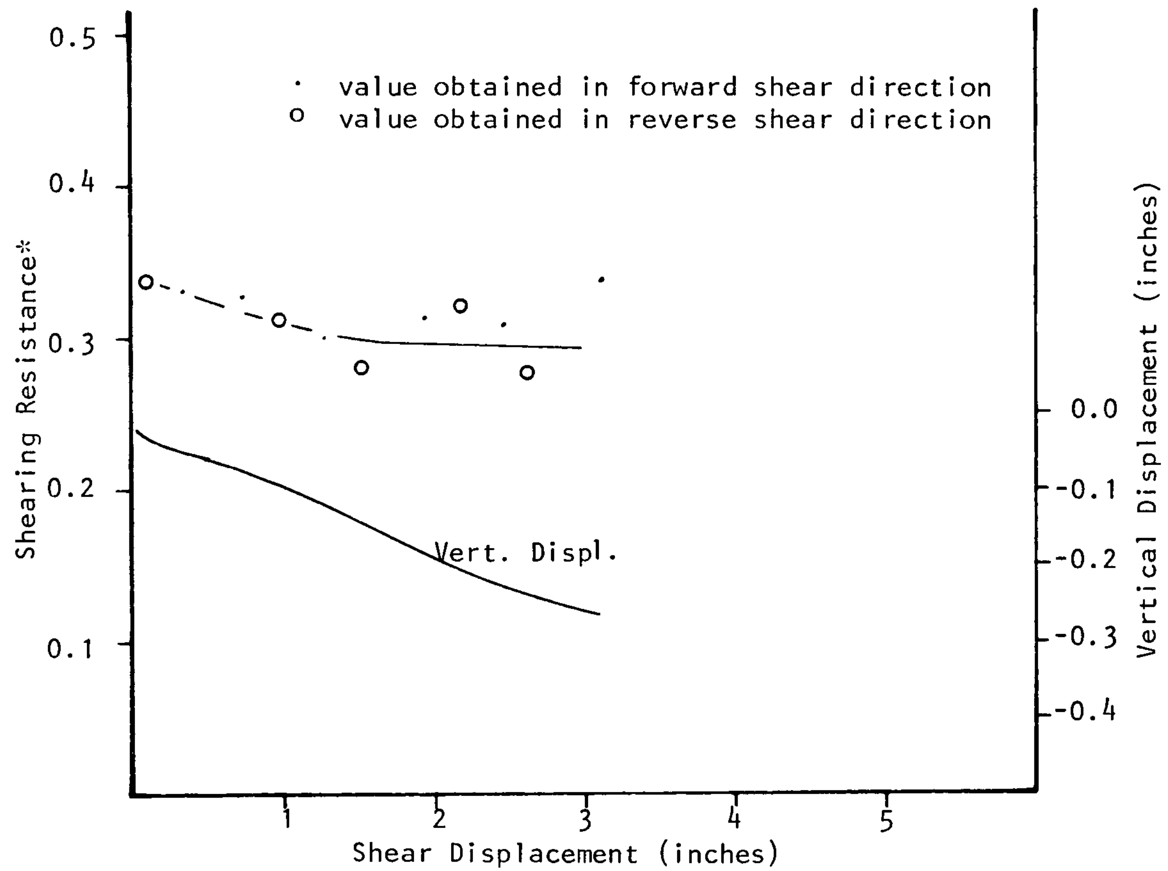


FIGURE 5.17 Shearing Resistance vs. Cumulative Displacement - DS 4
Specimen: Hat Creek Clay

Total Normal Pressure: 44.4 psi
Typical Rate of Shear: 4×10^{-5} in./min.
* Ultimate value in each recorded traverse

TABLE 5.7 Test Results: Direct Shear Tests on Hat Creek Clay

Test	Normal Stress	Shearing Resistance*	Ø' apparent**	Shearing Rate	Cumulative Displacement
DS 1	51.8 psi	F .166	9.4^{o}	5.0×10^{-4} in/min	3.28 inches
DS 2	78.4 "	F .198	11.2^{o}	2.5×10^{-4} "	5.44 "
DS 4	44.4 "	S .290	16.2^{o}	3.7×10^{-5} "	3.10 "

* F - value obtained in forward shear direction

S - 'average' value of scattered results

** Tests ended prematurely due to cumulative sample losses. Residual conditions not achieved.

The shearing resistance versus displacement plot for DS 1, Figure 5.15, does not show data for a rapid pre-shearing period carried out prior to regular testing. This sample was pre-sheared for a period of 5 consecutive test days during which the measured shear angle remained constant at 31 degrees. After a three day pause in testing to allow for equalization of pore pressures, the measured shear angle fell to 21.7 degrees. Subsequent test results are summarized in Figure 5.15.

STRAIN CONTROLLED CONSOLIDATION TEST

As previously discussed, soil later tested in DS 4 was remolded, sieved to remove the medium to coarse sand fraction, and reconsolidated in a strain controlled test. During reconsolidation, free drainage was permitted at the top of the sample only, while pore pressure was monitored at the sample base. The basal pore pressures remained in the order of 90 to 100 percent of the applied stress throughout the test due to the extremely low permeability of the soil. The applied strain rate was 2.4×10^{-5} inches per minute, the lowest rate of which the test apparatus was capable.

Although the assumptions used to analyze the results of the strain controlled consolidation test require that the basal pore pressure does not greatly exceed 10 percent of the applied stress (Byrne and Aoki, 1969), the results were nevertheless sufficient to demonstrate that the permeability of Hat Creek Clay must be in the order of 10^{-10} cm per second or less. This value is in agreement with that estimated by Quigley (1976).

DISCUSSION AND INTERPRETATION OF RESULTS

As described above, DS 1 exhibited high shear resistance over a 5 day rapid displacement preshearing period not recorded on the data plots. It is likely that the original dry state of the soil combined with its low permeability resulted in the initial testing being done on essentially dry soil. The high shear angle measured during preshearing (31 degrees) is not unreasonable for a crushed dry clay.

The subsequent decline in the measured shearing resistance, shown in Figure 5.15, would indicate that water has reached the shear plane. The low permeability of the soil appears to preclude the possibility of drainage from the porous base so early in testing. However, water may have been ingested into the shear plane as the ends of the soil sample were alternately thrust into the water reservoir and drawn back into the shear plane during testing. This mechanism, previously detailed in the review of test results on Haney Clay, could explain the drop in shear strength of the soil over a relatively short period of time.

At the completion of DS 1, the apparent angle of friction was about 9.5 degrees and decreasing. In DS 2, the apparent angle of friction remained approximately constant at 15 degrees throughout most of the test period, as shown in Figure 5.16. However, in the last few reversals the shear angle declined to about 11 degrees. In DS 4, performed on reconsolidated Hat Creek Clay, the shearing resistance fell gradually at the start of the test before levelling out at an angle of friction of about 16 degrees. In each case, testing was ended when cumulative sample loss made further testing impractical.

As encountered in direct shear tests on Haney Clay, the tests on Hat Creek Clay generally exhibited greater shear resistance when tested in one direction than in the other. This divergence of recorded resistance is attributed to an inclination of the failure surface. By applying the analysis summarized in Figure 5.8, it is calculated that the shear planes in DS 1, DS 2 and DS 4 had inclinations of about 2.2, 1.1 and 0.9 degrees, respectively, near the end of testing.

All of the direct shear tests on Hat Creek Clay exhibited a generally constant shearing resistance in the early stages of testing. In DS 1, the shearing resistance remained virtually constant throughout a 5 day rapid displacement preshearing period at a value corresponding to an apparent angle of friction of 31 degrees. In DS 2 and DS 4, a constant value of 15 to 16 degrees was attained after a displacement of about 1.5 inches.

The shearing resistance plateau may result from shear plane disruptions caused by the high percentage of sand and silt in the soil. The removal of the coarse sand fraction in DS 4, leaving the fine sand and silt in the sample, appears to have had little effect on this behaviour. From subsequent reductions in shearing resistance recorded in DS 1 and DS 2, it would appear that the interference to shear plane development suggested by the resistance plateau can be overcome by increased displacements.

As previously discussed, the higher shearing resistance plateau recorded in DS 1 is attributed to the low initial water content of the soil sample.

The results of direct shear tests on Hat Creek Clay fail to adequately define the residual strength of the soil. Residual conditions were not attained in DS 1 and 2, as indicated by the decreasing shearing resistance at the end of testing. Comparing the results of DS 4 (Figure 5.17) to DS 2 (Figure 5.16), it appears that further decreases in shearing resistance would be expected in DS 4 if the test had continued and that this test also was ended prematurely.

Considering the low permeability of the soil, the length of the drainage path and the possible effects of water ingested into the shear plane during shearing, it is probable that the soil was not fully drained during direct shear tests on Hat Creek Clay.

5.4.4 Review and Evaluation of Results

The results of ring shear tests on Hat Creek Clay indicate that this soil has a residual angle of friction in the order of 21 to 25 degrees. The ring shear samples appear to have been well-drained, showing little or no variation in measured shearing resistance with increased displacement or with changes in the rate of shear. However, the maintenance of a very narrow confining ring gap, which allows the clay fines to escape but retains the sand fraction, may have resulted in a concentration of the coarse fraction in the shear plane, increasing the shearing resistance of the soil.

The direct shear tests on Hat Creek Clay were ended due to cumulative sample loss before residual conditions could be established. The rate of sample loss was very high, generally in the order of 0.1 inch loss (vertical displacement) per inch of travel, some 15 to 60 times greater than sample losses incurred in ring shear tests.

It would appear that the direct shear tests on Hat Creek Clay were not fully drained. Considering that 10 to 20 percent of the shear plane is exposed to water at each end of the direct shear traverse and that montmorillonitic clays have a low permeability combined with a high capacity to swell when exposed to water, the potential for the development of excess pore pressures by ingestion of water into the shear plane is much greater with Hat Creek Clay than with most other soils.

The sand content and low permeability of Hat Creek Clay pose formidable problems in testing for residual strength. To ensure that the virtually impermeable soil is drained, it is necessary to reduce the thickness of the sample and/or the rate of displacement. As any significant reduction in the rate of shear greatly increases the time required to achieve the large displacements needed in residual strength testing, reduced sample thickness is the preferred option. However, attempts to test thin samples of clay containing sand may be futile if sand grains lodged against the porous platens protrude through or otherwise disrupt the shear plane. Tests may also be influenced by selective loss of fines from the sample which would alter the soil composition and increase the measured shearing resistance.

5.5 RESIDUAL STRENGTH TESTS ON IRANIAN CLAY

5.5.1 Soil Description

Iranian Clay is a homogeneous, light brown, clayey silt. The soil, which was chunk sampled from a Middle Eastern desert area, is hard and brittle and has a very low natural water content.

Typical characteristics and properties for Iranian Clay are as follows (Lum and Negussey, 1977):

Specific gravity	2.78
Liquid limit	42%
Plasticity index	18%

Percent silt	70 to 75
Percent clay	18 to 30

Iranian Clay has a strong tendency to flocculate such that a suspension of the soil will settle within minutes unless mixed with a deflocculant. This tendency to flocculate may have significantly reduced the apparent clay content, tabulated above, as determined by hydrometer analysis. The agency which provided the soil has indicated that the clay portion of the soil may be largely composed of attapulgite.

5.5.2 Ring Shear Tests

Three ring shear tests were performed on Iranian Clay: RS 9, RS 10 and RS 11. Shearing resistance versus displacement plots for these tests are presented in Figures 5.18, 5.19 and 5.20. The three samples were prepared by adding water and remolding the soil prior to placement.

The initial test of the series, RS 9, exhibited very high values of side friction on the walls of the upper confining rings. Shortly after the start of testing, the total load applied through the air piston was lowered in an attempt to reduce the side friction. However, side friction remained high, in the order of 37 percent of the total applied load, and the testing was ended to prevent possible damage to the equipment.

RS 10 was marred by fluctuations in applied normal pressure which were caused by a fault in the air pressure regulator controlling the loading piston. Due to the low permeability of the soil and the short period of the pressure oscillations, undrained conditions existed

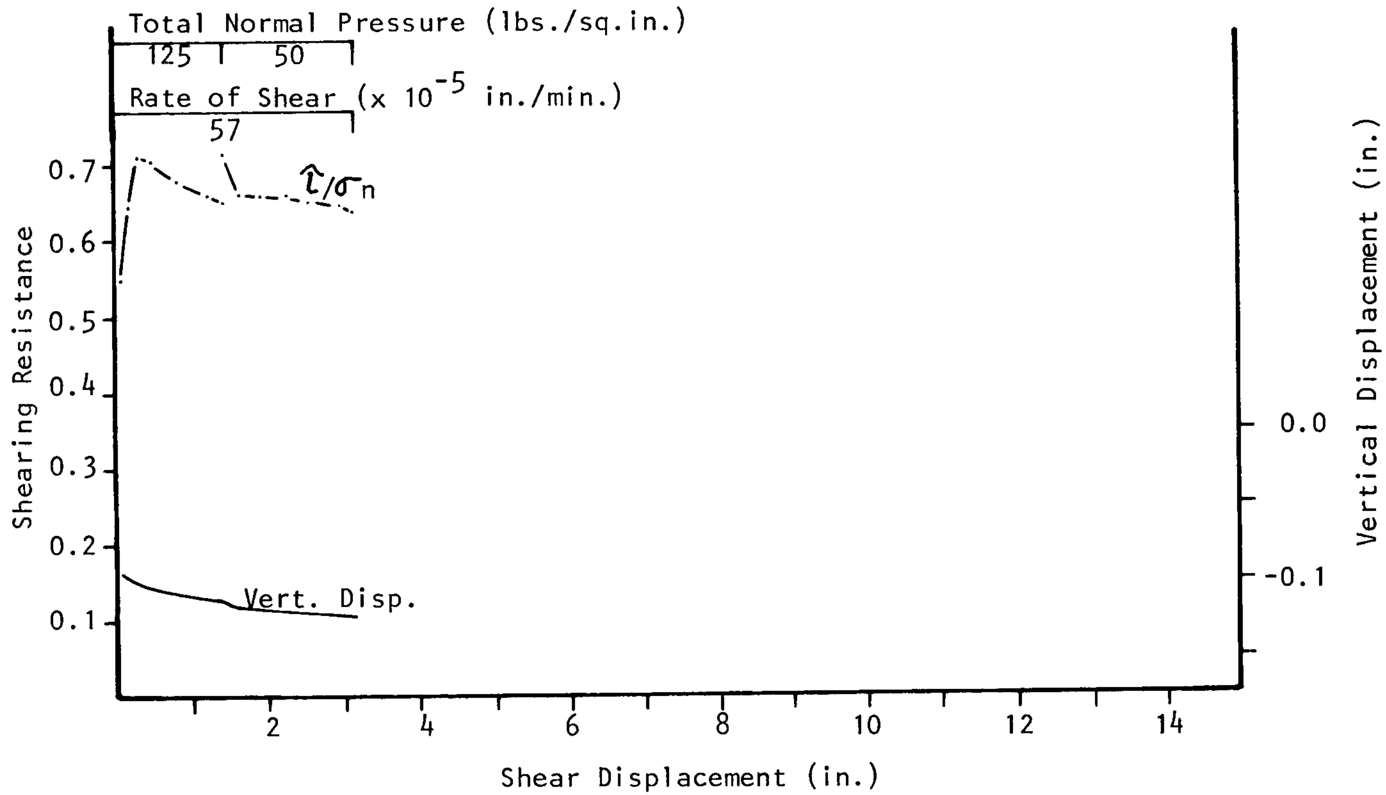


FIGURE 5.18 Shearing Resistance vs. Displacement - RS 9

Specimen: Iranian Clay

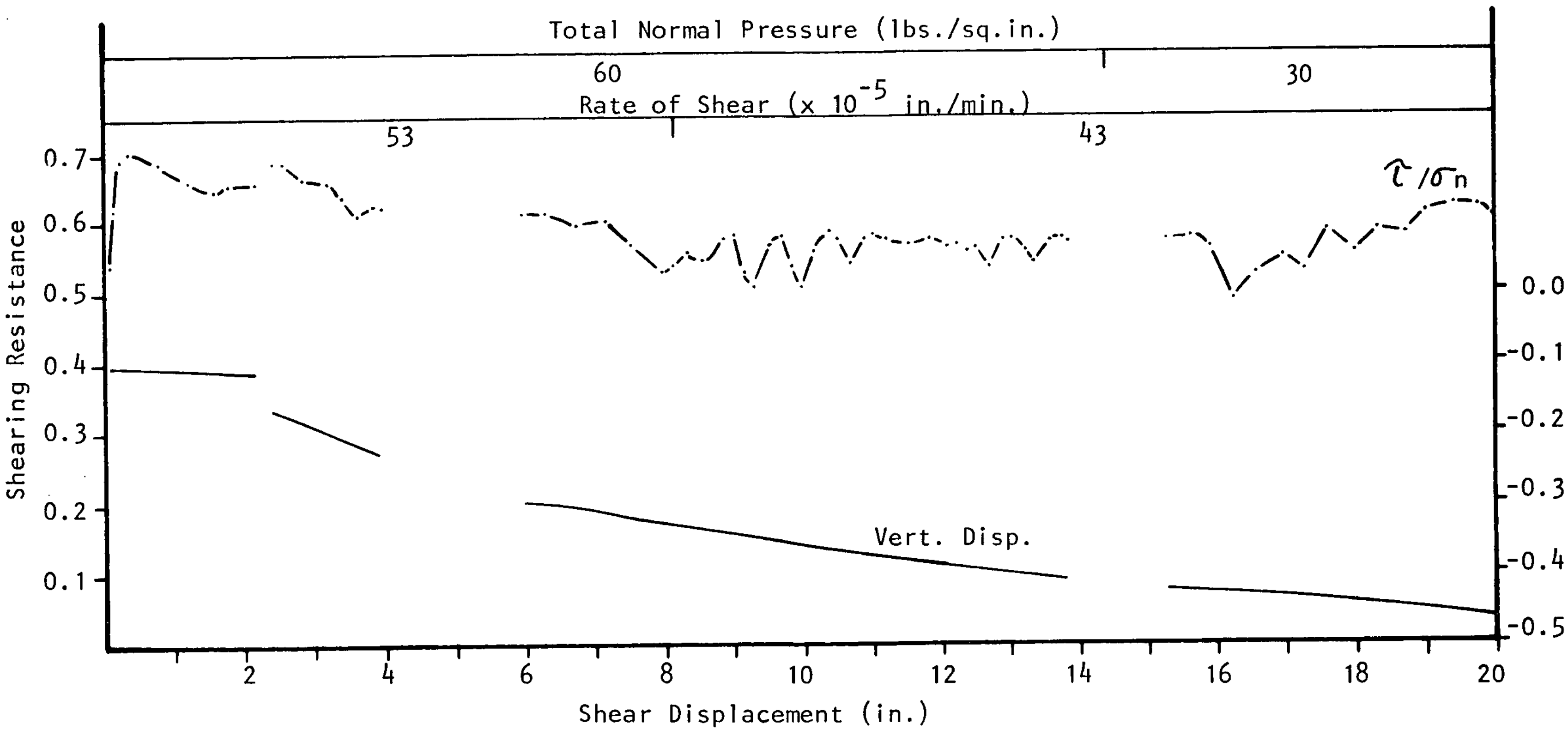


FIGURE 5.19 Shearing Resistance vs. Displacement - RS 10

Specimen: Remolded Iranian Clay

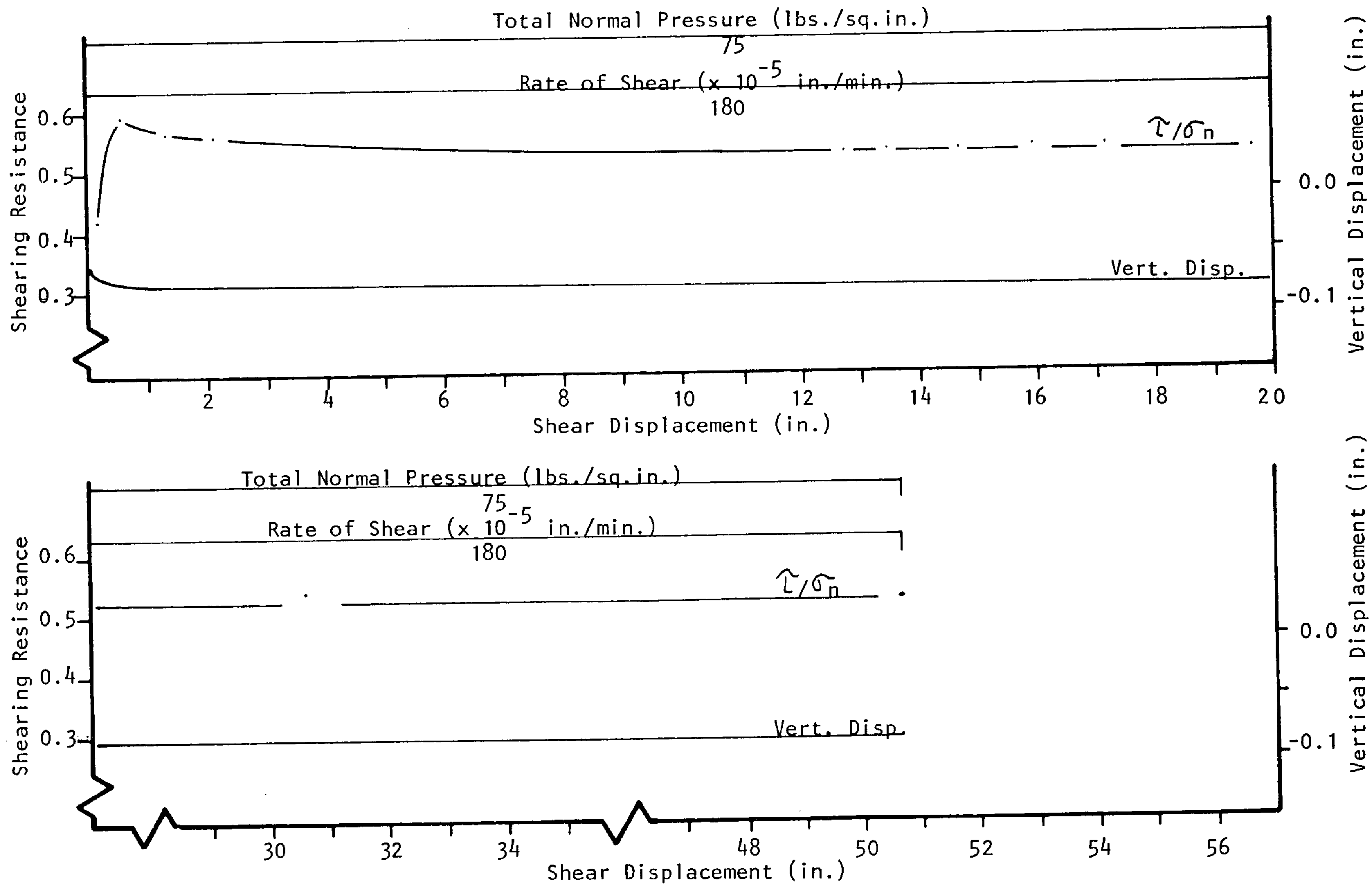


FIGURE 5.20 Shearing Resistance vs. Displacement - RS 11
Specimen: Iranian Clay

in the shear plane region of the sample such that fluctuations in the effective normal stress were small. This is demonstrated by the constancy of the measured shear strength which is a function of the effective normal stress. A plot of total applied normal stress and measured shear stress versus displacement is provided in Figure 5.21.

In ring shear testing, the test procedures are designed to create drained conditions in the sample such that the total and effective normal stresses are equal. This permits calculation of shearing resistance, the ratio of the effective shear stress to the effective normal stress, without monitoring pore pressure. When undrained conditions exist, the effective normal stress is unknown and the parameter calculated as shearing resistance is only the ratio of the shear stress to the total normal stress. This is the parameter shown on the shearing resistance plot for RS 10, Figure 5.19.

Despite the pressure oscillations, the effective normal stress within the sample can be estimated by the average total normal stress. Using this approach, the results of RS 10 indicate that Iranian Clay has a residual angle of friction of about 29 degrees, measured at an average applied normal stress of about 62.5 psi.

The final test of the series on Iranian Clay, RS 11, was hampered by a failure in the automatic data recording system. Data was acquired by manual operation of the equipment, reducing the quantity of data available for study. However, the results of this test, plotted in Figure 5.20, are consistent, indicating that Iranian

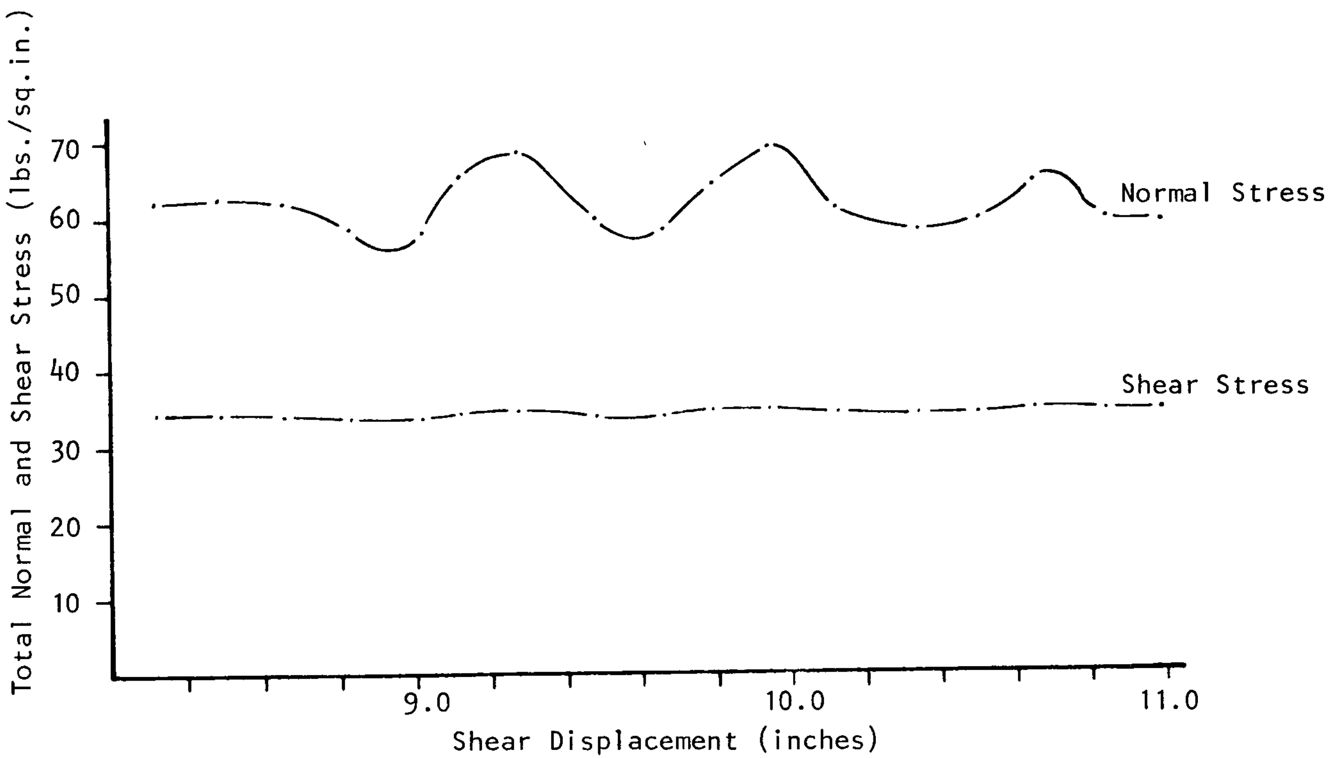


FIGURE 5.21 Total Normal and Shear Stress vs. Displacement - RS 10

The oscillations in total applied normal stress occurred as a result of a faulty air pressure regulator. The constancy in shear stress indicates that, despite total normal stress fluctuations, the effective stress on the shear plane remained nearly constant due to undrained conditions within the soil sample.

Clay has a residual friction angle of about 27.8 degrees. RS 11 was conducted at a normal stress of 75 psi.

The results of the ring shear tests on Iranian Clay, tabulated in Table 5.8, are shown on a Mohr diagram in Figure 5.22. These results are in close agreement with the accepted residual friction angle of attapulgite, 30 degrees, as tabulated in Tables 2.1 and 2.2.

TABLE 5.8 Test Results: Ring Shear Tests on Iranian Clay

Test	Stage	Normal Stress	Shearing Resistance	ϕ'_r	Shearing Rate	Cumulative Displacement
RS 9	1	125 psi	Test incomplete		5.7×10^{-4} in/min	1.40 inches
	2	60 "	Test incomplete		5.7×10^{-4} "	3.13 "
RS 10	1	60 "	Scatter due to load fluctuations (- average .555)	(29^{o})	4.3×10^{-4} "	14.40 "
	2	30 "	Test incomplete		4.3×10^{-4} "	20.12 "
RS 11	1	75 "	.529	27.9^{o}	1.8×10^{-3} "	50.68 "

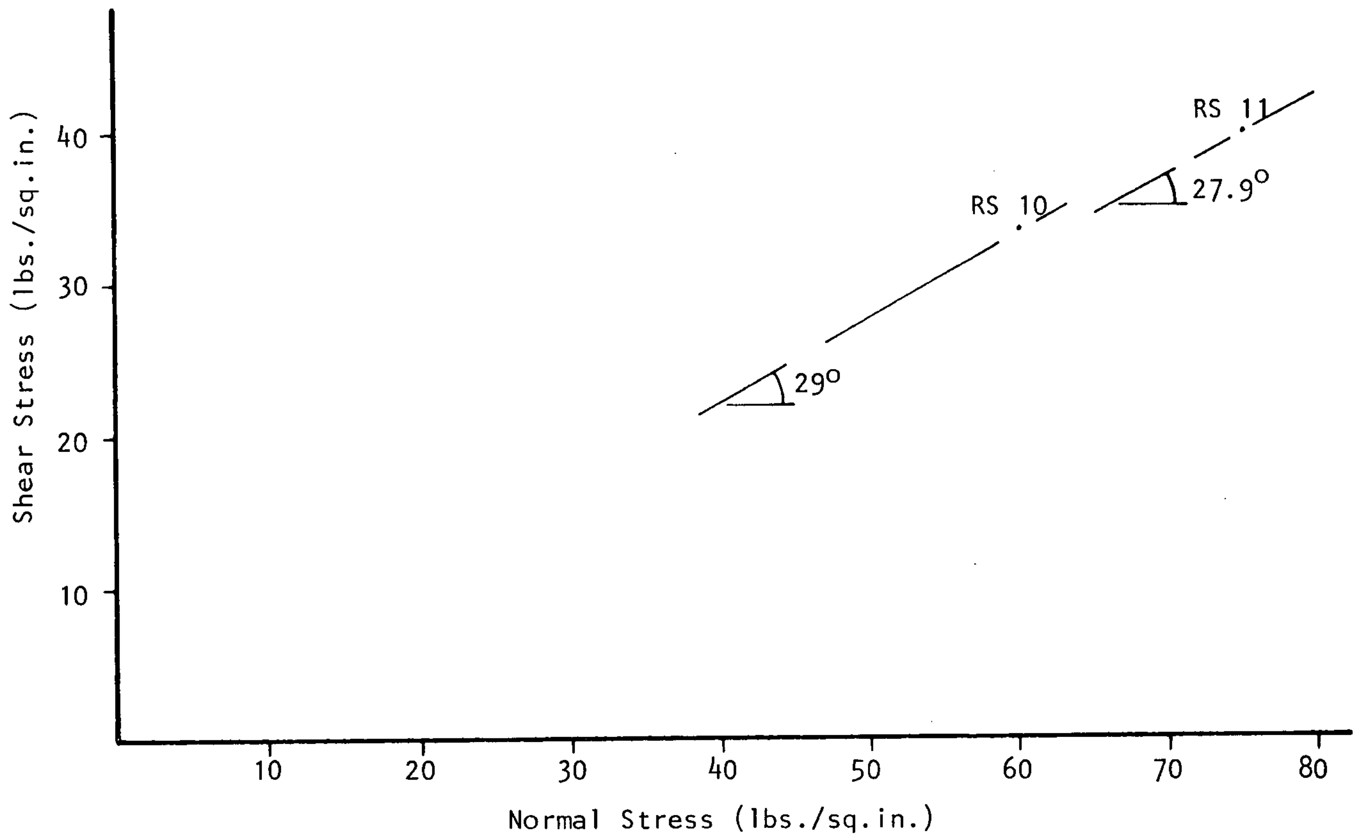


FIGURE 5.22 Mohr Rupture Diagram: Summary of Results of Residual Strength Tests on Iranian Clay

5.6 SUMMARY OF TESTING PROGRAM

Ring shear tests were conducted on three clay soils having widely contrasting properties. Multi-reversal direct shear tests were also conducted on two of the clays.

Positive determinations of residual strength were obtained in ring shear tests on Haney Clay and Iranian Clay. The results obtained in ring shear tests on Hat Creek Clay are less definitive due to the complicating effects of sand and swelling clay within the soil composition.

High values of side friction were encountered in some ring shear tests. The problem is believed to result from corrosion welts raised on the confining rings through chemical action of the soil pore fluid. Problems relating to improper calibration of transducers and malfunctions of the data acquisition unit were also encountered. Despite such considerations, the ring shear device proved simple to operate and efficient at producing consistent, easily interpreted results.

No positive determinations of residual strength were obtained in direct shear testing. All direct shear results were lower than the values obtained in the ring shear device, indicating that positive excess pore pressure existed in the direct shear samples. In most tests the apparent shearing resistance differed in forward and reverse shear directions, a variation attributed to inclination of the shear plane. Inconvenient procedures and the protracted duration of the tests added to these difficulties.

CHAPTER VI

SUMMARY AND EVALUATION

6.1 RESIDUAL STRENGTH

1. Residual strength is the lowest drained strength of a soil, attained at large shear displacements and unaffected by initial soil structure or stress history.
2. Many clayey soils, particularly if overconsolidated, exhibit a residual strength that is significantly lower than the peak value.
3. Residual strength has important engineering applications in determining stability of previously failed slopes, where residual conditions currently exist, and in evaluating the long-term stability of slopes in overconsolidated clays, in which residual conditions may develop through the mechanism of progressive failure.
4. Clay deforms in simple shear prior to reaching peak strength. Beyond peak strength, shear displacement occurs by slip along the cleavage planes of adjacent particles within shear bands located in the failure zone. Residual conditions develop as the particles in the shear bands attain parallel alignment. The preferred cleavage mode of soil influences its residual strength.

5. Shear strength in clays originates from inter-atomic bonding across solid-to-solid contacts between soil particles. Variations in the residual strength of different minerals occurs due to variations in the strength and concentration of bonding.

6. The residual strength of active clay minerals can be altered by the influence of pore water chemistry on physico-chemical interparticle forces of attraction and repulsion.

6.2 UBC RING SHEAR DEVICE

The objective of this thesis project was to design and develop a practical residual strength apparatus. Having considered alternatives, the ring shear apparatus was selected as the most practical, effective and adaptable testing device yet devised for this application. Although the direct shear and triaxial tests can be modified for residual strength purposes, the inability to obtain large uni-directional displacements and complicated or inconvenient test procedures severely limit the potential value of these methods.

The major features of the UBC Ring Shear Device are as follows:

1. The device is capable of unlimited displacements at smoothly variable rates of shear from 3.2 inches/year to 9 inches/hour.
2. The applied normal stress is smoothly variable up to 200 lb./sq.in. Higher stresses can be obtained by increasing the house line air pressure supplied to the loading piston.
3. The soil sample, with inner and outer radii of 1.75 and 2.75 inches, respectively, has a variable thickness up to a maximum of 0.75 inches.

4. The length of the basal drainage path is variable up to a maximum of 0.25 inches. The top drainage path varies in length with sample thickness.
5. Sample placement is made simple and efficient by using the upper confining rings as a mold for remolded samples and a cutting unit for undisturbed samples.
6. The confining ring gap may be set at a very narrow width without interfering with shear testing. Typical settings range from 0.002 to 0.004 inches.
7. Data acquisition units collect and record data automatically in a form that is easily adaptable to computerized data reduction.
8. Only minimal supervision is required during testing, a brief equipment check every day or two normally being sufficient to ensure smooth operation.

6.3 TESTING PROGRAM

A series of ring shear tests were undertaken on three varied clay soils as a means of evaluating the new device. Repeatable and consistent results were obtained in ring shear tests on Haney Clay and Iranian Clay, indicating that the residual angles of friction for these soils are 24 and 28 to 29 degrees, respectively. Variable results, ranging from 21 to 25 degrees, were obtained in ring shear tests on Hat Creek Clay. The variation is attributed to disruption of the shear plane by sand grains in the soil. This variation may have been aggravated by a gradual concentration of sand-sized particles in the shear plane as finer particles were selectively lost through the confining ring gap.

No determination of residual strength was obtained in the direct shear apparatus. The results were inconsistent and lower than the residual values as determined in the ring shear device, indicating that the tests were not fully drained and that positive pore pressures existed in the shear zone.

6.4 EVALUATION

The UBC Ring Shear Device provides a simple and practical method for determining residual strength. Specimen preparation is quick and convenient, while operation of the device requires minimal supervision. This versatile apparatus accomodates a wide range of sample heights, applied normal loads and rates of displacement. Test results are easily understood and interpreted. The device functions efficiently, averaging 13.5 test days per residual strength determination during the test program. Increased efficiency is anticipated as familiarity is gained with testing procedures and equipment.

The success of the UBC Ring Shear Device in determining residual strength is attributed in large measure to the natural advantages inherent in its mechanical configuration. This configuration permits unlimited unidirectional shear displacements to be applied to a soil sample of constant cross-sectional area under near-uniform stress conditions. These features provide an excellent laboratory model of the insitu field condition. Also contributing to the success of the device through enhanced practical operation are the precisely controlled confining ring gap and the automated testing and monitoring features.

While the UBC Ring Shear Device provides an efficient and effective method for determining residual strength, minor modifications will improve this capability. Problems encountered during ring shear testing include malfunctions of the data acquisition unit and excessive side friction on the upper confining rings. To avoid further malfunctions, the data acquisition unit has been replaced with a new sensitive and reliable unit. The excessive values of side friction are believed to result from corrosion welts on the surface of the confining rings. It is recommended that this problem be eliminated by replacing the existing aluminum rings with corrosion-resistant rings constructed of stainless steel or anodized aluminum.

The direct shear device proved to be an inconvenient and inadequate means of determining residual strength. The present and potential performance of the direct shear device in residual strength applications is limited by a mechanical configuration which lacks the natural advantages of the ring shear device. As a result, shear displacements must be reversed or repeated to accumulate large displacements. The stress conditions in the soil sample, which shift gradually during displacement, change abruptly with each reversal of shear direction. These frequent disruptions in applied stress and strain produce data that is commonly sparse, contradictory or difficult to interpret. However, within these limitations significant improvement in the performance of the direct shear device may be possible by restricting the motion of the upper loading frame to a single horizontal path.

As presently constructed, some vertical, lateral and rotational motions of the upper loading frame occur during testing. Modification of the device to prevent such motions, which could be accomplished by mounting the upper loading frame on a carriage, would provide precise control of the loading frame gap, reduce sample losses and permit testing of thin samples. The shortened drainage path of thin samples would improve sample drainage, providing greater confidence in the test results, and would probably reduce the duration of testing. Restricting the motion of the upper loading frame would also inhibit shear plane inclination which hampered many of the direct shear tests carried out during this study. Due to the extended duration of testing, further modifications to fully automate the direct shear device would be essential for convenient and practical operation.

Modification of the direct shear apparatus would improve its capability for determining residual strength. However, data would remain sparse and cumbersome to interpret due to the incremental approach to cumulative displacements employed in this test, and the duration of testing would remain extended as a result of shear plane disruptions at each reversal of shear direction. Therefore, the ring shear device will continue to provide the most practical, convenient, efficient and effective method for determining residual strength.

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APPENDIX I

DERIVATION OF EQUATION FOR DETERMINING $\tan \phi'$

Let τ = uniform shear stress over shear plane

σ = uniform normal stress

$\tan \phi' = \tau / \sigma$

r_1 = inside radius of soil sample

r_2 = outside radius of soil sample

M = sum of moments over shear plane as measured by force transducers

W = total normal load on shear plane

$$M = \tau \times \text{Sample Area} \times \text{Moment Arm}$$

$$= \int_{r_1}^{r_2} (\sigma \tan\phi')\,(2\pi r\,dr)\,(r)$$

$$= 2\pi\sigma\tan\phi' \int_{r_1}^{r_2} r^2\,dr$$

$$= 2\pi\sigma\tan\phi' \; \frac{r_2^3 - r_1^3}{3}$$

Therefore:

$$\tan\phi' = \frac{3M}{2\pi\sigma(r_2^3 - r_1^3)}$$

$$= \frac{3M}{2(r_2^3 - r_1^3)} \times \frac{(r_2^2 - r_1^2)}{\sigma\pi(r_2^2 - r_1^2)}$$

$$= \frac{3M}{2W} \; \frac{r_2^2 - r_1^2}{r_2^3 - r_1^3}$$

$$= \frac{3M(r_1 + r_2)}{2W(r_1^2 + r_1 r_2 + r_2^2)}$$

APPENDIX II

DIRECT SHEAR APPARATUS

The direct shear apparatus used in this study is a standard manual drive unit with a 2 by 2 inch shear box that has been fitted with a variable speed motor drive. The length of the shortest drainage path, from the porous base to the shear plane, is 3/8 inches.

Initially the device had no self-stopping mechanism and was operated only during hours when an attendant was available. The attendant was required to observe and record load and displacement data, to maintain and adjust the gap between the upper and lower loading frames of the shear box, and to reverse the direction of travel when necessary. During Direct Shear Test 3 (DS3), the apparatus was fitted with a micro-switch system to stop the motor drive at the desired end of travel. The self-stopping mechanism enabled a more continuous operation of the equipment to obtain large displacements in shorter time, but an attendant was still required to observe and record data, adjust the gap, and reverse the direction of travel. More recent modifications have fully automated the drive system and data collection, although periodic manual adjustments of the loading frame gap are still required.