Time-effect and instability behaviour of cohesive soils

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2008

Manish Tiwari. (2008). Time-effect and instability behaviour of cohesive soils. Doctoral thesis, Nanyang Technological University, Singapore.

<https://hdl.handle.net/10356/41307>

<https://doi.org/10.32657/10356/41307>

TIME-EFFECT AND INSTABILITY BEHAVIOUR OF COHESIVE SOILS

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A thesis submitted to the Nanyang Technological University
in fulfilment of the requirement for the degree of
Doctor of Philosophy

2008

This thesis is dedicated to my beloved wife, Shobhana, who has unconditionally and tirelessly helped and supported me during all the phases of the research.

Acknowledgements

The research work presented in this thesis was carried out under the supervision of Associate Professor Chu Jian. His continued support and encouragement was a great source of inspiration in going through various phases of this research work. He was always available for help, be it a work day or a weekend. I gratefully acknowledge his generosity in giving me the time whenever I needed it.

I also sincerely thank Assoc. Prof. Teh Cee Ing for his very kind support and enthusiastic encouragements during my meetings with him over the years.

I would also like to express my gratitude to all the technical staff of Geotechnics labs. Special thanks are due to Eugene and Han Guan for there tremendous help in repairing electronic parts and fabricating various things respectively.

I am grateful to my research colleagues Huili, Darek, Chin Liang and Inge for being so helpful and passing me the knowledge about setting up the triaxial cell and conducting the experiments. Thanks are also due to Miss Meng Guanghui for assisting me in various ways during the sample preparation and experiments.

I am thankful to all my colleagues in my office, Room 6, for being so friendly and lively, and thus making my stay in NTU memorable and cherishable.

I am grateful to Dr. P.V. Krishnan, on whose advice I opted to pursue PhD at NTU. His encouragement during all the phases of my PhD was invaluable. Thanks are due to my friends, Dr. Akshay Naidu, Dr. Ankush Mittal, Dr. Amit Kumar Gupta and Dr. Sujoy Roy, for their continued support.

I would also like to express my gratitude to Nanyang Technological University for granting the scholarship for this research program.

I am grateful to my parents, brother, sister and my parents-in-law who were constant source of inspiration for me and helped me to go forward through their love and care.

Last but not the least, I am grateful to my beloved wife, Shobhana, for her continuous support and encouragement through the thick and thins of research over the last four years of the research work, especially during the difficult phases of writing and numerous revisions of the thesis.

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SUMMARY

The stress-strain behaviour of cohesive soils is known to be affected significantly by time-effects such as shearing rate effects, creep and stress relaxation. Shearing rate may affect the shear strength and friction angle of soil. However, the mechanism behind the rate-effects is not well understood. The effect of loading mode, i.e. whether the test is conducted in load-controlled or deformation-controlled loading mode, has been seldom studied despite the fact that most of the field loading conditions are load-controlled, whereas the laboratory tests are mainly carried out using deformation-controlled loading mode. Undrained creep in clay, which is one of the major time related problems, is often studied in terms of axial strain rate at a given deviator stress. However, the variation of pore water pressure and effective stresses during undrained creep should be studied as well. Whether the instability will occur at a pre-failure state similar to that observed for sand is not yet known. It should be particularly mentioned that the study of time-effects in triaxial tests on Singapore marine clay has never been reported.

An experimental study of time-effect on cohesive soils was carried out. Three types of clays, namely undisturbed Singapore marine clay, reconstituted Singapore marine clay and Kaolin were used for the study. To ensure proper estimation of effective stresses during shearing, the pore water pressure was measured at the top, bottom and mid-height of the specimen. A motorized triaxial cell capable of running tests in both load-controlled and deformation-controlled loading modes was used for conducting the tests on normally consolidated specimens of the clay. The majority of tests were carried out on isotropically consolidated specimens of clay. The tests conducted included, undrained and drained compression tests at various shearing rates and in deformation-controlled and load-controlled loading modes. Undrained creep tests were also conducted.

One log cycle (or ten-fold) increase in shearing rate caused the undrained shear strength to increase by 6%, 5% and 2% respectively for undisturbed marine clay, reconstituted marine clay and Kaolin. The pore water pressure was found to decrease with the increase in shearing rate in both load-controlled and deformation-controlled

loading modes. This resulted in an increase in mean effective stress, which in turn resulted in an increase in peak deviator stress. Therefore, the rate effect resulted in different effective stress paths for undrained tests at different rates. For Kaolin and reconstituted marine clay, the failure occurred at higher deviator stress and higher mean effective stress in tests conducted at faster rates such that the failure point reached a higher point on the failure envelope. Thus the effect of shearing rate on the failure envelope was small. For undisturbed marine clay, the failure in faster tests occurred at effective stress states earlier than that defined by the effective failure envelope, implying that the effective friction angle is lower in faster tests.

The undrained shear strength and excess pore water pressure in undisturbed Singapore marine clay were not significantly affected by the loading mode when time taken to reach the peak deviator stress were the same in deformation-controlled and load-controlled tests. Instability occurred in load-controlled tests at the peak deviator stress. On the other hand, strain softening occurred in the form of gradual decrease in deviator stress in deformation-controlled tests. Loading mode had larger effect on the undrained shear strength of reconstituted marine clay and Kaolin. The undrained shear strengths in load-controlled tests were higher than the shear strengths in deformation-controlled tests by up to 8% and 5% respectively for reconstituted marine clay and Kaolin. Correspondingly lower pore pressures were recorded in load-controlled tests.

Drained behaviour of undisturbed and reconstituted marine clay was also affected by the loading rate and loading mode. At strain rates higher than the maximum permissible strain rate (determined using BS1377, 1990), an increase of 5 times in strain rate resulted in up to 15% lower drained shear strength and lower effective friction angles. This was due to the large non-uniformity of pore pressure in drained tests conducted at faster rates, which resulted in a partially drained condition. The method of using isotropic consolidation data for selecting the strain rate for drained test (BS1377, 1990) sometimes underestimates the time to failure, such as that observed for undisturbed Singapore marine clay. It was observed that a better way to select the shearing rate is to get an estimate of strain rate using the isotropic consolidation data (BS1377, 1990) and use mid-height pore pressure measurement to verify the uniformity of pore pressure.

Pre-failure instability, identified by a rapid increase in axial strain and axial strain rate, was observed to occur during undrained creep tests in undisturbed marine clay. A zone of potential instability could be defined, which was bound by the instability line and the failure envelope of soil. Instability line coincided with the line passing through the peaks of the effective stress paths of undrained compression tests. When the effective stress state was below the zone of potential instability, instability did not occur. However, when the effective stress state was within this zone, instability occurred in most cases. If the problem of undrained creep is foreseen in field loading conditions, use of failure envelope determined from the maximum q/p' may results in unconservative design. For reconstituted marine clay and Kaolin, which did not show pre-failure strain softening, the instability occurred on failure and the instability line coincides with the failure envelope. This indicates that pre-failure strain softening is a necessary condition for pre-failure instability to occur.

It was observed from the tests that the use of mid-height pore pressure measurement gives more consistent results. The top and bottom pore pressure transducers tend to over-predict the pore water pressure, especially in fast tests and an effective stress interpretation of the undrained test results is difficult when only top and bottom pore pressure measurements are used. In both undrained and drained tests, the mid-height pore pressure measurement is essential for monitoring the equalization of pore pressure within the specimen and to avoid the use of inappropriately high shearing rates.

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LIST OF SYMBOLS AND ABBREVIATIONS

BS	British Standard
CAU or CAUC	Anisotropically Consolidated Undrained Compression
CIDC/ CID/ CD	Isotropically Consolidated Drained Compression test
CIUC or CIU	Isotropically Consolidated Undrained Compression test
CIUE	Isotropically Consolidated Undrained Extension test
CR	Undrained Creep Test
CSD	Constant Stress Drained
DC	Deformation Controlled
DPVC	Digital Pressure Volume Controller
Fig.	Figure
Figs.	Figures
I	Onset of instability
KCU	Isotropically Consolidated Undrained Compression on Kaolin
KCRE	Undrained Creep in Extension on Kaolin
KCSD	Constant Stress Drained test on Kaolin
KCSDE	Constant Stress Drained test in Extension on Kaolin
kPa	Kilo Pascal
KUE	Isotropically Consolidated Undrained Extension on Kaolin
LC	Load-controlled test
LVDT	Linear Variable Differential Transducer
M	Slope of failure line in q-p' plane
min	Minutes
N/min	Newton per minute
OCR	Overconsolidation Ratio
PWP	Pore Water Pressure

p'	Mean Effective Stress, $(\sigma_1' + 2\sigma_3')/2$
p_0'	Initial Effective Confining Stress or Consolidation Pressure
q	Deviator Stress, $(\sigma_1' - \sigma_3')$
$(q/p')_f$	q/p' at failure
q_m or $q_{\max}$	Peak Deviator Stress
RCU	CU Test on Reconstituted Marine Clay
RCD	CD Test on Reconstituted Marine Clay
SB	Shear Band
SYS	Static Yield Surface
s_u	Undrained Shear Strength
t	Time
t_f	Time to Failure
UU	Unconsolidated Undrained
Y	Yielding Point
ε_1	Axial Strain, $(\Delta H/H)$
ε_v	Volumetric Strain $(\Delta V/V)$
$\dot{\varepsilon}'$ or $\dot{\varepsilon}$	Axial Strain Rate
η_c	Slope of q - p' curve (q/p')
x	Point at Shear Band
σ_1'	Effective Normal Stress
σ_3	Cell Pressure
σ_3'	Effective Confining Stress
σ'_{vc} or σ'_p	Pre-consolidation Pressure from Oedometer Test
Δu	Change in Pore Water Pressure

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

A large area of Singapore is underlain by thick deposits of marine clay up to 60m thick. This often poses problems of long-term settlement and stability. The stress-strain behaviour of the cohesive soils can be affected significantly by time-effect such as shearing rate effects and creep (Singh and Mitchell, 1969; Campanella and Vaid, 1974; Augustesen et al., 2004). If these shearing rate effects are not taken into consideration, the geotechnical design can be affected and sometimes lead to unsafe design.

Studies on time-effect on the stress-strain behaviour of cohesive soils have been carried out in terms of shearing rate effects, creep and stress relaxation (Casagrande and Wilson, 1951; Finn and Shead, 1973; Vaid and Campanella, 1977; Lefebvre & LeBoeuf 1987; Sheahan, 1994; Sheahan et al., 1996; Augustesen et al., 2004). It has been observed by most of these researchers that the undrained shear strength of clay may increase significantly due to the increase in shearing rate. This means that the undrained shear strength (c_u), which is commonly determined using fast undrained compression tests in laboratory (typically, 0.5 to 1 mm/min), may not be fully mobilized in slow loading conditions such as the construction of large structures, stage excavation etc. This may lead to a reduction in factor of safety. On the other hand, in cases of rapid loading such as sea-wave loading, use of c_u from conventional tests may lead to over-conservative and expensive design.

Although some researchers attributed the shearing rate effects to the change in pore pressure upon the change in shearing rate (Richardson and Whitman, 1963; Sheahan et al., 1996), others (Blight, 1965; Akai et al., 1975) did not observed much effect of shearing rate on the pore pressure and some (Lefebvre and LeBoeuf, 1987) attributed the changes in pore water pressure to the errors in pore water pressure measurement. Sheahan et al. (1996) emphasized the need for further study of the shearing rate effect on cohesive soils with proper pore pressure measurement. The difference of opinions still remains on the possible reasons for the shearing rate-effect exhibited by the cohesive soils.

When a soil specimen is sheared in the laboratory, it can be sheared in a load-controlled loading mode or in a deformation-controlled loading mode. Loading mode is another factor affecting the stress-strain behaviour of cohesive soils. Although the field loading conditions; such as the construction of a building, most pile-load tests, application of surcharge after installation of PVD etc.; are usually load-controlled, most of the studies of the stress-strain behaviour of cohesive soils in the laboratory are carried out using deformation-controlled tests. The studies of the loading mode effects on the behaviour of clay are very limited. Considerable differences between the critical void ratio lines for load controlled and deformation-controlled tests have been observed for sand (Hird and Hassona, 1990). Load-controlled tests were found to give up to 20% higher shear strength than the deformation-controlled tests (Lacasse, 1990) on clay in rapid tests (shearing to failure within 5 minutes). Vaid (1988) reported the effect of loading rate in load-controlled tests but did not compare the results from deformation-controlled tests with the results from load-controlled tests. The effect of the loading mode on the behaviour of cohesive soils is not yet understood.

Cohesive soils are known to undergo creep-induced deformations at constant load, e.g. in excavations opened for long time, under the structural load after completion of the construction work, during surcharging, in pile load tests etc. The creep behaviour is often studied in terms of the variations of axial strain and axial strain rate and the strength loss with time during creep (e.g. Mitchell et al., 1968; Vaid and Campanella, 1977; Kavazanjian and Mitchell 1980; Kuhn and Mitchell 1993; Zhu et al., 1999). In all these studies, the axial strain rate has been found to increase rapidly near the

failure. A similar process has been observed for sand, in which the specimen can become unstable at an effective stress state below the failure envelope (Lade, 1994; Chu and Leong, 2002; Chu et al., 2003). Whether such pre-failure instability behaviour can also occur in cohesive soils, has not been established. Creep rupture, which refers to the failure of the soil specimen by rapid increase in axial strain and axial strain rate under constant deviator stress conditions, has been studied for clay before. However, creep rupture was studied mainly in terms of total stresses. Whether creep rupture is a type of failure before the effective stress state reaches the failure state is yet to be established. Therefore, there is a need to study further the creep rupture behaviour of cohesive soils with particular attention paid to the variation of effective stresses.

Although Singapore marine clay has been encountered in many important projects in Singapore, time-effect on its behaviour in triaxial tests has not been studied. A study on the time-effect of Singapore marine clay will not only enhance the understanding of the fundamental behaviour of cohesive soils, but also provide valuable information on geotechnical practice in dealing with marine clay.

1.2 OBJECTIVES OF RESEARCH

The objectives of this study are:

1. to study the shearing rate effect on the stress-strain behaviour of cohesive soils with particular reference to the pore water pressure and the effective stress changes;
2. to study the effects of loading mode, namely load-controlled or deformation-controlled loading modes, on the strength and stress-strain behaviour of cohesive soils;
3. to study the effect of creep of cohesive soils on the strength and deformation behaviour of clay under undrained conditions and to attempt to establish the mechanism of creep rupture.

1.3 SCOPE OF STUDY

The proposed study was conducted on three types of cohesive soils, namely, undisturbed Singapore marine clay, reconstituted marine clay and Kaolin. The three clays, which have different structures and permeability, were subjected to similar tests to examine the role of structure and permeability of the clay in observed time-effect. A new motorized triaxial cell was used to study the stress-strain behaviour of the three soils under drained and undrained conditions. The tests were carried out in both load-controlled and deformation-controlled loading modes to study the shearing rate effects on the stress-strain behaviour in undrained compression tests. Results of deformation-controlled tests were compared with the results of load-controlled tests to study the effect of loading mode on the stress-strain behaviour. Undrained creep tests were carried out with the pore pressures measured at the top, bottom and mid-height of the specimen to study the variation of effective stresses during undrained creep. The conditions under which the specimen became unstable were established. Drained compression tests were also conducted under different shearing rates in both load-controlled and deformation-controlled loading modes to examine the effect of loading rate and loading mode on the drained behaviour of cohesive soils.

1.4 OUTLINE OF THESIS

The thesis is divided into eight chapters. Chapter 2 presents a review of the previous studies on the effects of shearing rate and loading mode on the undrained and drained compression behaviours of cohesive soils. A review of undrained creep behaviour of cohesive soils is also conducted. Experimental set-up, soils tested and methodology adopted to conduct the tests is presented in Chapter 3. The stress-strain behaviour of undisturbed Singapore marine clay under various shearing rates and loading modes is discussed in Chapter 4. The undrained creep behaviour of undisturbed Singapore marine clay is investigated in Chapter 5. The effect of shearing rate and loading modes on the undrained and drained behaviours and undrained creep behaviour of reconstituted marine clay is studied in Chapter 6. Chapter 7 presents the study of shearing rate and loading mode on the undrained behaviour of Kaolin along with a comparison of the behaviour of Kaolin with the behaviours of undisturbed and reconstituted marine clays. Finally, the conclusions, practical implications and recommendations for future study are discussed in Chapter 8.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

One of the stress-strain characteristics of clays is time dependence. The time-effect on clay is often studied in terms of strain-rate effects, undrained creep and relaxation. Numerous studies have been carried out in the past to study the time-effect on clays. A literature review is presented in this chapter to summarize the important aspects of time-effect on clays and to identify the aspects that are still open to questions.

This chapter is divided into five sections. Before discussing the time-effect, namely, rate effect, undrained creep and constant-shear-stress-drained behaviour, general behaviour and yielding of cohesive soils is discussed in section 2.2. Section 2.3 presents the review of the rate effects and effect of loading mode on the stress-strain behaviour of cohesive soils. Section 2.4 presents the generalized undrained creep behaviour and section 2.5 presents the constant-shear-stress-drained (CSD) behaviour of clay. The important issues related to the study of undrained creep behaviour of clay in the light of previous works by other researchers are highlighted.

2.2 STRESS-STRAIN BEHAVIOUR OF COHESIVE SOILS

Cohesive soils may be defined (Bishop 1971) as those in which inter-particle forces or bonds make a significant contribution to the mechanical behaviour of the soil. This category includes clays, clayey silts, boulder clays and tills, clay shale and chalk. The study in this research is limited to clay, both undisturbed and

reconstituted. The most marked characteristic of many cohesive soils is that the destruction of the cohesive bonds and the structure of undisturbed samples by strain or remoulding is irreversible (Wood 1990). The behaviour of the clay in nature, therefore, cannot be simulated truly by tests on reconstituted samples. Nevertheless, since it is difficult to get good quality undisturbed clay samples, most of the previous studies have been performed on reconstituted clay (Roscoe et al. 1963; Ladd 1974). These studies reflect to a certain extent the average behaviour of clay, which may be applied to natural clay by incorporating some additional factors.

In classical soil mechanics, the problems of stress distribution and deformation (Fig. 2.1b) have generally been solved by assuming the soil to be a homogeneous, linear elastic material (OA in Fig. 2.1a), where properties are defined by a single deformation modulus (Wood 1990). The problems of stability or limiting equilibrium (Fig. 2.1c) are solved on the basis of rigid plastic (or perfectly plastic) soil (BC in Fig. 2.1a), where properties are defined by a single value of strength (for a given effective normal stress). However, most real soils depart from those two idealizations. With a few exceptions real soils do not continue to yield at a constant stress (such as BC in Fig. 2.1a) after the point of failure is reached. A marked decrease in strength occurs (X to Y) with the increasing strain until the stress stabilizes (in deformation-controlled tests) at an ultimate or residual value (at point C) (Wood 1990; Atkinson 1993). In clays, the strain necessary to reach the residual state may be very large (Bishop 1971). In load-controlled tests, sudden collapse of soil after peak is observed (Fig. 2.2) and post peak behaviour is difficult to interpret (Lundgren 1968).

Fig. 2.1: (a) Observed and idealized shearing behaviour of soil for (b) settlement and (c) stability calculations (after Wood, 1990)

Fig 2.2: Sudden failure during load controlled or stress controlled test (after Lundgren, 1968)

2.2.1 Yielding

Yielding is associated with a sharp change from elastic to plastic behaviour (Smith et al., 1992). Wood (1990) defined yielding as ‘the departure from stiff elastic response that occurs as reloading proceeds beyond the past maximum load’. Soils, in common with a number of engineering materials, exhibit a stress-strain response as shown in Fig. 2.3 (Parry & Wroth 1982; Wood 1990; Atkinson 1993).

It can be seen from Fig. 2.3 that when the soil is loaded from a point A at zero strain, a sharp turn in the stress-strain curve occurs at a point Y, which is called the point of yielding. Unloading at a point such as B will produce a curve BC, with the closed stress cycle A to B and B to C causing a permanent or plastic deformation AC. On reloading from C, a path similar to BC is retraced, but with a small hysteresis loop, until a point D, close to B, is reached. At D, an abrupt change in the stress-strain curve is observed, subsequent to which the extended virgin loading curve DE is followed. The point D indicates the yield of the material for the reloading curve CDE; it marks a major change in the behaviour in that the local tangent stiffness is greatly reduced and further plastic strains are induced.

In Fig. 2.3 the second yield point D is higher than the first yield point B. This increase in yield point due to the plastic straining is called *Strain Hardening* (Schofield and Wroth 1967; Wood 1990), and the relationship between the increase in the yield stress, $\delta\sigma'$, and the plastic straining $\delta\varepsilon^p$ is known as the *Hardening Law*.

Fig. 2.3: A typical loading-unloading curve demonstrating the yielding of soil

Yielding and plastic straining may also cause *softening* as shown in Fig. 2.4. Softening or strain softening refers to the phenomenon wherein the shear resistance of soil element decreases with further shearing (e.g. DE in Fig. 2.4) after the deviatoric stress has passed a peak in the stress-strain curve. In each case, the total strains are the sum of the elastic and plastic components.

Fig. 2.4: Strain softening (after Atkinson, 1993)

The successive yielding and ultimately failure can be represented by a series of yield curves and a failure envelope respectively, as shown in Fig. 2.5a (Atkinson 1993, Schofield and Wroth 1968). Plastic strains occur once the stress state moves out of the region bounded by a yield curve (Fig. 2.5b). It is assumed that, irrespective of the stress path by which a new yield surface is created, its shape remains the same. In other words, subsequent yield surfaces always have the same shape. The vectors of plastic strains are normal to the yield curves and failure envelope. This is known as the *Normality Condition*. The relationship between the yield surface and the direction of plastic strain vector is called the *Flow Rule*. Another way of describing the flow rule for plastic straining is to define a potential envelope that is orthogonal to all the vector of plastic straining. If the yield surfaces and plastic potential are identical, then the soil is said to obey the postulate of normality or *associated flow rule* since the nature of plastic deformations, or flow, is associated with the yield surface of soil.

Fig. 2.5: (a) Successive yield curves (b) Plastic strains on yield surface (after Atkinson, 1993)

2.2.2 Yielding, Failure and Instability

It should be emphasized that the ‘yielding’ is not the same as ‘failure’. Depending on the precise stress history, a specimen may indicate a succession of yield states before (or after) it fails in the sense of maximum shear stress or maximum shear stress ratio is reached. In general, yielding is associated with a transition from stiff to less stiff response as shown earlier in Fig. 2.3. According to the critical state soil mechanics, failure refers to the state when the critical state (q_f in Fig. 2.6) is reached (Atkinson, 1993). Critical state is reached when the soil specimen continues to deform at constant shear stress, constant normal stress and constant volume. For the soils, which exhibit strain softening, critical state occurs at maximum q/p' ratio while for other soils, critical state occurs at peak deviator stress. Nevertheless, in common geotechnical design the failure is said to have occurred when the peak strength is achieved.

Instability has been defined for granular materials (Lade, 1994) as a condition in which large plastic strains are generated rapidly due to the inability of a soil element to carry or sustain a given load. By definition, instability can occur only in load-controlled shearing mode. Conventional shearing is typically carried out in deformation-controlled (or strain-controlled) loading mode, in which the specimen is deformed at a fixed rate. So the axial strain increases linearly at a pre-determined rate (Fig. 2.7a). During a deformation-controlled test, specimen continues to deform

under constant strain rate after the peak is reached but there may be a marked decrease in the deviator stress (Fig. 2.7b) (Wood 1990). The strain softening is generally observed and the stress state may reach a residual state after which there may be no significant drop in deviator stress. Load-controlled test refers to a test in which a certain load increment rate is imposed on the specimen. The variation of axial strain in load-controlled test is non-linear, as schematically shown in Fig. 2.7a. The strain rate is small in the beginning of shearing. However, as the load increases, the strain rate also increases continuously till the failure stress is reached. In a load-controlled test, specimen collapses after the peak deviator stress as shown in Fig. 2.7b and is accompanied by a sudden drop in q and rapid increase in axial strain rate, or instability.

Fig. 2.6: Failure in terms of critical state

Fig. 2.7: Schematic diagrams showing (a) comparison of axial strain variation with time & (b) stress-strain curves, for load controlled and strain controlled tests

Fig. 2.8: Comparison of effective stress paths in load-controlled and deformation-controlled tests

A comparison of typical effective stress paths in load-controlled and deformation-controlled tests is shown in Fig. 2.8. Since the deviator stress drops rapidly after the peak q due to the collapse of the specimen in load-controlled tests (Fig. 2.8, LC test), it is expected that there will be no significant strain softening before failure in terms of maximum q/p' . Whether the failure or collapse of the specimen in load-controlled tests will occur below the failure envelope at maximum q/p' (as determined from deformation-controlled tests) is not known. The difference between the failure envelopes in load-controlled and deformation-controlled tests is seldom discussed. Failure envelope is always determined from deformation-controlled tests. A comparative study of the failure states in load-controlled and deformation-controlled tests will be done in this thesis.

2.3 SHEARING RATE EFFECT ON THE BEHAVIOUR OF COHESIVE SOILS

The effect of shearing rate on the undrained and drained behaviour of cohesive soils, either reconstituted or undisturbed, has been studied by numerous researchers over the years (Casagrande and Wilson 1951; Gibson 1954; Richardson and Whitman 1963; Blight 1965; Saada 1967; Lundgren et al. 1968; Roy and Sarathi 1976; Vaid and Campanella 1977; Graham et al. 1983; Leroueil et al. 1985; Lefebvre & LeBoeuf 1987; Lacasse 1995; Shibuya et al. 1995; Sheahan et al. 1996; Penumadu

and Chameau 1997; Hinchberger and Rowe 1998; Zhu et al. 1999; Lo Presti et al. 1999; De Magistris et al. 1999; King et al. 2000; Teachavorasinskun et al. 2002; Katti et al. 2003). The cohesive soils have been found to be highly rate sensitive (Arulanandan et al. 1971; Hicher 1988; O' Reilly et al. 1988). The undrained shear strength of most of the clays, both undisturbed and reconstituted, has been found to increase with the increase in shearing rate, although some clays are affected much lesser (Fourie and Dong 1991) than the others. Secant modulus of cohesive soils has been found to increase with the increase in shearing rate (Teachavorasinskun et al. 2002). Preconsolidation pressure was also found to increase with the shearing rate (Graham et al., 1983; Leroueil et al. 1985). There have also been a limited number of studies on the rate effect on the drained behaviour of cohesive soils (Gibson and Henkel, 1954; Roy and Sarathi, 1976; Newson et al. 1997). However, the majority of the studies have been limited to the undrained behaviour because of the fact that cohesive soils are mostly subjected to undrained loading conditions in the field and undrained strength is more critical for the stability analysis. Many researchers have concluded that the rate-effect on undrained shear strength and secant modulus is caused by changes in pore water pressure due to the change in shearing rate (Richardson and Whitman, 1963; Lefebvre and LeBoeuf, 1987; Sheahan et al., 1996). However, there remain the differences of opinions (Blight, 1965; Akai et al., 1975). Blight (1965) believed that the strain rate effects and effects resulting from non-uniform pore water pressures are two separate phenomena. Akai et al. (1975) reported that there was no effect of strain rate on the pore water pressure developed in Fukakasa clay although the undrained shear strength was affected significantly. Sheahan et al. (1996) argued that the decrease in excess pore water pressure with increase in rate may represent a uniform measurement error and without special measurement equipments and proof testing, measured pore water pressure at faster rates will lag behind the true value.

While discussing the rate-effect on the behaviour of soils, the study of the effect of loading mode, i.e. whether the axial load is applied in a deformation-controlled or load-controlled mode, becomes necessary since the strain rate variations are different in load-controlled and deformation-controlled loading modes. Nevertheless, studies of effect of loading mode have been rather few (Lundgren et al. 1968, Vaid 1988,

Lacasse 1995). Lacasse (1995) reported an increase in undrained shear strength in stress controlled loading as compared to the deformation-controlled loading.

A detailed literature review on the effect of loading rate and loading mode on the undrained and drained behaviours of cohesive soils is presented in the following sections.

2.3.1 Rate Effects on the Undrained Behaviour of Cohesive Soils

2.3.1.1 Early Studies of Rate-effect (1950s and 1960s)

The effect of shearing rate on the behaviour of cohesive soils has been studied by many researchers over the years. In their classic study of rate effect, Casagrande and Wilson (1951) conducted unconfined undrained compression tests on many brittle clays and clay shales and concluded that the compressive strength and secant modulus increased and the strain at failure decreased as the loading rate was increased. However, the specimens were not saturated and pore water pressure was not measured in their study.

Richardson and Whitman (1963) conducted CIU tests on reconstituted saturated specimens of a slightly organic clay from back swamp alluvial deposits of Mississippi river valley at two different rates. A fast rate of 1% strain in one minute and a slow rate of 1% strain in 500 minutes were used. The pore water pressure was measured at the mid-height of the specimen in all the tests using a probe inserted into the specimen at its mid-height (similar to that reported by Hight, 1982). The pore water pressure was also measured at one end of specimen in some tests. The peak deviator stress and stress ratio were found to be higher at faster rate. Undrained shear strength was found to increase by 10%. A linear relationship between undrained shear strength and $\log t_f$ was observed where t_f is time to failure. Pore water pressure was found to be lesser in faster test although it was found to be independent of rate for axial strains less than 1%. The stress path for the faster test was higher and towards the right of the slower test due to less pore water pressure in faster test, as shown in Fig. 2.9a. Comparison of base and mid pore water pressures showed that pore water pressure equalized in slow test but not in fast test. Also, the deviator stress

increased (see Fig. 2.9b) and pore pressure decreased (see Fig. 2.9c) when strain rate was increased suddenly thus indicating a fundamentally inverse relationship between strain rate and pore water pressure. This behaviour is shown at points A and B in Figs. 2.9b and 2.9c.

Some other studies conducted after Richardson and Whitman (1963) have ended in the conclusions similar to those drawn by Richardson and Whitman (1963) although some aspects of the rate-effect on pore pressure are still not fully understood. For example, it has been argued that the use of rough end platens results in non-uniformity of pore water pressure and the trends observed in such tests may not be conclusive. Blight (1965) conducted a study with lubricated end platens or free ends (Rowe and Barden, 1964; Barden and McDermott, 1965) using segmental specimens of a Kaolinitic silty clay while measuring the pore water pressures at the mid-height and base of the specimen and concluded that the stress conditions were more uniform in tests with free ends than those using rough end platens.

Fig. 2.9a: Average effective stress paths in undrained tests on NC clay (Richardson and Whitman, 1963)

Fig. 2.9b: Deviator stress versus axial strain curves from tests with change in strain rate (from Richardson and Whitman, 1963)

Fig. 2.9c: Pore pressure versus axial strain curves from tests with change in strain rate (from Richardson and Whitman, 1963)

Blight (1965) observed an increase in undrained shear strength and corresponding decrease in pore water pressure with increase in strain rate for a Kaolinitic silty clay. However, Blight argued that the strain rate effects and the effects resulting from the equalization of non-uniform pore water pressures are two separate phenomena that take place simultaneously and one is not the cause of the other.

2.3.1.2 Studies in 1970s Proposing Unique Stress-Strain-Time Relationships

Akai et al. (1975), from their study on reconstituted Fukakasa clay over a wide range of strain rates concluded that there might exist a unique stress-strain-time relation of clay. They also reported an increase in deviator stress which is proportional to the increase in logarithm of strain rate. They concluded that strain rate had no obvious effect on the pore water pressure. The variation of deviator stress and pore water pressure with axial strain in their tests are shown in Figs. 2.10a and 2.10b. Akai et al. (1975) measured the pore water pressure only at the base. As pointed out by Blight (1965), the measurement of pore water pressure at the base cannot be relied upon, especially in faster tests. So, the conclusions of Akai et al. (1975) may not be completely reliable.

Fig. 2.10a: Deviator stress-strain relations for different strain rates for Fukakasa Clay (Akai et al., 1975)

Fig. 2.10b: Induced pore pressure-strain relations for different strain rates (Akai et al., 1975)

Vaid and Campanella (1977), based on their study on Undisturbed Haney clay, found out that the peak deviator stress increased with the increase in strain rate (Fig. 2.11a) and peak deviator stress at the fastest test was as much as 30% higher than corresponding value at the slowest rate. Most of the researchers report around 5-10% increase in undrained strength for a 10-fold increase in strain rate. Haney clay showed higher increase in strength. There was essentially a linear increase in undrained strength with the logarithm of strain rate in the region of higher rates in both constant strain rate and constant rate of load increment tests (Fig. 2.11b). The increase in peak deviator stress and hence the undrained shear strength of clay with the increase in strain rate has been reported by numerous researchers (Casagrande and Wilson 1951; Richardson and Whitman 1963; Blight 1965; Saada 1967; Lundgren and Mitchell 1968; Akai et al. 1975; Vaid and Campanella 1977; Graham et al. 1983; Lefebvre & LeBoeuf 1987; Vaid 1988; Fourie and Dong 1991; Sheahan et al. 1996; Zhu et al. 1999; Lo Presti et al. 1999; Li and Rowe 2002; Katti et al. 2003; Ahnberg 2004). However, for Haney clay, in the domain of low strain rates, a lower limit of undrained strength (commonly called upper yield) was approached and a further reduction in rate did not result in additional loss in strength. It was

hypothesized that at a given value of strain, ϵ , the shear stress, q , is a function only of the instantaneous strain rate and is independent of the past strain rate history. Similar hypothesis proposing the existence of a unique stress-strain-time relationship for any clay was earlier proposed by Singh and Mitchell (1969) and Akai et al. (1975). However, Vaid and Campanella (1977) did not present or discuss the pore water pressure variation data. It had been earlier shown (Blight 1965; Lundgren and Mitchell 1968) that the difference in pore water pressures between tests due to a change in shearing rate might be responsible for the change in undrained shear strength.

Fig. 2.11a: Influence of strain rate on undrained stress-strain behaviour of undisturbed Haney Clay (Vaid & Campanella, 1977)

Fig. 2.11b: Variation of undrained shear strength in constant strain rate shear (Vaid & Campanella, 1977)

2.3.1.3 More Recent Studies of Rate-Effect on clay

Graham et al. (1983) presented a good review of test data from literature on rate effect from various undisturbed and reconstituted clays. It was concluded that the undrained strength in CIUC and CAUC tests as well as preconsolidation pressure increase by 10% or more due to a ten fold increase in strain rate regardless of the soils type, plasticity index and stress history (OCR). The increase in preconsolidation pressure with the increase in strain rate was later also reported by Leroueil et al. (1985). It was also concluded by Graham et al. (1983) that at low strain rates, the gradient of shear stress-log (strain rate) relationship can decrease markedly and further decrease in strain rate will not cause any significant reduction in undrained shear strength. This threshold strain rate has been reported to be about 0.2%/hour for Haney Clay (Vaid and Campanella 1977) and about 0.05%/hour for plastic Drammen Clay (Berre & Bjerrum 1973) However, for the tests conducted by Graham et al. (1983) there was no evidence of a threshold strain rate down to 0.01% per hour, and only a weak evidence from one set of tests down to 0.001% per hour was obtained. It was also mentioned that the yield envelope expands with increase in strain rate. It was also shown using the results from various clays that the influence of strain rate on undrained shear strength appears to be independent of soil plasticity. However,

Graham et al. (1983) did not discuss the pore water pressure variation caused by the change in strain rate.

Another good study on rate effect, on three types of undisturbed soft sensitive clays from Canada, was presented by Lefebvre & LeBoeuf (1987). Tests were conducted on specimens with two different types of stress-histories. First type of clay is referred to as 'structured clay' in which the specimen was consolidated isotropically or anisotropically under a consolidation pressure slightly less than the preconsolidation pressure. Second type of clay is referred to as 'destructured clay', in which the specimen was consolidated under a consolidation pressure 1.2 to 2 times greater than the preconsolidation pressure. Three types of clays, namely, Grande Baleine clay, Olga clay and B6 clays, were tested at strain rates varying from 0.05-132.0%/hr. Some results for Grande-Baleine clay are reproduced here in Fig. 2.12a to 2.12d for illustration. For structured clays, the failure occurred at very low axial strain of 1% or less and peak deviator stress increased with strain rate (Fig. 2.12a), as identified in many previous studies. Pore water pressure development up to the failure was not significantly influenced by the strain rate (see Fig. 2.12a). After failure, however, higher pore water pressures were generated at lower strain rates. For Grande Baleine and B-6 clays, the tests performed at different rates defined almost unique effective stress paths, which were located progressively higher in the stress space as the strain rate increased (Fig. 2.12b). However, for Olga clay (Fig. 2.12c) the stress paths for slow tests were towards the left side of faster tests indicating higher pore water pressure in faster tests. However, this difference for Olga clay was attributed to the influence of silty layers in Olga clay. It was concluded that for structured marine clay, the decrease in undrained shear strength that accompanies lower strain rates did not appear to be related to the generation of larger pore water pressures but, rather to a lowering of the failure envelope. Contrary to the structured marine clay, pore water pressure generation was found to be significantly higher with decrease in strain rate in destructured marine clay (Fig. 2.12d). The undrained shear strength increased with the increase in strain rate for destructured clay as well (Fig. 2.12d). Stress paths for the tests at faster rates were towards the right of slow tests owing to the lower pore water pressure in faster tests (2.12e). However, all the stress paths converged to reach the same failure envelope irrespective of strain rate rate. It was also mentioned

that the (c_u/σ'_{vc}) or (c_u/σ'_p) relationship for all clays tested was linear over about five log cycles of strain rate and the gain in strength per log cycle was between 7-14% giving an average of around 10% per log cycle. Therefore, it was concluded by Lefebvre and LeBoeuf (1987) that the effect of strain rate on the undrained shear strength of structured and destructured clays are fundamentally different. For destructured clays, the rate effect is related to the pore water pressure generation and the failure envelope is more or less unique. For structured clays, the pore water pressures are independent of strain rate; however, the effective strength envelope appears to be lowered with decreasing strain rate as a result of fatigue.

Fig. 2.12a: Stress-strain and pore pressure-strain curved for structured Grande-Baleine Clay (Lefebvre and LeBoeuf, 1987)

Fig. 2.12b: Effective stress paths for Grande-Baleine Clay, showing higher stress paths for tests conducted at higher strain rates

Fig. 2.12c: Effective stress paths for Olga Clay showing the shifting of stress paths of fast tests towards the right hand side

Fig. 2.12d: Stress-strain and pore pressure strain curves for normally consolidated (destructured) Grande-Baleine Clay, showing higher peak deviator stress and lower pore water pressure in tests conducted at higher strain rates

Fig. 2.12e: Stress paths for normally consolidated (destructured) Grande-Baleine Clay, showing convergence of all the stress paths towards the same failure envelope

It is important to highlight that the conclusions by Lefebvre and Leboeuf (1987) were drawn based on the measurement of pore water pressure at the base of the specimens alone. Therefore, the conclusions are subject to uncertainties related to possible non-uniformity in the pore water pressures distribution, i.e., whether the pore water pressure measured at the base are representative of the pore water pressure behaviour (Bishop and Henkel 1962; Hight, 1982; Fourie and Dong, 1991).

Fourie and Dong (1991) conducted undrained compression and extension tests at different strain rates on reconstituted Black clay and Kaolin while measuring bottom and mid pore water pressures. They observed that the peak deviator stresses were not affected significantly in both the clays. The mid-height pore water pressures were also found to be unaffected by the strain rate in either clays. However, the base pore water pressures were significantly higher in faster tests thus indicating that large non-uniformity of pore water pressures occurs in faster undrained tests. This paper highlighted the importance of proper pore water pressure measurement and indicated that the top or base measured pore water pressure could not be relied upon in all cases.

Lacasse (1995), based on tests on undisturbed Haga clay, showed that the undrained shear strength and secant modulus of Haga clay increased with the increase in strain rate in compression, extension and direct simple shear tests. It was also observed that the pore water pressure decreased with the increase in strain rate. It was also concluded by Lacasse (1995) that the rate effect increased with plasticity index of soil.

Sheahan et al. (1996) presented results from 25 K_0 consolidated triaxial compression tests on re-sedimented Boston Blue Clay using a computer-automated triaxial apparatus with lubricated end platens and a mid-height pore water pressure measurement device. Specimens were consolidated to four OCR values 1, 2, 4 and 8 and four strain rates 0.05, 0.5, 5 and 50% per hour were used for undrained shearing. The variation of normalized shear stress and shear induced pore pressure for OCR = 1 soil in the tests conducted by Sheahan et al. (1996) are presented in Fig. 2.13a. The shear strength was found to increase with the increase in strain rate for all OCRs (Fig. 2.13b). Test results presented in this paper showed that the strength increase with increasing strain rate is primarily related to the suppression of pore water pressure (see, for example, Fig. 2.13a). Sheahan et al. (1996) concluded that for high OCR soils it is solely due to the reduced pore water pressure while for low OCR soils, the increase in undrained shear strength is caused by the suppression of shear-induced pore water pressure as well as increase in the effective friction angle. It was also concluded by Sheahan et al. (1996) that there exists a threshold strain rate below which there would be no further decrease in s_u and the threshold strain rate increases with increasing OCR (see Fig. 2.13b). Hence the rate dependence is a function of stress history. Effective friction angle at peak was found to increase with rate. However, the final friction angle after softening was found to be independent of the strain rate which is indicative of an intrinsic physical soil property. It was highlighted in this paper that the literature does not contain sufficient data to determine if these same basic trends apply to other nonstructured clays; however, all clays (regardless of stress history) appear to be rate sensitive at “fast” rates of undrained shearing. It was also emphasized that there is a need for further research on the rate-dependent behaviour for different modes of undrained shearing (e.g. direct simple shear and triaxial extension).

Fig. 2.13a: Normalized shear stress and shear induced pore pressure versus strain curves for resedimented Boston Blue Clay with OCR =1 in CK₀UC test (Sheahan et al., 1996)

Fig. 2.13b: Normalized shear strength versus strain rate curves for CK₀UC tests on BBC (Sheahan et al. 1996)

Zhu et al. (1999) presented a study of rate effect on compression and extension behaviours of Hong Kong Marine Deposits (HKMD). One order increment of logarithmic strain rate caused average increase in undrained shear strength of 5% in compression and 9% in extension tests (see Fig. 2.14a). Pore water pressure was found to decrease with the increase in rate in both compression and extension. Pore water pressure variation was more in extension than in compression, thus indicating that the magnitude of rate effect is proportional to the magnitude of pore water pressure variation for extension tests (Fig. 2.14b). However, the data in Fig. 2.14b shows that the pore pressures between the compression tests conducted at different rates was very small. Stress paths and effective friction angles for the soil were not discussed in this paper.

Katti et al. (2003) analyzed the test data from Sheahan et al. (1996) and concluded that any clay at a given OCR, when sheared at higher strain rate, will respond similar to the same clay at higher OCR. Thus, the strain rate response in clays can be modelled as an apparent OC clay response.

Fig. 2.14a: Stress-strain curves in compression and extension on Hong Kong marine deposits (Zhu et al., 1999)

Fig. 2.14b: Pore pressure variation with strain for HKMD (Zhu et al., 1999)

From the discussions in this section, it is clear that there are some contradictory trends regarding the rate effect on clays. The mechanism of rate effect is still not

fully understood. As pointed out by Sheahan et al. (1996), it is important to study the rate effect on more types of clay.

2.3.2 Effect of Loading Mode and Rate of Load Increment on the Undrained Behaviour of Clay

There are only a few studies available in the literatures that discuss load-controlled or stress-controlled tests on clay (Saada 1967; Lundgren et al. 1968; Vaid and Campanella 1977; Vaid 1988; Lacasse 1995; Teachavorasinskun et al. 2002).

Saada (1967) argued that one seldom finds constant rate of deformation loading in the field conditions and load-controlled tests are better representation of most of the field loading conditions. Based on constant stress increment rate tests on Kaolin, Saada (1967) found that the failure strain was independent of the stress increment rate. However, the failure stress decreased linearly with the logarithmic decrease in rate of stress increment. It was also found that the excess pore water pressure was lesser at faster rate of stress increment. The strain rate was found to continuously increase during the stress controlled test and the variation of strain rate was affected by the stress increment rate.

Lundgren et al. (1968) discussed the advantages and disadvantages of stress-controlled tests over the conventional deformation-controlled tests. It was mentioned that the pore water pressure equalization is better in stress-controlled tests than in deformation-controlled tests in the beginning of shearing. However, as the failure is approached, the strain rate increases significantly in stress controlled tests and non-uniformity of pore water pressure increases inside the specimen in undrained tests. A drained test becomes a partially drained test towards the end of stress controlled test. An indication of the yield stress of structure-sensitive clays can be obtained in stress controlled test while it is more difficult in deformation-controlled tests due to large rate of stress increment in the beginning of deformation-controlled test. One disadvantage of stress controlled tests is that abrupt failure or complete collapse of the specimen occurs when failure is approached. This makes it impossible to study the post-peak behaviour of soil, which is especially important for soils showing large strain softening behaviour. Lundgren et al. (1968) discussed a triaxial testing set-up

which combines stress-controlled and deformation-controlled tests by switching from stress-controlled to deformation-controlled mode of shearing when failure is approached. This combines the advantages of both stress-controlled and deformation-controlled tests. A typical stress-strain test on compacted Kaolin is compared with conventional stress-controlled and strain-controlled tests in Fig. 2.15. No study of effect of stress increment rate was made in this paper.

Fig. 2.15: Combined stress-strain controlled loading (Lundgren et al., 1968)

Vaid & Campanella (1977) presented the data from load-controlled undrained compression tests on Haney clay. The undrained shear strength was found to increase linearly with the logarithmic increase in rate of load increment (Fig. 2.16a), which was similar to the deformation-controlled tests. However, there was smaller post-peak drop in deviator stress (implying higher post-peak resistance and lesser strain softening) in load-controlled test than deformation-controlled tests (see Fig. 2.16b and 2.11a) which was attributed to the continuously increasing strain rate towards the end of test in load-controlled tests. Pore water pressure variation was not discussed in

this paper. However, it was observed that the effective failure envelope of the soils was unique irrespective of rate and loading mode (Fig. 2.16c). Axial strain at peak also was largely unaffected by the loading rate and loading mode.

Fig. 2.16a: Variation of undrained shear strength of undisturbed Haney Clay in constant loading rate shear (Vaid & Campanella, 1977)

Fig. 2.16b: Influence of loading rate on undrained stress-strain behaviour on undisturbed Haney Clay (Vaid & Campanella, 1977)

Fig. 2.16c: Unique failure envelope for undisturbed Haney Clay (Vaid and Campanella, 1977)

Lacasse (1995) reported stress controlled and deformation-controlled compression, extension and direct simple shear tests on anisotropically consolidated undisturbed samples of Haga clay. Constant rate of stress tests were found to give higher undrained shear strength but smaller modulus than the constant strain rate tests in both triaxial and direct simple shear tests. Constant rate of stress loading gave as much as 20% higher shear strength than the constant strain rate loading for OCR of 40. The stress-strain and pore water pressure-strain curves up to a shear strain of 5% were quite similar in the two types of loading modes on Haga clay.

Teachavorasinskun et al. (2002) presented the results of stress-controlled compression, extension and cyclic triaxial tests conducted at different rates of stress increment on undisturbed Bangkok clay. The secant modulus (Fig. 2.17a) was found to increase and excess pore water pressure (Fig. 2.17b) was found to decrease with the increase in stress-increment rate for both compression and extension tests. The effect on undrained shear strength and effective stresses was not discussed.

From the limited number of studies presented above, it is clear that there is a need to study the effect of loading mode on the undrained and drained behaviour of clay. It is

especially important because contradictory trends emerge from at least two studies (Vaid and Campanella, 1977 and Lacasse, 1995).

Fig. 2.17a: Increase in secant modulus with the increase in stress rate on Bangkok clay (Teachavorasinskun, 2002)

Fig. 2.17b: Reduction in pore water pressure with increase in stress rate on Bangkok clay (Teachavorasinskun, 2002)

2.3.3 Drained Behaviour of Clay

Clays are known to have very low permeability and in most of the field loading conditions clay is in undrained condition. However, drained behaviour of clay is important in long term loading problems such as stage construction of embankments and stability of structure after a long duration after the construction is over. Studies of drained behaviour of clay have been limited. Theoretically, the conditions of a fully drained test are strictly satisfied only if the loading rate is infinitely slow, and therefore, in practice, a certain amount of excess pore water pressure will always remain undissipated in a drained test. The fully drained strength of soil will differ from the measured drained strength by an amount proportional to the magnitude of undissipated excess pore water pressure (Gibson and Henkel 1954). The principal factors affecting the excess pore water pressure, and therefore the measured drained strength, are compressibility and permeability of soil, location of the drainage surface around the specimen and loading rate. Amongst these factors, the loading rate is the easiest to control. Gibson and Henkel (1954) presented a theoretical method for choosing the 'time to failure' in a drained test. If U_c is the degree of dissipation of pore water pressure at the centre of the specimen, $2H$ is the height of specimen and c_v is the coefficient of consolidation, then time to failure, t_f , can be determined using the following equation.

$$t_f = \frac{H^2}{2c_v(1-U_c)} \quad (2.1)$$

It was shown by drained tests conducted at different rates on Weald clay, London clay and Kaolinite that the excess pore water pressure rises at the start of a drained test and remains constant after reaching a certain value. The excess pore water pressure dissipates again after some time. It was observed that the drained strength increases with the increase in time to failure at first but when the time to failure is very long, the drained strength falls again (Fig. 2.18). The reduction in strength on increase in duration of test was attributed by Gibson and Henkel to the small viscous component present in the strength of clay. The applicability of formula derived by Gibson and Henkel for all soils, however, is not clear. For example, Hvorslev (1960)

found that for normally consolidated Little Belt clay, the test duration required for carrying out a drained test was only about one-tenth of the value obtained from the Gibson-Henkel formula. It is necessary, therefore, to study the effect of loading rate on the drained behaviour of other clays.

Newson et al. (1997) proposed a mathematical procedure for determining the loading rate for drained stress path tests and validated their theory by drained stress path tests on Kaolinite. It was observed that on using a fast shearing rate, the resulting stress path deviates significantly from the required stress path due to the development of high excess pore water pressures. It was predicted that the sample will fail prematurely if the loading rate is higher than a certain value.

Although there have been many studies on the drained behaviour of clay (Parry, 1960; Lundgren et al., 1968; Lewin et al., 1970; Lo and Morin, 1972; Parry and Nadarajah, 1973), studies of effect of shearing rate on the drained stress-strain and strength behaviour are limited. Considering the fact that the undrained behaviour of clay is significantly affected by the shearing rate, it appears necessary to study the shearing rate effects on the drained behaviour of clay.

Fig. 2.18: Variation of drained strength with time to failure in shear box test on London clay (Gibson and Henkel, 1954)

2.4 UNDRAINED CREEP BEHAVIOUR OF CLAY

Apart from the rate-effects discussed above, two other important time-dependent deformation behaviours, which are shown by clay, are undrained creep and the behaviour in constant stress drained (CSD) tests. The creep can be defined as ‘a gradual loss of strength of a soil specimen under constant shear stress, which is accompanied by increasing strains and changing strain rate’ (Vaid & Campanella, 1977). The schematic of variation of effective stresses and axial strain rate is shown in Figs. 2.19a and b. When the soil is loaded to a certain load or deviator stress level (see Fig. 2.19a), which is below the failure and load or stress is held constant, the soil undergoes undrained creep deformation. Depending on the level of shear stress the changing strain rate during creep may either lead to a stabilization of strain rate, or to sudden increase in strain rate and ultimately creep rupture. If the applied deviator stress level is below a threshold level, the axial strain rate (Fig. 2.19b) continuously decreases. If the applied stress level is higher than a threshold value, the axial strain rate reaches a transient minimum (Fig. 2.19b) and starts to increase again. The specimen becomes unstable and failure or creep rupture occurs.

Fig. 2.19 (a) Stress path during creep and CSD tests (b) variation of strain rate vs. time

The undrained creep behaviour of soils has been studied by numerous researchers (Casagrande and Wilson 1951; Christensen and Wu, 1964; Singh and Mitchell 1968; Mitchell and Singh 1968; Singh and Mitchell 1969; Walker 1969; Arulanandan et al.

1971; Duncan and Buchignani 1973; Sekiguchi 1973; Finn and Shead 1973; Campanella and Vaid 1974; Akai et al. 1975; Vaid and Campanella 1977; Wu et al. 1978; Kavazanjian and Mitchell 1980; Mesri et al. 1981; Murayama et al. 1984; O'Reilly et al. 1988; Mejia et al. 1988; Vaid 1988; Hird and Hassona 1990; Kuhn and Mitchell 1993; Sheahan 1995; Morsy et al. 1995; Zhu et al. 1999; Futai et al., 2004; Liingaard et al., 2004; Augustesen et al., 2004). It can be seen from this list, that undrained creep behaviour of clay has been a subject of intense study from 1960's till now.

2.4.1 Early Studies on Undrained Creep Behaviour of Clay

One of the earliest studies on undrained creep was reported by Casagrande and Wilson (1951). The investigation showed that some types of brittle undisturbed clays and clay shales crept under a sustained load, and they ultimately failed under a sustained load appreciably less than the strength indicated by a normal laboratory compression test. In contrast, it was found that two laboratory-compacted soils and one undisturbed soil which were not fully saturated tended to become stronger and stiffer under sustained loads. Casagrande and Wilson (1951) defined a 'creep-strength test' as 'a test in which load is built up quickly and maintained constant until the specimen fails'. For such tests, 'time to failure' was defined as the elapsed time between application of the load and failure. Casagrande and Wilson (1951) observed that in creep-strength tests failure was preceded by a reversal of slope of the time-deformation curve, followed by continuous deformation at an increasing rate. The point of reversal of slope was identified as the 'start of failure'. Casagrande and Wilson (1951) did not measure the pore water pressure and therefore the mechanism of creep in their test results was not discussed.

2.4.2 Undrained Creep and Stress-Strain-Time Relationships

Singh and Mitchell (1968), Mitchell et al. (1968) and Singh and Mitchell (1969) analyzed the creep data obtained for a number of clays from published literature. A schematic representation of the influence of creep stress intensity or creep stress level on creep rate at any given time after the stress is applied as presented by Singh and Mitchell (1969) is shown in Fig. 2.20a. Creep rates are small at low stresses and

are not important for real field cases. A linear relationship was found between the logarithm of the strain rate and stress in the midrange of stresses, which are representative of stresses used in practice. As stress level approaches the strength of material, the strain rate increases rapidly and finally creep-rupture occurs, as shown in Fig. 2.20b for stress level of 0.676kg/cm² for Osaka Alluvial Clay (as discussed by Singh and Mitchell, 1968). Singh and Mitchell (1968) used the Theory of Rate Processes to propose a three-parameter, phenomenological general function for soils which expresses the strain rate, $\dot{\epsilon}$, at any time t , after application of sustained deviator stress, D

$$\dot{\epsilon} = Ae^{\alpha D} \left(\frac{t_1}{t} \right)^m \quad (2.2)$$

where,

A = strain rate at time t_1 and $D = 0$ (Projected value)

α = value of the slope of the mid-range linear portion of a plot of logarithmic strain rate versus deviator stress, all points corresponding to the same time after load application

t_1 = unit time

m = slope of a logarithmic strain rate versus logarithmic time straight line

Eq. 2.2 is applicable for the linear part of Fig. 2.20a. It was mentioned by Mitchell et al. (1968) that this equation seemed to be valid irrespective of whether the clays were undisturbed or remoulded, wet or dry, normally consolidated or overconsolidated, or tested drained or undrained. This equation can be further integrated to get the strain at any time during creep. It was concluded that depending upon the creep potential of soil, creep might cease, continue or culminate in creep rupture under a given value of sustained stress. If the stress-strain-time behaviour is expressed in the form of Eq. 2.2, then the parameter 'm' quantifies the creep potential of soil. The smaller the value of 'm', the larger will be the creep strains and the higher is the probability of creep rupture. It was mentioned by Singh and Mitchell (1968) that if $m > 1$ then creep will continue at ever decreasing strain rate and rupture will not take place. However, if $m \leq 1$, then creep rupture will most likely occur.

Fig. 2.20a: Schematic representation of influence of creep stress level on creep rate (after Singh and Mitchell, 1968)

Mitchell et al. (1968) and Singh and Mitchell (1968) mentioned that the pore water pressure was not affected much by creep and after the initial rise in pore water pressure for a short duration after the application of creep stress, the pore water pressure became constant. The stress paths and effective stresses during creep were not discussed. Also, Singh and Mitchell (1968) mentioned that Eq. 2.2 is only valid for creep deformations under the first application of shear stress and not for successive increments of load as encountered in multi-stage-creep problems such as stage construction of embankments. It may be pointed out that although the creep potential and creep rupture were discussed, only the pattern of axial strain and strain rate changes were discussed and the mechanisms responsible for creep rupture, effective stresses during creep and effective stress state at which the creep rupture will occur were not discussed by Mitchell et al. (1968) or Singh and Mitchell (1968).

Fig. 2.20b: Variation of log (strain rate) with log (time) for Osaka Alluvial Clay at various creep stress levels (after Murayama and Shibata, 1958, as quoted by Singh and Mitchell, 1968)

Campanella and Vaid (1974) proposed a model similar to Mitchell and Singh (1968). Campanella and Vaid (1974) characterized the creep behaviour by up to three regimes as shown in Fig. 2.21a. When the creep shear stress is first applied, a primary creep phase always occurs (Singh and Mitchell 1969; Walker 1969) and strain accumulates progressively at lower strain rates. At lower shear levels, the strain rate essentially vanishes and the stress state stabilizes. At higher stress levels the strain rate does not vanish, but reaches a minimum value, $\dot{\epsilon}_{\min}$ (Fig. 2.21b). Secondary creep is the phase when strain occurs at $\dot{\epsilon}_{\min}$ for some time period, followed by a tertiary creep, which is when the specimen becomes unstable, and the strain rate rapidly increases and the soil fails or ruptures. It has been shown that the secondary creep does not exist for some clays (Finn & Shead 1973; Hicher 1988) and there is a well defined variation of strain rate during complete test and strain rate was not constant in any particular interval of time.

Campanella and Vaid (1974) and Vaid and Campanella (1977) suggested that the test results obtained under all time-loading histories would complement each other if suitable account can be taken of the manner in which the time affects the stress-strain-strength response of a cohesive soil. With the time effect duly accounted for, it would then be possible to predict clay behaviour under one type of loading, e.g. constant strain rate from the results of another type of loading, e.g. constant stress creep tests and vice-versa. It basically means that there exists a unique stress-strain-time relationship for any clay. It was hypothesized that 'at a given value of strain, the shear stress is a function only of the instantaneous strain rate, and is independent of the past strain rate history'. Vaid and Campanella (1977) showed that this is actually true for Haney clay. Campanella and Vaid (1974) and Vaid and Campanella (1977) concluded that pore water pressure and axial strain in undrained tests are uniquely related to each other and in terms of effective stresses (Fig. 2.21c). It was shown by Campanella and Vaid (1974) and Vaid (1988) that both creep rupture and failure in conventional constant strain rate shear tests are defined by unique failure envelope. Vaid and Campanella (1977) obtained a linear failure envelope for creep rupture (see Fig. 2.16c), which is identical to the envelope obtained from conventional constant shearing rate tests for Haney Clay.

Fig. 2.21a: Stages of creep after initial application of load or deviator stress that is large enough to cause failure (after, Campanella and Vaid, 1974)

Fig. 2.21b: Variation of axial strain rate with time during undrained creep (Campanella and Vaid, 1974)

Fig. 2.21c: Unique relationship between pore pressure and axial strain (Campanella and Vaid, 1974)

2.4.3 Effective Stress Interpretations and Pore Pressures during Creep

Walker (1969) presented the data from undrained creep tests (at constant q) on Leda clay and described the creep phenomenon from a perspective, different from that described by Mitchell et al. (1968) and Singh and Mitchell (1969). Walker studied the undrained creep behaviour using effective stresses and pore water pressure variation as the basis and emphasized that during the creep, the effective stress state of the specimen shifts with time towards the failure envelope of the clay (Fig. 2.22a). It was shown that the pore water pressure continued to increase during creep even after a long duration after the application of creep stress (Fig. 2.22b). The creep in pore water pressure was proportional to the mean effective stress and creep strain and pore water pressure were directly related (Fig. 2.22c). Walker (1969) divided the undrained creep curve into two parts: the primary part which is associated with equalization of pore water pressures and a secondary part which takes place under uniform pore water pressures. Walker (1969) considered that the secondary region is a result of the same mechanism which causes secondary compression in consolidation and drained shear tests. Walker emphasized that an appreciation of the creep phenomenon is only possible within the framework of effective stress concept. The support for this argument was outlined with particular reference to a highway embankment on a thick clay deposit where pore water pressures were monitored using piezometers. Piezometer readings indicated that after the completion of construction, a significant build-up in pore water pressure occurred, this being accompanied by increasing settlement throughout the clay mass (Fig. 2.22d).

Fig. 2.22a: Effective stress paths during undrained creep (Walker, 1969)

Fig. 2.22b: Typical pore water pressure variation during undrained creep (Walker, 1969)

Fig. 2.22c: Pore pressure vs. axial strain curves during creep (Walker, 1969)

Fig. 2.22d: Field data from embankment illustrating undrained creep (Walker, 1969)

Fig. 2.23: Variation of pore pressure with time in undrained creep tests (Arulanandan et al., 1971)

Arulanandan et al. (1971) conducted a study of undrained creep on San Francisco Bay Mud and came to conclusions similar to those by Walker (1969). Significant amount of pore water pressures developed during undrained creep (Fig. 2.23) which led to the stress-state reaching the failure envelope and creep rupture of specimen. The pore water pressure developed during undrained creep was attributed to the prevention of secondary compression as well as thixotropic effects. The pore water pressure was found to build up when a specimen was left undrained at zero deviator stress (see Fig. 2.23), thus establishing that the pore water pressure generation during undrained creep may also be primarily due to the prevention of secondary compression. Arulanandan et al. (1971) emphasized that any theory developed for the prediction of stress paths for undrained strength and creep behaviour of sensitive soils should take into account the pore water pressures developed during secondary compression at zero deviator stress.

O' Reilly et al. (1988) observed that, in many cases, 'failure occurred at an early stage' during the undrained creep tests on reconstituted Keuper Marl, a silty clay. O' Reilly et al. (1988) did not clarify what they meant by 'early failure'. For Keuper Marl, it was indicated that the 'failure during undrained creep is not determined exclusively by the proximity to the critical state'.

2.4.4 Practical Implications of Undrained Creep in Clay

The engineering significance of creep has been illustrated by case-studies conducted by Duncan & Buchignani (1973) for the stability of slopes, excavations and tunnels (Clough & Schmidt, 1982). The plate anchors, when subjected to a sustained load (Ponniah, 1988) were observed to creep with significant movements at loads above 60 % of the ultimate capacity. Morsy et al. (1995) simulated creep deformation in the foundation of Tar Island Dyke. Movement of this dyke was monitored over 25 years and showed significant creep deformation of over 1 meter in the foundation clay although the loading due to the dyke on the clay was constant for over 15 years. The pore water pressure remained constant over this period indicating that the deformation was due to the undrained creep rather than secondary consolidation. It is clear from these studies that the creep behaviour of clay is important for many different field loading problems.

Duncan and Buchignani (1973) analyzed an underwater slope in San Francisco Bay Mud that had failed during construction of a ship docking terminal. On conducting undrained creep tests on soil specimens from the failure site, it was observed that the undrained strength of the soil reduced drastically during creep and shearing resistance of a soil which underwent creep for 1 week and longer was only 70% of the value measured in conventional triaxial test (Fig. 2.24). It was recommended that for any clayey soil, the loss of shearing resistance under sustained load should be taken into account during design and appropriate correction factor should be applied.

Rowe and Hinchberger (1998), during the modelling of Sackville Test Embankment, observed that both the observed and calculated excess pore pressures increased during periods when there was no construction and this was attributed to “creep-induced” pore pressures. It was also argued that the most critical time with respect to stability of embankment on clay may not be during construction but some time after construction has been completed. This type of failure is referred to as “progressive failure”.

Fig. 2.24: Effect of sustained loading on the undrained shear strength of San Francisco Bay Mud (Duncan and Buchignani, 1973)

In view of the importance of undrained creep in engineering problems as described above, numerous attempts have been made to model the undrained creep behaviour of clay. A brief introduction to some of these is given in the following section.

2.4.5 Modelling of Creep Behaviour in Clay

Uniqueness of the stress-strain-time relationship for any clay, as shown by Mitchell and Singh (1968) and Vaid and Campanella (1977), was earlier shown for reconstituted Fukakasa clay by Akai et al. (1975). Wu et al. (1978) also obtained similar relationships for a variety of clayey soils. This model has been proposed by these authors with various modifications for different clays. A more general model was later proposed by Kavazanjian and Mitchell (1980) to include deviatoric and volumetric model components. Time-dependent values of secant elastic moduli for use in numerical analysis can be evaluated by these model components. However, this model could not predict well the time-dependent pore water pressure variation and need for further investigation was emphasized. The exponential stress-strain model proposed by Singh and Mitchell (1969) was later used to propose more complex hyperbolic stress-strain model by Mesri et al. (1981).

Kuhn and Mitchell (1993), in the light of creep equation proposed by Singh and Mitchell (1968), conducted numerical simulations and proposed a viscofrictional sliding mechanism for slow sliding velocities to represent creep in soil. Sheahan (1995) presented an experimental method for interpreting undrained creep behaviour for cohesive soils based on effective stresses. Based on many previous studies on different soils, it was shown that a static yield surface (SYS) exists for most of the soils, which is a locus of effective stress states that represent a soil's yield surface for behaviour at negligible or zero strain rate. It was indicated that with strain decay during primary creep, as long as the applied creep stress level is below the top of the SYS, a strength threshold known as the upper yield strength (s_{uy}), no creep rupture will occur and the stress state will stabilize on SYS (Fig. 2.25). At shear stress levels greater than s_{uy} , the stress state will remain above the SYS and migrate to the soil's failure envelope and rupture. A method to obtain SYS and s_{uy} from constant strain rate undrained shear test was also presented in this paper. However, it was pointed out that SYS should exist for the soil in question and it needs research for more clays to verify the existence of SYS and the concept proposed in this paper.

Fig. 2.25: Static yield surface and stress states in constant strain rate and undrained creep tests for SFBM (Sheahan, 1996)

Oka et al. (1994) proposed a viscoplastic softening constitutive model for clay by introducing a second material function into the overstress type viscoplastic flow rule. According to this model, there is a critical value of the stress ratio, η_c , at minimum strain rate during undrained creep. If η_c falls in the stable region (Fig. 2.26), failure will not take place. However, if the stress ratio is equal to or greater than η_c , then stress state will lie in the unstable region and failure may occur. Only two tests on Osaka alluvial clay were conducted to verify the model and it needs further investigation.

Fig. 2.26: Stable and unstable regions in the stress space (after Oka et al., 1994)

Lin and Wang (1998) developed an experimental model for the prediction of the undrained creep strain of normally and overconsolidated clays adjacent to an excavation site. An attempt was also made to derive a formulation for the calculation of the stress-strain-strain rate relationship of a constant-rate-of-strain test using constant stress creep test results based on the model proposed by Vaid and Campanella (1977). The predicted results were in good agreement with the experimental results from undrained creep tests in axial compression and lateral extension on undisturbed samples of Taipei clay. Undrained creep behaviour consisting of threshold stress level and strain rates during creep in axial compression

and lateral extension were found to be identical except that pore water pressure variations were different. However, the model could not account for strain softening.

Augustesen et al. (2004) and Liingaard et al. (2004) presented a review of time-dependent behaviour of soils and a review of models for characterization of time-dependent behaviour of soils respectively. Earlier studies related to creep; relaxation and rate-effect in undrained shearing of clay and sand were reviewed. This paper described the creep behaviour in the light of variation of axial strain and axial strain rate with time. However, the variation of effective stresses in stress space was not discussed. It confirms the observation earlier made in this chapter that most of the studies of creep behaviour of clay have focused on variation of strain rate with time and the threshold creep-stress level below which rupture will not occur, without giving due attention to the variation of pore water pressure and effective stresses during the process of creep.

2.4.6 Undrained Creep in Extension Tests

Zhu et al. (1999) presented the only known study of undrained creep in extension test on clay. Undrained multi-stage creep tests under both compression and extension were carried out on isotropically consolidated specimens of Hong Kong Marine Deposits (HKMD). The variation of axial strain and axial strain rate in compression creep tests are shown in Figs. 2.27a and 2.27b. The axial strain increased at an increasing rate in the creep stage causing rupture (Fig. 2.27a). In both compression and extension creep tests, the $\log(\dot{\epsilon}_a) - \log(t)$ relationship at lower stress level was linear (Fig. 2.27b). However, the slope of these lines was dependent upon current deviator stress as well as the stress history. In multi-stage triaxial creep tests, extension tests reached rupture at a lower stress level (0.90) than compression (0.99). Therefore, the soil under extension might be more susceptible to creep than the soil under compression. However, the paper presented the results of only one each of compression and extension creep tests and it was pointed out by Zhu et al. (1999) that there is a need for further research.

Fig. 2.27a: Variation of axial strain with time in multistage undrained creep tests on Honk Kong Marine Deposits (Zhu et al., 1999)

Fig. 2.27b: Variation of axial strain rate with time in undrained creep (Zhu et al., 1999)

2.4.7 Creep Rupture in Clay versus Instability in Sand

The reversal of slope of strain vs. time and strain rate vs. time curves after initial reduction as shown in Fig. 2.21b, 2.27a and 2.27b above, is a phenomenon similar to that observed in loose sand when effective stress is reduced at constant vertical load

and is referred to as ‘instability’. It may be pointed out that the term ‘instability’ is generally not used in relation to the cohesive soils and is used primarily in relation to the granular soil (Lade 1994; Chu and Leong 2002; Lipinski, 2001). Chu and Leong (2002) defined instability as ‘a behaviour in which large plastic strains are generated rapidly owing to inability of a soil element to sustain a given load or stress’. At the onset of instability, the soil loses its capacity to resist further shear stresses and rapid deformations develop, which may ultimately lead to collapse. It has been established for liquefaction of sand (Lade 1994; Chu and Leong 2002; Lipinsky 2001; Lade 1999) that the terms instability and failure are not synonymous. Instability refers to the initiation of flow when large strains develop at an accelerating rate whereas failure is a state when maximum shear strength is achieved. For loose sand, instability may occur much before the effective stress state reaches the failure envelope and it is referred to as ‘pre-failure instability’. The region bounded by instability line and failure envelop is called the ‘zone of potential instability’ (Fig. 2.28), in which soil is in unstable state of stresses and instability may occur. In Fig. 2.28, the instability line is defined as the line that separates potentially unstable stress states from the stable stress states and it can be determined experimentally by a line connecting the peak of a series of effective stress paths obtained from undrained tests (Chu and Leong 2002). For instability to occur, the stress-state must be in the zone of instability.

Fig. 2.28: Schematic diagram of instability line and region of potential instability on q - p' plane for a soil showing strain-softening (after Lade, 1994)

Whether a concept similar to the concept of instability and ‘pre-failure instability’ can be applied to the clay to describe its undrained creep behaviour is not known. Oka et al. (1996) gave an indication (see Fig. 2.26) through their numerical model that a similar concept could be true for clay. Carefully conducted undrained creep tests with proper measurement of pore pressures and effective stresses can throw more light on it.

2.5 CONSTANT STRESS DRAINED (CSD) BEHAVIOUR OF CLAY

As discussed earlier by Walker (1969) and Arulanandan et al. (1971), the pore water pressure develops and effective stress path moves towards the effective failure envelope (see Fig. 2.19a) during undrained creep and rupture occurs when the stress path hits the failure envelope. Similar change in stress conditions occurs when confining stress is reduced or pore water pressure is increased in a soil loaded at a constant vertical stress or overburden. Such a test is sometimes called the constant-shear-stress-drained (CSD) test (Clough and Schmidt 1982; Anderson & Reimer 1995; Zhu and Anderson 1998; Dai et al. 1999; Farooq et al. 2002; Futai et al. 2004). Since the stress state continuously moves towards the failure envelope at constant load or deviator stress, it may be expected that the soil will become unstable when the stress path hits the failure envelope. A runaway type of failure similar to the creep rupture will occur in such a condition. Some of the field loading conditions, where such a reduction in confining stress under constant shear stress is encountered are, by soil in a slopes during excavation (Clough and Schmidt 1982), due to weathering in natural slopes and cuts, during rainfall induced infiltration of water into the soil and on the surrounding soil during tunnelling. A review of some of the studies of CSD tests is presented in the following paragraphs.

Anderson and Reimer (1995) conducted CSD tests on a uniformly graded sand and an undisturbed clayey colluvial soil. In CSD test on colluvial soil, CSD path was applied on some of the specimens by reducing the confining pressure while in some specimens it was applied by increasing the back pressure at constant shear stress. Stress path generated in both methods were same and the difference in drainage conditions within the specimen due to the two techniques was found to be

insignificant. The collapse occurred when the stress state reached the failure envelope. However, it may be noted that the colluvial soil used in this test was not clay and further study of CSD path on clay will be useful.

Zhu & Anderson (1998) conducted CSD tests on a Hawaiian residual soil to study the possible mechanism of rainfall induced slope failure that had occurred in the region from where the samples were taken. Substantial differences of shear strength and soil behaviour were found between anisotropically consolidated undrained tests (CAU) and CSD tests. Stress paths during CSD tests crossed the yield envelope and approached the large strain failure envelope determined from CAU tests. The slope of large strain envelope for CSD tests was lower than the large strain envelope for CAU test (Fig. 2.29b). Abrupt failure was observed after reaching the failure envelope (Fig. 2.29c). Considering the large differences between the CAU and CSD tests, it was recommended by Zhu and Anderson (1998) that different stress paths should be used during soil investigation for different design problems. A similar study was conducted by Dai et al. (1999) on a loosely compacted volcanic-derived soil. It was concluded that the critical states obtained from CID, CIU and CSD tests are same, indicating that critical state line was not affected by the type of tests conducted. It was again observed that the failure in CSD test is sudden. The failure was caused by the increase in pore water pressure due to the infiltration of water. For compressive soils, the increased pore water pressure generated due to the infiltration of water led to a compressive failure in drained manner. For dilative soils, the increased pore water pressure due to the infiltration led to further dilation, which reduced pore water pressure and increased shear resistance of soils. Similar observations were made by Farooq et al. (1999) for a silty sand in Japan.

From the studies of CSD tests on mostly sandy soils that have been presented above, it appears that the behaviour of soil in CSD test may be significantly different from that in conventional CAU, CIU or CID tests. Since CSD path represents more closely many of the field conditions, it is worth studying the behaviour of clay in constant-shear-stress-drained tests. Also, the stress path for CSD test resembles the stress path for undrained creep tests and, therefore, some important correlation may exist between undrained creep and CSD tests.

Fig. 2.29a: Effective stress path of CSD tests (After Anderson & Reimer, 1995)

Fig. 2.29b: Failure envelopes for CSD test and CAU tests on Hawaiian residual soil (Zhu and Anderson, 1998)

Fig. 2.29c: Runaway type of failure in CSD tests on Hawaiian residual soil (Zhu and Anderson, 1998)

2.6 SUMMARY

This chapter presented a literature review of general stress-strain behaviour and time-dependent behaviour of clay. The behaviour of clay under various loading rates and undrained creep was examined.

Most of the cohesive soils have been found to be affected by the shearing rate. Most of the researchers report an increase of around 10% in undrained shear strength due to an increase in rate by one order of magnitude. A linear increase in undrained shear strength with logarithmic increase in strain rate has been reported by many researchers (Singh and Mitchell 1968; Vaid and Campanella 1977; Akai et al. 1975; Lefebvre & LeBoeuf 1987). However some soils have been found to be affected much lesser (Fourie and Dong 1991) than others. Secant modulus and preconsolidation pressure of clay have also been found to increase with the increase in strain rate (Graham et al. 1983; Leroueil et al. 1985). However, the mechanism of strain-rate effect is not clear despite many studies over the years. Singh and Mitchell (1969) proposed the existence of a unique stress-strain-time relationship for any clay

and it was later confirmed by many other researchers. However, this model does not take into account the pore water pressure variation and effective stress changes due to the change in shearing rate. Most of the researchers believe that as the shearing rate increases, the pore water pressure is suppressed and leads to an increase in undrained shear strength. However, some researchers do not agree with this interpretation. Blight (1965) believed that the strain rate effects and effects resulting from the equalization of non-uniform pore water pressures are two separate phenomena that take place simultaneously and one is not the cause of the other. On the other hand Akai et al. (1975) reported that there was no effect of strain rate on the pore water pressure developed in Fukakasa clay although the undrained shear strength was affected. Sheahan et al. (1996) argued that the observation of decreasing excess pore pressure with increase in strain rate might be only a measurement error for tests without special pore pressure measurement such as the use of mid-height pore pressure transducer (Hight, 1982).

The behaviour of clay is affected by loading mode, i.e. whether the test is conducted under load-controlled or deformation-controlled loading. The loading mode effects may be related to the shearing rate effect. The difference between the deformation-controlled and load-controlled tests is in the variation of axial strain rate and axial strain rate with time. However, the studies on the effect of loading mode are very limited and yet give contradictory results (Lacasse, 1995; Vaid and Campanella, 1977). Vaid and Campanella (1977) reported that there were no differences between load-controlled and deformation-controlled tests on Haney clay. However, Lacasse (1995) reported that the undrained shear strength of undisturbed Haga clay was higher by 20% in load-controlled test than that in deformation-controlled test. Therefore, the effect of mode of shearing on the behaviour of clay needs further investigation.

Another important aspect of the time-effect on the behaviour of clay is undrained creep. Cohesive soils have been found to undergo large creep deformations at stress levels much lesser than their undrained shear strength and if the stress level is high enough, creep rupture occurs (Singh and Mitchell, 1969; Fin and Shead, 1973; Vaid and Campanella, 1977; Sheahan, 1994; Augustesen et al., 2004). Creep rupture in clay is in some ways similar to the liquefaction in sand. Sand is known to exhibit

instability much before the stress state reaches the failure envelope and a zone of potential instability exists in effective stress space in which the pre-failure instability leading to collapse may occur (Lade 1994; Chu and Leong 2002). In a similar way, a zone of instability for clay has been also indicated by Oka et al. (1994). However, the possible existence of pre-failure instability behaviour in clay has not been examined since most of the studies on undrained creep behaviour of clay have focused on time-dependent variation of axial strain and strain rate alone without paying much attention to the effective stresses. Vaid and Campanella (1977) proposed a hypothesis that there is a unique stress-strain-strain rate relationship for soil and soil behaviour under one kind of shearing (e.g. constant strain rate shear) can be studied by other kind of tests (e. g. undrained creep tests). However, pore water pressure variation and effective stresses are not considered in this hypothesis. On the other hand Walker (1969) and Arulanandan et al. (1971) argued that an appreciation of the creep phenomenon is only possible within the framework of effective stress concept.

2.7 ASPECTS FOR FURTHER STUDY

In the light of the literature review presented above, it is felt that further study of time-effects on clay, namely, effect of rate and loading mode and undrained creep behaviour of cohesive soils, is required. Proper pore water pressure measurements with due attention given to the effective stress variations is necessary in addition to the variation of strains and strain rate with time. The important aspects for further study can be listed as follows.

1. Study of rate effect on the undrained behaviour of clay with proper pore pressure measurement, that is, mid-height pore pressure measurement, is required for better understanding of effective stress changes due to the rate effects and to determine the possible reasons for it.
2. Effect of loading mode, namely load-controlled and deformation-controlled loading modes, on the undrained behaviour of clay is required. The runaway type of failure that occurs in load-controlled tests should be examined.

3. Although rate-effect on the undrained behaviour is well studied, there have been few studies on rate effect on the drained behaviour of clay. Therefore, the effect of loading rate and loading mode on the drained behaviour should be studied.
4. The process of undrained creep needs to be studied with special emphasis on proper pore pressure measurement. Time-dependent variation of effective stresses during undrained creep tests must be examined along with the variation of strain rate and axial strain rate.
5. The distinction between instability and failure is clear and pre-failure instability is found to occur during certain loading conditions for sand. However, a distinction between the instability and failure is not made for cohesive soils. Hence, there is a need to examine whether such a concept can be applicable to cohesive soils also. There are indications from the published data in literature that the instability might occur in load-controlled shearing and undrained creep of clay. Whether or not it is pre-failure, needs to be examined.

It may be pointed out that the study of rate effect in triaxial compression and undrained creep tests has not been done yet on Singapore marine clay and the results to be later presented in this thesis will be an important addition to the already available vast database for different soils on rate effect and undrained creep. It will also serve to better understand the stress-strain behaviour of Singapore marine clay.

CHAPTER 3

EXPERIMENTAL SET-UP, MATERIALS AND METHODOLOGY

3.1 INTRODUCTION

A new generation triaxial machine, the so-called Motorized Triaxial Cell, was used to carry out the compression and extension tests. In this chapter, various measuring equipment used to record the data are discussed. A description of types of soils used for studying the drained and undrained behaviour in compression and extension is also presented. The basic properties of the three main cohesive soils, namely undisturbed Singapore marine clay, reconstituted Singapore marine clay and Kaolin, are also discussed. Methodology adopted to conduct various types of tests is briefly described.

3.2 THE MOTORIZED TRIAXIAL CELL

The Motorized Triaxial Cell used in this study was manufactured by GDS Instruments Limited. A picture of the motorized triaxial cell is shown in Fig. 3.1. The Motorized Triaxial Cell is an improvement to the conventional Bishop and Wesley Stress Path Cell in the sense that either load or deformation can be controlled independently. Thus it can be used to conduct both load-controlled and deformation-controlled tests in the same triaxial cell. The axial force is provided by a direct screw drive to actuate the base pedestal through the bottom of the cell, enabling the axial strain and therefore the axial force up to 7kN to be applied directly. The submersible

load cell is fixed in its position. The motorized loading system makes the axial load and displacement control more accurate and reliable. This feature of the cell also helped in conducting the tests in load-controlled and deformation-controlled modes, for studying the effects of rate and mode of loading on the behaviour of clay. The load can be measured and controlled to an accuracy of ± 1 Newton. The accuracy of measured displacement is ± 1 micrometer. Motorized cell was supplied with a specially designed extension top cap which could replace the loading ram of the load cell to enable the machine to conduct extension tests.

Motorized Triaxial Cell provides a test chamber (Fig. 3.1) where the test conditions may be accurately controlled for testing 38 mm and 50 mm samples at cell pressure of up to 1700 kPa. It comprises a control box with keyboard and digital display for targeting and reading the load or displacement. It also enables data recording into the computer via IEEE interface.

Fig. 3.1: The motorized triaxial cell (picture taken from GDS motorized triaxial cell handbook)

The motorized cell is also fitted with a displacement transducer that measures displacement directly from the movement of the axial ram. Free ports are available for connecting additional transducers and measuring devices.

The complete experimental set-up is shown in Fig. 3.2. A schematic arrangement of the experimental set-up is shown in Fig. 3.3. It consisted of the motorized triaxial cell, one GDS digital pressure volume controllers (DPVC) for back pressure and volume control, one DPVC for cell pressure control, motorized cell control box to control the load and displacement, two pore pressure transducers, one miniature pore pressure transducer, two submersible LVDTs, a data-logger and a personal computer.

3.3 INTERFACE WITH THE HOST COMPUTER

The host computer connects all the components of the experimental set-up. Digital pressure volume controllers and motorized triaxial cell were directly connected to the computer via IEEE port. Data from transducers was recorded through a data logger. Two external pore water pressure transducers, one submersible miniature pore pressure transducer, two submersible LVDTs and an external LVDT, that were used to record various parameters, were connected to the data logger.

3.4 LOADING SYSTEM AND DISPLACEMENT CONTROL

The desired rate of loading or rate of displacement could be applied using the digital load/displacement control box. The values of load and displacement were recorded from the control box by the computer.

3.5 PRESSURE AND VOLUME CONTROL

3.5.1 Cell Pressure Control and Measurement

The cell pressure was controlled using a digital pressure volume controller (DPVC), as indicated in Fig. 3.2. The cell pressure was measured to an accuracy of 1 kPa. The volume change and cell pressure were recorded from DPVC by computer using the

control program. The DPVC could apply a pressure of upto 2000 kPa. The desired cell pressure could be either targeted manually or by sending a command from computer.

Fig. 3.2: Photograph of the complete experimental set-up

3.5.2 Back Pressure Control and Pore Water Pressure Measurement

A digital pressure volume controlled was used to apply the back pressure from top and bottom of the specimen (Fig. 3.2). The back pressure controller, having a capacity of 3000 kPa, was used for both pressure and volume control and measurement depending upon the type of test conducted. The minimum volume change was 1 mm^3 , which corresponds to a volumetric strain of $5.1 \times 10^{-4} \%$ of the specimen. The minimum pressure change was 1 kPa.

1. Load cell 2a. Submersible LVDT -1 2b. LVDT – 2 (hidden behind)
3. Mid pwp transducer 4. O-rings to seal mpwp transducer
5. Demodulator for LVDT 6. Top pwp transducer
7. Bottom pwp transducer 8. External displacement transducer

Fig. 3.3: A schematic diagram of experimental set-up

Additionally, two pore pressure transducers were used to measure the pore pressures at the top and bottom of the specimen. These transducers had a measuring capacity of 1000 kPa. One submersible miniature pore pressure transducer (see Fig. 3.4) of 700 kPa capacity was used to measure the pore pressure at the mid-height of the specimen. The air-entry value of the transducer was about 100 kPa. This transducer was in direct contact with the specimen. A small rubber grummet was fixed into a hole made at the centre of the rubber membrane prior to the test. The miniature pore pressure transducer was installed into the grummet and sealed with the help of two small O-rings. Being in direct contact with the specimen, it gave quick response (Hight, 1983) for any change in pore pressure within the specimen. The mid-pore pressure measurement enabled the monitoring of excess pore pressure at the mid-height of the specimen during both undrained and drained tests.

Whenever not in use, the triaxial cell was filled with de-aired water and pressurized to about 200 kPa while keep the miniature pore pressure transducer submerged. This ensured the saturation of the transducer during test. Top and bottom pore pressure transducers were regularly de-aired using the de-airing blocks by running the de-aired water through the transducer.

Fig. 3.4: Setting up of mid pore pressure transducer and internal LVDTs

3.6 AXIAL STRAIN MEASUREMENT

One external Linear Variable Differential Transformer (LVDT) and a pair of submersible LVDTs were used for axial strain measurement. The external LVDT (see Fig. 3.3) had a maximum range of 100 mm and it measured displacement directly from the movement of axial ram. The motorized cell control box also calculates the displacement of the pedestal from the rotation of the stepper motor with a resolution of 1 micrometer. This value can be read from LCD of control box and can be electronically recorded by control program.

A pair of submersible LVDTs was used for local strain measurement (Fig. 3.4). These internal LVDTs were used for measuring the initial displacement in the sample upto about 1 % axial strain. This allowed the small strains to be recorded during the crucial beginning stage of the tests. The LVDTs and spindle were attached to the specimen using small LVDT holders. These were attached to the membrane on the specimen, diametrically opposite to each other, using super glue.

3.7 SOFTWARE FOR CONDUCTING TRIAXIAL TESTS

A computer program, originally written to conduct stress and strain path tests was further modified to conduct undrained creep tests. The program was used to record the data from data-logger, DPVCs and load controller.

3.8 SOILS FOR TESTING

The tests were performed on three different soils, which included Kaolin, undisturbed Singapore marine clay and clay reconstituted from undisturbed marine clay. All the test specimens were 100 mm in height and 50 mm in diameter. The basic properties, i.e. specific gravity, liquid limit, plastic limit, permeability (at $\sigma_v' = 200$ kPa) and initial water content of all the clays used for tests are presented in Table 3.1.

Table 3.1: Basic properties of clays used for triaxial tests

Soil type	Specific gravity	Liquid limit (%)	Plastic limit (%)	Permeability (m/s) (at $\sigma_v' = 200$ kPa)	Initial water content (%)
Undisturbed marine clay	2.66	70 ± 2	32 ± 2	7×10^{-9}	54 ± 2
Reconstituted marine clay (from undisturbed clay)	2.66	70 ± 2	32 ± 2	4×10^{-9}	48 ± 1
Kaolin	2.42	74	33.5	2×10^{-5}	58.5 ± 1

3.8.1 Undisturbed Marine Clay Samples

The undisturbed marine clay samples were retrieved from a land reclamation site at Changi East in Singapore. Samples were retrieved using thin-walled piston sampling tubes from depths varying from 15 to 34 meters. However, only the sample tubes from depths between 20 to 23 meters taken from nearby boreholes were used for this research so that the samples were most likely from same stratum of clay. Some tubes were of 100 mm diameter while others were of 70 mm diameter. The length of extruded sample was between 600 mm and 800 mm. The extruded sample was cut into smaller samples of lengths of around 130 mm and was wrapped in aluminium foil, waxed and stored in the moist room. Some samples had small pockets of peat which were carefully eliminated while trimming the specimens. The clay was grey in colour. The basic properties of this clay are listed in Table 3.1. Since the clay is from a reclaimed land, it is still undergoing consolidation (so called under-consolidated clay) under the weight of reclaimed sand fill. The vertical effective stress at 20 to 23m depth is between 160 to 190 kPa, which should also be the pre-consolidation pressure as the clay is undergoing consolidation. 1-D consolidation tests in oedometer and results from consolidation stage in anisotropically consolidated undrained test (CAU or K0CU) also showed a maximum past pressure or pre-consolidation pressure (σ_{vp}') of 170 to 195 kPa.

3.8.2 Reconstituted Marine Clay Samples

The reconstituted marine clay was reconstituted from the trimmings of undisturbed marine clay samples described in section 3.8.1. The clay was clean and had no shell fragments. The clay was soaked in water overnight before mixing in tank for preparing the slurry. The clay was reconstituted by mixing the clay with water to form slurry with a moisture content of 125%, which was about 1.8 times the liquid limit (70%). The slurry was prepared in a large mixing tank by mixing for 5-6 hours. The slurry was consolidated in a consolidation tank of 300 mm diameter and 600 mm height under a consolidation pressure of 190 kPa. A filter paper and geosynthetic fabric was placed on the top and bottom of the slurry to prevent loss of any fines. The drainage of water took place from both top and bottom of the slurry. Consolidation was considered completed when the drainage of water ceased. The consolidated sample, which had a diameter of 300 mm and a height of 140-160 mm was extruded and cut to get samples of 75mm x 75mm cross section each. The samples were properly wrapped in plastic sheets and aluminium foils. The wrapped samples were sealed by waxing and were kept in a moist room. Each sample was later trimmed before the test to get a cylindrical test specimen of 50 mm diameter and 100 mm height. The basic properties of this clay are listed in Table 3.1.

3.8.3 Kaolin Samples

Commercial Kaolin powder was used to prepare specimens for testing. The procedure for preparing the slurry, consolidation, extrusion and storage of Kaolin samples was same as that for reconstituted marine clay. The basic properties of this Kaolin clay are given in Table 3.1.

It is important to highlight that the initial water contents of undisturbed marine clay, reconstituted marine clay and Kaolin were different from each other. This is mainly due to the fact that both Kaolin and reconstituted marine clay block samples were prepared by consolidating the slurry having high water content of 125% under consolidation pressure of 190 kPa in a large consolidation tank. This ensured that the pre-consolidation pressure of resulting samples was close to the estimated pre-consolidation pressure of undisturbed marine clay. However, in this method, water content can not be controlled.

3.9 SPECIMEN SET-UP

The tests were carried out on specimens of 50 mm diameter and 100 mm height. During the specimen set-up a porous stone and filter paper were placed over the base pedestal of motorized triaxial cell. Filter papers and porous stone were also kept on top of the specimen. The rubber membrane was placed over the specimen using a splitting type membrane stretcher. A pair of O-rings was placed over the membrane on top cap and the base pedestal to prevent water leakage into or out of the specimen. Two internal LVDTs were attached (Fig. 3.4) at diametrically opposite sides to the specimen using super glue in such a way that the gauged length was about 40 mm. After installing the LVDTs, the miniature pore pressure transducer was installed at the mid-height of specimen as described in section 3.5.2. The outer cover of the triaxial cell was then lowered and the water was filled in the cell.

3.10 TESTING PROCEDURE

The tests were generally conducted in 3 stages, namely, saturation, consolidation and shearing as described below.

3.10.1 Saturation

The saturation of the system was achieved mainly by the use of back pressure. A back pressure of 300 to 400 kPa was used. A difference of 10 kPa was maintained between the cell pressure and back pressure during the increment of pressures to attain the desired final cell pressure and back pressure. The pore pressures at the top, bottom and mid-height were closely monitored during this process.

To check the degree of saturation at any stage B-value check was performed by increasing the cell pressure by 20 kPa while keeping the back pressure valve closed. When value of $\Delta u / \Delta \sigma_3$ (B-value) as measured from top and bottom pore pressure changes was found to be greater or equal to 0.95, the saturation was considered complete. It was observed that the B-value was greater than 0.98 when Δu was calculated from the changes in mid-height pore pressure. The pore pressures at the

top and bottom of the specimen were found to be slightly lower (1-2 kPa) than that at the mid-height.

3.10.2 Consolidation

Consolidation of specimens was carried out either isotropically or anisotropically. For isotropic consolidation, the cell pressure was raised (while keeping the back pressure valve closed) to reach a pre-determined effective confining stress. The pore pressure increased due to the increase in cell pressure since the back pressure valve was kept closed. Consolidation was started by opening the back pressure valves. The volumetric strain and mid pore pressure change with time were monitored during the consolidation. The consolidation was considered complete when volume change rate became almost zero and the mid pore pressure reduced to become equal to the back pressure. A typical volume change versus time curve is shown in Fig. 3.5 from one of the tests (CR9). It can be seen that the volume change ceases at the end of consolidation.

Fig. 3.5: Volume change during isotropic consolidation

For anisotropic consolidation of the specimens, the specimens were sheared at a slow rate of shearing while maintaining a strain increment ratio equal to one ($d\varepsilon_v/d\varepsilon_1 = 1$), a technique developed by Chu (1991) and Lo and Chu (1991). The strain increment ratio was controlled by controlling the volume change of the specimen so that the change in volumetric strain was equal to the change in axial strain.

When, $d\varepsilon_v/d\varepsilon_1 = 1$; $d\varepsilon_v = d\varepsilon_1$, that is, $d\varepsilon_3 = 0$, then K_0 condition is achieved.

The specimens were left for creep at constant load in K_0 condition upon reaching the desired mean effective stress to avoid development of excess pore pressure due to switching from K_0 compression effective stress path to undrained effective stress path for undrained shearing.

3.10.3 Shearing

Shearing was performed in two different loading modes, deformation-controlled (DC) and load-controlled (LC) loading modes. Isotropically consolidated undrained compression (CIUC), isotropically consolidated drained compression (CIDC) and undrained creep tests were conducted on all the three soils. Some strain path tests and some isotropically consolidated undrained extension (CIUE) tests were conducted on Kaolin specimens only. In the conventional CIUC and CIDC tests, the samples were sheared till failure occurred, which was identified by the development of shear band.

The time to failure during shearing for CIUC and CIDC tests was determined using volume change data during isotropic consolidation, according to BS 1377:Part 8 (1990). Detailed calculations for undisturbed marine clay, reconstituted marine clay and Kaolin are shown in Appendix A. The method is illustrated here for undisturbed marine clay.

According to BS1377 (1990), as the first step, the volume change data during isotropic consolidation is plotted against square root of time, as shown in Fig. 3.6 for undisturbed marine clay. A straight line is then drawn through the initial linear portion of the plot to intersect the horizontal line drawn through the final point on

plot. (In Fig. 3.6, final point on the plot appears to be close to the end of consolidation) The time corresponding to the point of intersection of the two lines is taken as t_{100} . Significant testing time, t_f , is then determined as

$$t_f = Ft_{100}$$

where, F is a coefficient which depends on drainage conditions and the type of compression test, i.e. undrained or drained. For drainage from both ends, BS1377 (1990) recommends the use of values 8.5 and 2.1 for drained and undrained shearing respectively.

Fig. 3.6: Determination of T_{100} for the calculation of time to failure for undisturbed Singapore marine clay

The second step involves choosing the significant strain interval (ϵ_f), which, in current case, happens to be the strain at failure. However, the strain at failure can be accurately determined only after shearing. Hence one has to choose the strain at failure from past experience from the soil to be tested. This may lead to errors in selection of rate, especially if data from previous studies are not available or if the behaviour of soil has not been previously studied. A possible solution for this problem is discussed in subsequent chapters in this thesis.

The third step is to estimate the rate of axial displacement (d_r , in mm/min), which is determined using the following formula

$$d_r = \frac{\epsilon_f \times L_c}{t_f}$$

where, L_c is the length of consolidated specimen.

The time to failure and shearing rates estimated using the above method for the three tested soils are listed Table 3.2. Assumed axial strains at failure (ϵ_f) are also listed. Discussions on these assumptions are made in subsequent chapters of this thesis. The values of axial strain rate in Table 3.2 are referred to as the ‘maximum permissible axial strain rate’ in this thesis because rates higher than these are expected to result in non-equalization of pore pressure within the specimen.

Table 3.2: Estimated shearing rates in triaxial tests based on BS 1377 : Part 8, 1990

Soil type	CIUC tests			CIDC tests		
	Time to failure (min)	Assumed ϵ_f (%)	Maximum Permissible Strain rate (mm/min)	Time to failure (min)	Assumed ϵ_f (%)	Maximum permissible strain rate (mm/min)
Undisturbed marine clay	470	2.5	0.005	1910	25	0.013
Reconstituted marine clay	1760	7	0.004	7150	22	0.003
Kaolin	40	7	0.175	165	21	0.13

In this thesis, strain rates higher and lower than the maximum permissible axial strain rates estimated in Table 3.2 were also used during shearing of the three clays to study the rate effects on the behaviour of clay.

In undrained creep and constant stress drained (CSD) compression tests the specimen was sheared in an undrained condition up to a certain pre-determined value of deviator stress, which was below failure observed in CIUC test. After this, the load was kept constant and specimen was allowed to undergo undrained creep.

In undrained extension tests, a special extension top cap system was used. The loading ram used during compression test was replaced with extension cap. During shearing, the top of the specimen was fixed and the base pedestal was moved down, thus applying the extension force on the specimen.

It should be pointed out that the results presented in this thesis are only from representative tests showing the generally observed trends. Numerous tests were repeated to examine the repeatability of test results, especially for Kaolin and reconstituted marine clay. Fewer tests were repeated for undisturbed marine clay due to the scarcity of samples. For Kaolin and reconstituted marine clay, similar series of tests were conducted on specimens consolidated under different effective confining pressures (typically, 200/300/400 kPa). However, only the results of tests on specimens consolidated under 200kPa and 300kPa respectively will be presented for reconstituted marine clay and Kaolin because the trends observed from different series of tests were similar.

3.11 SUMMARY

The experimental set-up used in this research has been described in this chapter. The GDS Motorized Triaxial Cell, used for conducting the tests, was described. Basic properties of the soils used for the tests have also been presented. Methodology adopted to conduct various types of tests was discussed.

CHAPTER 4

EFFECT OF LOADING RATE AND LOADING MODE ON THE STRESS-STRAIN BEHAVIOUR OF UNDISTURBED MARINE CLAY

4.1 INTRODUCTION

As discussed in Chapter 2, the strength and stress-strain behaviour of most of the clays has been found to be highly rate sensitive (Arulanandan et al., 1971; Hicher, 1988; O' Reilly et al., 1988). The undrained shear strength of clays has been found to increase with the increase in axial strain rate and it has been commonly observed that the undrained shear strength of clay increases by about 5% to 15% due to a 10-fold increase in axial strain rate (Singh and Mitchell, 1969; Hicher, 1988; Flavigny and Nova, 1990). Many studies have indicated that the increase in undrained shear strength could be related to a decrease in pore water pressure with the increase in shearing rate (Gibson and Henkel, 1954; Lacasse, 1995; Richardson and Whitman, 1963; Berre and Bjerrum, 1973; Sheahan et al., 1996; Lefebvre and LeBoeuf, 1987; Zhu et al., 1999; Teachavorasinskun et al., 2002). However, some studies have shown no significant change in pore water pressure with the change in shearing rate (Bjerrum et al., 1958; Alberro and Santoyo, 1973; Akai et al., 1975; Fourie and Dong, 1991). Lefebvre and LeBoeuf (1987) attributed the change in pore water pressure with change in loading rate to delayed development of pore water pressure and measurement errors. Sheahan et al. (1996) emphasized that the study of rate effect should be carried out with proper pore water pressure measurement to properly understand the mechanism responsible for the rate effect.

Loading mode, that is, whether a test is conducted under a load-controlled or deformation-controlled loading mode, also affects the behaviour of cohesive soils, although the studies in this aspect have been limited (Lundgren et al., 1968; Vaid, 1988; Lacasse, 1995; Teachavorasinskun et al. 2002). Drained behaviour of cohesive soils also has been found to be affected by the loading rate (Gibson and Henkel, 1954; Roy and Sarathi, 1976; Newson et al., 1997). However, the mechanisms responsible for the rate effect and mode effect are not yet clear.

The effects of loading rate and loading mode on the stress-strain behaviour of Singapore marine clay have not been properly studied. Only the effects of shearing rate on the 1-D consolidation behaviour of Singapore marine clay in constant rate of strain oedometer tests were studied by Lee et al. (1993). In Lee et al.'s tests, the pore water pressure was measured at the base of the oedometer specimen. It was observed that there was a large difference between e - $\log p'$ curves determined using pore water pressures at undrained and drained face of specimen. The effect of shearing rate on the stress-strain behaviour of Singapore marine clay in triaxial tests has not been reported. In this research program, isotropically consolidated undrained compression (CIUC) tests were conducted at various rates to study the rate-effect on the undrained behaviour of undisturbed Singapore marine clay. Both load-controlled and deformation-controlled CIUC tests were conducted. The results are presented in this chapter. A comparison between the results of load-controlled and deformation-controlled tests is made to analyze the effect of loading mode on the undrained behaviour of undisturbed marine clay. Results of two drained tests are also presented.

Most of the tests reported in this chapter were conducted on isotropically consolidated specimens of undisturbed Singapore marine clay. The basic properties of the tested undisturbed Singapore marine clay have been presented in Table 3.1 of Chapter 3.

4.2 FAILURE ENVELOPE

A summary of all the CIU, CAU and CID tests on undisturbed Singapore marine clay is given in Table 4.1. The failure envelope for the clay was determined using 3 undrained and one drained tests as shown in Fig. 4.1a. Undrained tests CIU1 and

HCU1 were conducted on specimens isotropically consolidated under 200 kPa and 300 kPa respectively. Test CAU2 was conducted on specimen anisotropically consolidated to a mean effective stress of 192 kPa. The drained compression test was conducted on a specimen isotropically consolidated under an effective confining pressure of 200 kPa.

Table 4.1: Summary of tests on undisturbed marine clay

Test no.	Shearing rate	Unit	p_o' (kPa)	W % (pre test)	W % (post test)	q_m	$(q/p')_{mid}$ at failure	ϕ' at failure [^] (mid)	ϕ' at peak* (mid)	ϕ'_{av} at peak [#] (top & base)	ϵ_{peak}
CIU1	0.005	mm/min	200	57.9	55.1	117.5	0.986	24.8	19.8	20.4	1.699
CIU2	0.05	mm/min	200	57.9	55.2	121.7	0.813	20.9	20.2	21.4	2.431
CIU3	0.5	mm/min	200	58.5	54.9	132.0	0.833	21.5	21.2	23.6	3.142
CIU4	0.5	N/min	200	56.2	54.6	118.5	0.901	23.1	23.1	23.6	3.707
CIU5	5	N/min	200	57.6	54.3	122.8	0.857	22.0	21.1	22.5	2.723
CIU6	50	N/min	200	57.9	54.4	130.7	0.878	22.5	22	24.7	3.685
CAU2	0.005	mm/min	192	58.2	55.3	112.1	0.989	25.1	18.9	18.8	0.816
HCU1	0.005	mm/min	300	57.8	51.5	148.8	0.903	24.0	18.8	19.3	3.766
CID1	0.002	mm/min	200	57.6	48.2	212.7	0.801	20.7	20.7	20.2	29.94
CID2	0.01	mm/min	200	56.6	48.5	181.5	0.776	20.1	20.1	18.2	20.4

[^] ϕ' at failure is the effective friction angle at maximum σ_1'/σ_3' (or maximum q/p')

* ϕ' at peak is the effective friction angle at maximum $(\sigma_1' - \sigma_3')$ (or maximum q)

[#] ϕ'_{av} at peak is the average of effective friction angles (at maximum $\sigma_1' - \sigma_3'$) calculated using σ_1' and σ_3' estimated from pore pressures at top and base of the specimen

In undrained tests, the peak deviator stress was attained at only 1% to 4% axial strain (Fig. 4.1b). This was followed by significant amount of strain softening. The value of q/p' reached its maximum value well after the peak deviator stress (Fig. 4.2c). The specimen failed mainly by bulging and shear band occurred after large axial strain. A notable drop in load and q/p' was recorded after the shear band. The pore water pressure (Fig. 4.2d) seemed to reach an almost constant value towards the end of the test in the undrained tests.

The failure envelope was determined from the points at maximum q/p' . The failure envelope within the tested stress range (BE in Fig. 4.1a) is curved. Curved failure

envelopes have earlier been observed for many other clays (Lo and Morin, 1972; Burland et al., 1996; Asghari et al., 2003; Baker, 2004). The failure envelope in the low effective confining stress range is assumed to be curved and passing through the origin, as shown by dotted curve AB in Fig. 4.1a. This assumption does not affect the analysis of results presented in this Chapter and Chapter 5 because the failure in all the tests occurred in the zone marked with CD in Fig. 4.1a. The envelope AE is chosen as the reference failure envelope for comparison with all the subsequent tests on undisturbed marine clay.

In drained test, large plastic yielding occurred at about 2% axial strain at point 'Y' as shown in Figs. 4.1a, 4.1b and 4.1c. The deviator stress kept increasing until the peak strength was achieved at about 24% axial strain. The deviator stress and the ratio, q/p' , remained almost constant at peak over an axial strain of about 5%. Failure occurred when the shear band developed. The shear band was accompanied by a drop in q/p' and is indicated by the 'x' signs in Fig. 4.1c. The volume change (Fig. 4.1d) continued throughout the test although it seemed to reduce towards the end of the test.

Fig. 4.1a: Failure envelope for undisturbed marine clay determined from CIU and CID tests

Fig. 4.1b: Stress-strain curves in various CIU and CID tests

Fig. 4.1c: Variation of q/p' with axial strain

Fig. 4.1d: Pore pressure and volume change variation in undrained and drained tests

4.3 EFFECT OF SHEARING RATE ON THE UNDRAINED BEHAVIOUR IN DEFORMATION-CONTROLLED TESTS

Three isotropically consolidated undrained (CIU) tests, CIU1, CIU2 and CIU3, were conducted in deformation-controlled loading mode at axial strain rates of 0.005, 0.05 and 0.5 mm/min. The results of the three deformation-controlled tests are presented in Figs. 4.2a to 4.2l. All the specimens were isotropically consolidated under an effective confining pressure of 200 kPa. Thus, all the specimens were normally consolidated (pre-consolidation pressure, $p_c' = 170$ to 190 kPa). The maximum permissible axial strain rate (see Chapter 3, section 3.10.3), as determined using volume change versus time curve from isotropic consolidation data (BS1377, 1990), was 0.005 mm/min (see Appendix A for calculations). The axial strain at failure (ϵ_f) was assumed to be 2.5% for calculating the maximum permissible strain rate.

4.3.1 Strength and Stress-Strain Behaviour

A comparison of the stress-strain behaviours observed in the three tests is shown in Fig. 4.2a. It can be observed that the peak deviator stress increased with the increase in axial strain rate. Also, the axial strain at peak deviator stress increased with the increase in axial strain rate. There was an increase of about 0.8% in axial strain at peak with a 10-fold increase in axial strain rate. Increase in axial strain at peak with the increase in rate has been earlier reported by Richardson and Whitman (1963) and Bjerrum et al. (1958) while others (Alberro and Santoyo, 1973; Vaid and Campanella, 1977; Sheahan et al., 1996) observed no significant change in axial strain at peak. It may be noted that the peak deviator stress was followed by strain softening (Fig. 4.2a) and failure (maximum q/p' and shear band formation) occurred after a large strain softening. However, in faster tests CIU2 and CIU3, shear band occurred after small strain softening and was accompanied by steep drop in deviator stress (shown by 'x' in Fig. 4.2a). It may be noted that the drop in q was steepest in the fastest test CIU3. The shear band was identified by the decrease in ratio q/p' . The identification of shear band will be further elaborated in section 4.3.3.

The peak deviator stresses obtained in the three deformation-controlled tests are plotted versus the axial strain rate in Fig. 4.2b. The peak deviator stress increased with the increase in axial strain rate. There was a linear increase in peak deviator stress with the logarithmic increase in axial strain rate as shown in Fig. 4.2b. It shows an average increase of about 6% in peak deviator stress per log cycle increase in axial strain rate. The values of peak deviator stress in the three tests are given in Table 4.1. The increase in axial strain rate by 10 times (0.05 mm/min) and 100 times (0.5 mm/min) over the slowest strain rate (0.005 mm/min) caused an increase of 3.6% and 12.3% respectively in the peak deviator stress. It implies similar increase in the undrained shear strength ($s_u = q_m/2$). The increase in undrained shear strength of clay with increase in axial strain rate is a well known phenomenon observed by many researchers, e.g. Gibson and Henkel (1954), Graham et al. (1983), Augustesen et al. (2004) etc. It may be mentioned that the percentage change in undrained shear strength increased with the increase in shearing rate. That is, the peak deviator stress increased by only 3.6% when axial strain rate increased from 0.005 mm/min to 0.05 mm/min while there was an increase of 8.5% when axial strain rate was increased

further to 0.5 mm/min. Similar observation has been made by some earlier studies (Akai et al., 1975; Sheahan et al., 1996). However, the increase in peak deviator stress (6% per log cycle of strain rate) due to the rate-effect on Singapore marine clay appears to be lesser as compared to the other clays reported in literature. For instance, the observed increase in undrained shear strength per log cycle of axial strain rate was 9% for Mexico City Clay (Alberro and Santoyo, 1973), 8% for Haney Clay (Vaid and Campanella, 1977) and 10 to 13% for three types of Canadian clays (Lefebvre and LeBoeuf, 1987). The axial strain rates used in these studies were typically between 0.0007%/min to 2%/min. Sheahan et al. (1996) observed an increase of 6% to 10% in undrained shear strength per log cycle of axial strain rate for resedimented Boston Blue Clay for axial strain rates between 0.0008%/min to 0.8%/min.

Fig. 4.2a: Stress-strain curves in DC tests on undisturbed marine clay

Fig. 4.2b: Variation of peak deviator stress with axial strain rate in DC tests

4.3.2 Variation of Pore Water Pressure

The pore water pressures were measured at the top, bottom and mid-height of the specimens in all the tests. Since the pore water pressure measured at the mid-height is considered to be more accurate than the end measurements (Richardson and Whitman, 1963; Fourie and Dong, 1991) and mid-height probe has the shortest response time (Hight, 1982; Sheahan et al., 1996), the pore pressures measured at the mid-height are most representative and have been used to compare the pore pressure responses in different tests, as shown in Fig. 4.2c. It is important to mention here that the 'change in pore pressure', as plotted on Y-axis in Fig. 4.2c and subsequent figures in this thesis, refers to the excess pore pressure developed during shearing, i.e. the total pore pressure inside the specimen minus the applied back pressure. Fig. 4.2c shows that as the axial strain rate increased, the mid-height pore water pressure decreased. This led to a higher effective confining stress during shearing and thus a higher peak deviatoric stress. Similar observation had been earlier made for other clays (Richardson and Whitman, 1963; Alberro and Santoyo, 1973; Sheahan et al., 1996; Zhu et al., 1999). However, some authors (Lefebvre and LeBoeuf, 1987)

attributed it to the measurement error due to the pore pressure being measured at the top or bottom of the specimen only. The results from undisturbed Singapore marine clay show that the decrease in pore pressure with increase in rate is true soil behaviour. Also, the observation that for a given axial strain, the pore water pressure is higher in slower test (Fig. 4.2c) shows the phenomenon of creep in pore water pressure (Sheahan et al., 1996). The higher the time taken to reach a certain axial strain, the higher is the pore pressure due to the creep induced pore pressure.

For element tests, the results are only meaningful when the stress and strain distributions within the specimen are essentially uniform. Since in this study the pore water pressure was measured at the top, bottom and middle of the specimen, the uniformity of pore water pressure and effective stress distribution could be checked. To examine the pore water pressure distribution within the specimen in the three deformation-controlled tests, top, bottom and mid-height pore water pressures in each test are plotted in Figs. 4.2d, 4.2e and 4.2f. These figures show that the pore water pressure distribution within the specimen becomes increasingly non-uniform as the axial strain rate increases. The highest non-uniformity was observed in the fastest test CIU3 (0.5 mm/min) (Fig. 4.2f), which showed more than 20 kPa difference between the top and mid pore pressure at peak deviator stress. Thus, the pore water pressure non-uniformity is intolerable when axial strain rate is 0.5 mm/min. It is also evident that the pore water pressures measured at the top and bottom of the specimen over-predict the pore water pressure within the specimen in faster tests. Similar observation was made by Fourie and Dong (1991) for undrained compression tests on black clay from Australia. Although, an indication of the pore water pressure non-uniformity can be obtained if both the top and bottom pore water pressures are measured, the mid-height pore water pressure measurement gives more consistent results and makes it easier to interpret the results of tests. For example, in test CIU2 (Fig. 4.2e), while the top and bottom pore water pressures show only a small difference till shear band formation, the mid-height pore pressure is much lower than the top and bottom pore pressure, thus showing larger non-uniformity.

Another notable feature of the pore pressure variations in tests CIU2 and CIU3 in Figs. 4.2e and 4.2f is that there was a sudden change in pore water pressure after the formation of the shear band. For example, in test CIU2 (Fig. 4.2e), the top pore

pressure increased rapidly after the shear band formation, while bottom and mid pore pressures increased at a smaller rate than before the shear band formation. Similarly, in test CIU3 (Fig. 4.2f), bottom pore pressure increased rapidly, top pore pressure dropped and mid pore pressure increased at lesser rate after the shear band. There was no obvious change in pore pressure variation pattern in test CIU1 (Fig. 4.2d) since the specimen failed mainly by bulging.

It should be mentioned that at the threshold shearing rate of 0.005 mm/min (test CIU1), which was calculated from BS1377 (1990) (see Appendix A), pore pressure distribution within the specimen was reasonably uniform (Fig. 4.2d). Pore pressure distribution became non-uniform at the axial strain rates higher than this (Figs. 4.2e and 4.2f). It can be concluded that the estimation of time to failure in undrained shearing using BS1377 (1990) is fairly accurate. However, this method only estimates the ‘time to failure’ and the shearing rate will depend upon the chosen axial strain at failure (see Appendix A), which one has to assume before conducting the test. Considering the fact that the axial strain at failure changes with the axial strain rate (see Fig. 4.2a), it would be better to conduct the test with proper pore pressure measurement at a chosen rate to estimate the homogeneity of pore pressure distribution within the specimen and verify the accuracy of the chosen rate.

Fig. 4.2c Variation of mid-height pore pressures in deformation-controlled tests

Fig. 4.2d: Variation of pore pressure at top, bottom and mid pore pressure in test CIU1

Fig. 4.2: Variation of pore pressure at top, bottom and mid pore pressure in (e) test CIU2, (f) test CIU3

4.3.3 Effective Stress Paths

The effective stress paths for the three tests CIU1, CIU2 and CIU3 as determined using the pore water pressure measured at the mid-height are shown in Fig. 4.2g. It can be seen from Fig. 4.2g that the stress paths from tests CIU2 and CIU3 did not reach the failure envelope determined earlier in Fig. 4.1a and reproduced in Fig. 4.2g. An early failure of specimens, indicated by a drop in q/p' , as shown in Fig. 4.2h, occurred in faster tests CIU2 and CIU3. Failure of the specimen was accompanied by the shear band formation, as shown in Figs. 4.2i. Also, it can be seen from Fig. 4.2i, that the failure mode was affected by the axial strain rate. While the specimen in the slowest test CIU1 failed mainly due to bulging after large axial strain (13%, see Fig. 4.2i), the specimens in tests CIU2 and CIU3 failed due to the formation of shear band at smaller axial strains (3.1% and 3.8% respectively). This shows that a brittle failure occurred at higher axial strain rates and resulted in lesser pre-failure strain-softening (Fig. 4.2g).

It is important to mention that the shear band formation was found to be accompanied by a drop in q/p' . As shown in Fig. 4.2h, the ratio, q/p' , continuously increased at the start of test. In test CIU1, the increase in q/p' was more gradual after the peak deviator stress. This is indicated by the flatter curve close to the maximum q/p' value. q/p' dropped after large bulging and shear band formation. In tests CIU2 and CIU3, the drop in q/p' occurred soon after the peak deviator stress was reached due to the shear band formation. The drop was more rapid in faster test CIU3 than the other two tests (see Fig. 4.2h). The drop in q/p' is used as the method of identification of shear band throughout this thesis. It is also notable (Fig. 4.2g) that the deviator stress also dropped rapidly after the shear band formation. As seen from Fig. 4.2g, the drop in q was gradual in test CIU1 after the peak due to strain softening. However, in tests CIU2 and CIU3, the drop in q was rapid due to shear band and led to rapid drop in q/p' (Fig. 4.2h).

It can also be observed from Fig. 4.2g that the effective stress paths shift towards the right hand side as the rate of deformation increases. This is due to the lower pore water pressure in faster tests, which results in an increase in the mean effective stress. This observation shows that the limit state surface shrinks with the decrease in

axial strain rate. It may be observed from Fig. 4.2g that the shifting of stress path accompanied with the increase in deviator stress tries to bring the failure point close to the curved failure envelope, e.g. the stress path in test CIU3. It shows that when the axial strain rate increases, the peak deviator stress increases, accompanied by an increase in mean effective stress. Thus the failure tends to occur at a higher point on the effective failure envelope in faster tests. However, early failure occurs in faster tests and leads to smaller effective friction angle at maximum obliquity. This can also be seen from the q/p' - ε_1 plot of Fig. 4.2h, which shows higher q/p' value in the slowest test CIU1. It should be pointed out that the effective friction angle at maximum obliquity has been generally reported to be independent of strain rate (Lefebvre and LeBoeuf, 1988; Sheahan et al., 1996). However, Fig. 4.2g (also see Table 4.1) indicates that the failure envelope will be lower in faster test.

It is also notable that the effective friction angle at peak strength (as obtained from q/p' values at points P_1 , P_2 and P_3 in Fig. 4.2g. Also see table 4.1) increases with the increase in shearing rate. Table 4.1 shows that effective friction angles at peak may vary by 1 to 3 degrees due to the change in shearing rate. Similar observation has been earlier reported in some previous studies on natural clay (Bjerrum et al., 1958; Alberro and Santoyo, 1973; Berre and Bjerrum, 1973).

To compare the stress paths determined using the top, bottom and middle of the specimen, the stress paths determined using top and bottom pore pressures are shown in Figs. 4.2j and 4.2k. It can be seen from Fig. 4.2j that if only the top pore pressures were measured, there will be apparently no significant difference between the effective stress paths in the three tests until the peak deviator stress. However, the effective stress paths determined using bottom pore pressures (Fig. 4.2k) show the shifting of effective stress paths towards the right hand side with the increase in shearing rate, which is similar to the observation from the effective stress paths determined using mid-height pore pressures (Fig. 4.2g). This shows that the pore pressures, especially in faster tests, can be more reliably measured if mid-height pore pressure is also measured along with the top and bottom measurements. It is also important to note from Table 4.1 that the effective friction angle at peak can be overestimated by 1 to 3 degrees if top or bottom pore pressures are used for calculation.

It may be highlighted that the increase in pore water pressure non-uniformity with increasing shearing rate makes it difficult to interpret the results of faster tests. This is illustrated by Fig. 4.2l, in which the stress paths determined using the pore water pressures at the three locations in the specimen are plotted together for fastest test CIU3. The stress path determined using mid-height pore water pressure stays away from failure envelope compared to the stress path determined from top or bottom pore water pressures. The stress states at failure at the top and bottom of the specimen are on the failure envelope and this could be the reason for apparent early failure in this test (see Fig. 4.2g). Similar observation was made for test CIU2, in which apparent early failure was observed. This observation indicates that the cause of apparent early failure in the faster tests could be the non-uniformity of pore water pressure, which leads to non-homogenous stress conditions in the specimen. The failure occurred because the stress-states at some points in the specimen reached the effective stress failure state earlier than others. This is also reflected by the fact that the specimens in fast tests failed by formation of shear band at the top or bottom of the specimen (Fig. 4.2i). On the other hand, in slow test CIU1, pore pressure distribution was uniform throughout the specimen and the specimen failed mainly by bulging after large axial strain (see Fig. 4.2i), thus indicating the homogenous deformation of the specimen.

Fig. 4.2g: Effective stress paths determined using mid pore pressure in DC tests

Fig. 4.2h: Variation of q/p' with axial strain in DC tests

Fig. 4.2i: Failed specimens in test CIU1, CIU2 and CIU3

Fig. 4.2j: Effective stress paths determined using top pore pressure in DC tests

Fig. 4.2k: Effective stress paths determined using bottom pore pressure in DC tests

Fig. 4.21: Differences between stress paths determined from top, bottom and mid pore pressures for test CIU3 (0.5 mm/min)

4.4 EFFECT OF SHEARING RATE ON THE UNDRAINED BEHAVIOUR IN LOAD-CONTROLLED TESTS

Three load-controlled undrained compression tests, CIU4, CIU5 and CIU6, were conducted at load increment rates of 0.5 N/min, 5 N/min and 50 N/min respectively. To keep the load-controlled tests comparable with the deformation-controlled tests, the load increment rate was chosen in such a way that the time to reach the load corresponding to the peak deviator stress will remain the same as that in deformation-controlled tests. It was assumed that the peak deviator stress and load will remain the same in both types of tests. The results of these tests are shown in Figs. 4.3a to 4.3r. A comparison of behaviours in load-controlled and deformation-controlled tests will be made later in section 4.4. However, references will be made to the results of deformation-controlled tests wherever necessary.

4.4.1 Stress-Strain Behaviour

The stress-strain curves of the three load-controlled tests CIU4 (0.5 N/min), CIU5 (5 N/min) and CIU6 (50 N/min) are shown in Fig. 4.3a. The peak deviator stress increased with the increase in load increment rate. However, the axial strains at peak deviator stress were similar in the three tests, which can be seen from Fig. 4.3a. The instability occurred at peak q and was followed by quick failure, as shown by the rapid increase in axial strain and axial strain rate in Figs. 4.3b to 4.3g. The failure was characterized by the formation of shear band and was accompanied by a steep drop in q as shown in Fig. 4.3a. The drop in q was sharper in faster tests.

The variation of axial strain and axial strain rate in load-controlled tests is quite different from that in deformation-controlled tests. As evident from Figs. 4.3b to 4.3g, the axial strain and axial strain rate are small at the beginning of load-controlled tests and increase gradually as the test proceeds. At a certain point, the axial strain, as well as the axial strain rate, starts to increase rapidly owing to the inability of soil to sustain any further increase in load. This point is said to be the point of 'onset of instability' (Lade, 1994; Chu and Leong, 2002), as defined earlier in section 2.4.1 in Chapter 2. This point (marked as 'I' in the figures in this thesis) was identified as the point of marked change in the slope of axial strain vs. time and axial strain rate vs. time curves, which, for instance, can be seen in Figs. 4.3b and 4.3c respectively for test CIU4. Similar observations can be made for test CIU5 in Figs. 4.3d and 4.3e and for CIU6 in Figs. 4.3f and 4.3g. It should be mentioned that the axial strain rate at any point was calculated by dividing the increment in axial strain from previous point by the time interval between the previous point and the current point. It can be seen from Fig. 4.3a that the instability occurred at a point close to the peak deviator stress. It is also evident from Figs. 4.3a that the shear band occurred soon after the instability. The points at which shear band occurred are marked on Figs. 4.3b to 4.3g. The axial strain and axial strain rate increased rapidly after the shear band leading to the collapse of the specimen.

Fig. 4.3a: Stress-strain curves for load-controlled tests on undisturbed marine clay

Fig. 4.3b: Variation of axial strain with time in load-controlled test CIU4

Fig. 4.3c: Variation of axial strain rate with time in load-controlled test CIU4

Fig. 4.3d: Variation of axial strain with time in load-controlled test CIU5

Fig. 4.3e: Variation of axial strain rate with time in load-controlled test CIU5

Fig. 4.3f: Variation of axial strain with time in load-controlled test CIU6

Fig. 4.3g: Variation of axial strain rate with time in load-controlled test CIU6

The peak deviator stress, q_m (and hence the undrained shear strength $s_u = q_m/2$), increased linearly with the logarithmic increase in load increment rate, as shown in

Fig. 4.3h. The peak deviator stress increased with the increase in load increment rate by 3.6% and 10.3% respectively for increase in load increment rate of 10 times and 100 times. At an average, the peak deviator stress increased by 5% per log cycle increase in load increment rate. The increase in undrained shear strength with the increase in loading increment rate (for loading rates between 0.01 N/min to 18 N/min) has earlier been reported by Vaid and Campanella (1977) for Haney clay. However, most of the earlier studies were limited to the study of rate-effect on deformation-controlled tests. The results presented here show that the rate-effect on the peak deviator stress in load-controlled tests is similar to the effect on deformation-controlled tests.

Fig. 4.3h: Variation of peak deviator stress with load increment rate

4.4.2 Variation of Pore Water Pressure

The variation of mid-height pore water pressures in the three load-controlled tests, CIU4, CIU5 and CIU6, is shown in Fig. 4.3i. It can be seen that lower the load increment rate, the higher is the pore water pressure. Hence, the increase in peak deviator stress (Fig. 4.3h) is related to the decrease in pore water pressure. This is

consistent with the observation made in deformation-controlled tests. The fact that the pore water pressure at any given axial strain is higher in slow test shows the phenomenon of creep related increase in pore water pressure (Walker, 1969).

The non-uniformity of pore water pressure distribution increased with the increase in load increment rate, as observed from the differences between pore water pressures measured at top, bottom and mid-height of the specimen in Figs. 4.3j, 4.3k and 4.3l for the three load-controlled tests. While bottom pore water pressure is much higher in tests CIU4 (Fig. 4.3j) and CIU6 (Fig. 4.3l), top pore water pressure is higher in test CIU5 (Fig. 4.3k). The inconsistency in variation of top and bottom pore water pressure indicates that the pore water pressures measured at the top and bottom could be seriously affected by the end constraints at high axial strain rates and may not be reliable at fast strain rates. In tests CIU5 (Fig. 4.3k) and CIU6 (Fig. 4.3l), a drop in pore water pressure was recorded after the shear band. This could be related to the phenomena of suppression of pore pressure with the increase in axial strain rate as observed in strain controlled tests in section 4.3.2.

Fig. 4.3i: Variation of mid pore pressure in DC tests

Fig. 4.3j: Variation of pore pressure measured at top, bottom and middle of the specimen in test CIU4

Fig. 4.3k: Variation of pore pressure measured at top, bottom and middle of the specimen in test CIU5

Fig. 4.3l: Variation of pore pressure measured at top, bottom and middle of the specimen in test CIU6

4.4.3 Effective Stress Paths

The effective stress paths for the three load-controlled tests, as determined using pore water pressure measured at the mid-height, are shown in Fig. 4.3m. The failure occurred before the stress path could reach the failure envelope determined from drained and undrained tests in section 4.2. However, the failure stress states tend to approach the failure envelope. It can be observed from Fig. 4.3m that the increase in peak deviator stresses due to the increase in axial strain rate is accompanied by an increase in mean effective stress. Thus the failure seems to be governed by the effective failure envelope of the soil. The rate effect may just be due to the different effective paths followed at different rates. The effective stress states at failure in different tests tend to reach the same effective failure envelope with the faster tests reaching a higher point. However, in load controlled tests, early failure of the specimen occurs.

It is also observed from Fig. 4.3m that the failure, and hence a steep drop in stress path, occurred after small strain softening (also see Fig. 4.3a). Onset of instability occurred before the effective stress state could reach the failure envelope. The instability occurred because in load-controlled tests, the load is not allowed to reduce and the axial strain rate keeps increasing (see Figs. 4.3c, e and g) to increase the load. Failure occurs after the onset of instability with the formation of shear band and results in steep drop in load and hence the deviator stress. The shear band typically occurred slightly after the peak q/p' , as indicated by sharp drop in q/p' vs. strain curves in Fig. 4.3n.

The effective stress paths determined using top and bottom pore pressures are shown in Figs. 4.3o and 4.3p. It can be seen that for the two faster tests CIU5 and CIU6, the effective stress state at failure appears to lie well beyond the failure envelope, which is in contrast with the effective stress path determined using mid-height pore pressures (Fig. 4.3m). This shows that the pore pressures measured at the top and bottom of the specimen in faster tests could be unrealistically high. This could be due to the end constraints due to the rough ends. Hence the pore pressure measurement at the mid-height is more reliable in faster tests, where large non-uniformity of pore pressure occurs.

Fig. 4.3m: Effective stress paths determined using mid-height pore pressure in load-controlled CIU tests on undisturbed marine clay

Fig. 4.3n: Variation of q/p' in load-controlled CIU tests on undisturbed marine clay

Fig. 4.3o: Effective stress paths determined using TOP pore pressure in load-controlled CIU tests on undisturbed marine clay

Fig. 4.3p: Effective stress paths determined using BOTTOM pore pressure in load-controlled CIU tests on undisturbed marine clay

Fig. 4.3q: Stress paths determined using different measurements in test CIU6

To illustrate how the non-uniformity of pore water pressure distribution within the specimens affects the interpretation of results, the stress paths determined using top, mid-height and bottom pore water pressures in test CIU6 (50 N/min) are shown together in Fig. 4.3q. It can be seen that if only the top or bottom pore water pressure were used, effective friction angle would be overestimated.

Photographs of the failed specimens are shown in Fig. 4.3r. Typically, specimen collapses completely in load-controlled test. This type of failure is known to occur in stress controlled or load-controlled testing conditions (Saada, 1967; Lundgren et al., 1968). However, in the load-controlled tests discussed in this chapter, the tests were terminated after the shear band had fully developed and significant drop in deviator stress had taken place. The complete collapse of the specimen was not allowed to prevent any damage to the testing equipment. One interesting feature of the results was that the pore pressure recorded (Fig. 4.3j, 4.3k and 4.3l) was higher at the specimen end on which shear band occurred. For example, the shear band occurred at the bottom of the specimen in tests CIU4 and CIU6 (Fig. 4.3r) and correspondingly the pore pressures at the bottom of the specimen were found to be higher (see Figs.

4.3j and 4.3l). On the other hand, in test CIU5, in which the shear band occurred at the top of the specimen, the pore pressure (Fig. 4.3k) was higher at the top of the specimen. This shows that the non-uniformity of pore pressure results in the formation of localized shear band within the specimen.

(i) CIU4 (0.5 N/min)

(ii) CIU5 (5 N/min)

(iii) CIU6 (50 N/min)

Fig. 4.3r: Failed specimens in test CIU4, CIU5 and CIU6

4.5 COMPARISON OF BEHAVIOURS IN DEFORMATION-CONTROLLED AND LOAD-CONTROLLED TESTS

For the clarity of figures, only two pairs of load-controlled and deformation-controlled tests are compared in Figs. 4.4a to 4.4j. These are: test CIU2 (0.05 mm/min) versus CIU5 (5 N/min) and test CIU3 (0.5 mm/min) versus with test CIU6 (50 N/min). The third pair of tests (CIU1 and CIU4) showed similar trends.

4.5.1 Variation of Axial Strain and Axial Strain Rate

Two of the major differences between deformation-controlled and load-controlled tests are in the variation of axial strain with time and load with time, as shown in Figs. 4.4a to 4.4c. It can be seen that while axial strain (Fig. 4.4a) varies linearly with time in deformation-controlled tests, the axial strain vs. time curves are non-linear in

load-controlled tests. Similar trend is observed in the variation of axial strain rate (Fig. 4.4b). It is notable that instability only occurs in load-controlled tests. It can also be seen from Fig. 4.4b that the axial strain rate continuously increases in load-controlled test and strain rate at peak deviator stress is higher in load-controlled tests than in the corresponding deformation-controlled tests. On the other hand, while there is a linear increase in load in load-controlled test (Fig. 4.4c), the load increases non-linearly in deformation-controlled tests. Load-increment rate is much higher at the beginning of deformation-controlled test than corresponding load-controlled test.

Fig. 4.4a: Comparison of axial strain variation in LC and DC tests

Fig. 4.4b: Comparison of **axial strain rate** variation in LC and DC tests

Fig. 4.4c: Comparison of **load increment** with time in LC and DC tests

4.5.2 Comparison of Stress-Strain Curves

Fig. 4.4d presents the stress-strain curves of the two pairs of deformation-controlled and load-controlled tests. It can be seen from Fig. 4.4d that the stress-strain curves and the peak deviator stresses are quite similar in comparable load-controlled and deformation-controlled tests, e.g. CIU2 and CIU5. Thus, the loading mode does not seem to significantly affect the stress-strain behaviour up to the peak strength.

The average axial strain rate resulting from the chosen load-increment rate in load-controlled test was calculated by dividing the axial strain at peak q with time taken in reaching the peak q . Average axial strain rates corresponding to the load-increment rates of 0.5, 5 and 50 N/min were 0.006, 0.051 and 0.51 mm/min respectively, which are very close to the axial strain rates of 0.005, 0.05 and 0.5 mm/min used in deformation-controlled tests. Thus, a chosen average load-increment rate (based on a deformation-controlled test) resulted in similar average axial strain rate at peak as the axial strain rate used in reference test. This observation gives a method of choosing comparable rates in load-controlled and deformation-controlled tests on undisturbed clay.

The peak deviator stresses obtained in load-controlled tests and deformation-controlled tests are plotted versus axial strain rate in Fig. 4.4e and versus time taken to reach peak in Fig. 4.4f. The peak deviator stress varies linearly with the logarithmic increase in axial strain rate for both load-controlled and deformation-controlled tests (Fig. 4.4e). Conversely, peak deviator stress reduces linearly with the logarithmic increase in time take to reach the peak deviator stress. This shows that the undrained shear strength of undisturbed marine clay is a function of the loading time, regardless of the loading mode used.

The post peak behaviours are different in load-controlled and deformation-controlled tests as shown in Fig. 4.4d. There was a sharper point of peak q (Fig. 4.4d) followed by strain softening in deformation-controlled tests. On the other hand, the stress-strain curve is flatter near the peak for load-controlled tests. Flatter peak in load-controlled test is because in load-controlled test, the experimental system tries to continue increasing the load even after peak deviator stress. In this process, axial

strain rate continuously increases and the specimen is not allowed to soften. It results in a flatter peak and brittle type of failure in load-controlled test.

Fig. 4.4d: Comparison of stress-strain curves in LC and DC tests

Fig. 4.4e: Variation of q_m with axial strain rate

Fig. 4.4f: Variation of q_m with time take to reach q_m

4.5.3 Comparison of Pore Water Pressure Variation

The pore water pressures developed in load-controlled and deformation-controlled tests are compared in Fig. 4.4g. The pore water pressure vs. axial strain curves are similar up to the peak deviator stress in load-controlled and deformation-controlled tests when average axial strain rates are similar. For example, tests CIU2 and CIU5 have similar pore water pressure variation till peak deviator stress. The pore water pressures differ after the peak due to the onset of instability in load-controlled tests. Similar trend is observed for tests CIU3 and CIU6.

The difference between top and mid-height pore water pressures is compared in Figs. 4.4h-(i) and 4.4h-(ii). It is seen that the non-uniformity of pore water pressure distribution within the specimen is higher at the beginning of deformation-controlled tests than in comparable load-controlled tests (e.g. CIU2 & CIU5 and CIU3 & CIU6). This is due to the higher initial axial strain rate in deformation-controlled test than that in load-controlled test as shown earlier in Figs. 4.4a and 4.4b. It may be pointed out that the non-uniformity of pore water pressure increased after the peak deviator stress in load-controlled tests as seen in Fig. 4.4h-(i) and 4.4h-(ii), which is due to the onset of instability and large axial strain rates.

Fig. 4.4g: Comparison of mid-pore pressure variation in LC and DC tests

Fig. 4.4h-(i). Comparison of pore pressure non-uniformity in tests CIU2 and CIU5

Fig. 4.4h-(ii). Comparison of pore pressure non-uniformity in tests CIU3 and CIU6

4.5.4 Comparison of Effective Stress Paths

Effective stress paths of two pairs of load-controlled and deformation-controlled tests are compared in Fig. 4.4i. The difference between the effective stress paths resulting from comparable load-controlled and deformation-controlled tests is small. However, it may be observed from Fig. 4.4i that the effective stress paths in both load-controlled and deformation-controlled tests are unable to reach the failure envelope. For deformation-controlled tests, this appears to be the result of higher non-uniformity of pore water pressure in faster tests, as shown earlier in Figs. 4.2f and 4.2g, which leads to early shear band. For load-controlled tests, this is the result of non-uniformity of pore water pressure (see Fig. 4.3k, 4.3l) caused by increasing axial strain rate due to the onset of instability, which causes shear band to develop early (see Fig. 4.3i). This can be further explained by Fig. 4.4j, which shows the effective stress paths determined using the top pore water pressure measurement. It shows that the effective stress states at failure in top part of the specimen in tests CIU3, CIU5 and CIU6 had crossed the effective failure envelope. Hence the failure occurred.

Therefore, the non-uniformity of pore water pressure within the specimen may lead to pre-mature failure of the specimen in both load-controlled and deformation-controlled tests.

Based on the comparison of load-controlled and deformation-controlled tests some important observations have been made. Firstly, the pre-peak stress-strain behaviour, peak deviator stress and mid-height pore water pressure are not affected significantly by the loading mode if the average time to reach the peak deviator stress is kept the same. Secondly, early failure occurs in faster tests in both the loading modes due to the non-uniformity of pore water pressure distribution. Thirdly, the failure modes are affected by the loading mode. While pre-failure strain softening is observed in slow deformation-controlled tests, instability and rapid failure of the specimen occur in load-controlled tests in the post-peak region. It shows that the conventional method of using deformation-controlled tests for determining the failure envelope may overestimate the effective friction angle and may lead to unsafe design where the subsoil is subjected to load-controlled loading mode.

Fig. 4.4i: Comparison of stress paths in LC and DC tests

Fig. 4.4j: Comparison of effective stress paths determined using top pore pressure in LC and DC tests

4.6 RATE EFFECT ON THE DRAINED BEHAVIOUR OF MARINE CLAY

To examine the effect of shearing rate on the drained behaviour of undisturbed marine clay, two CID tests, CID1 and CID2, were conducted at deformation rates of 0.002 mm/min and 0.01 mm/min respectively. The specimens in both the tests were isotropically consolidated under effective confining stress of 200 kPa. Drainage was allowed from both ends of the specimen and excess pore water pressure was measured at the mid-height. The results of the two tests are presented in Figures 4.5a to 4.5e.

The stress-strain curves of the two CID tests are compared in Fig. 4.5a. The stress strain curve of test CIU1 is also plotted for comparison. The peak deviator stress in faster test CID2 is about 15% lower than the peak deviator stress in slower test CID1. Also, a sharp peak occurs in faster test CID2 while deviator stress remains almost constant near the peak deviator stress in slower test CID1 and the stress-strain curve

is flat near the peak. Also, the peak deviator stress occurs at much larger axial strain in slower test CID1 than that in faster test CID2.

Volumetric strains in the two drained tests are plotted in Fig. 4.5b. It can be seen that there is significantly lower volume change in faster test CID2 than the volume change in test CID1, which indicates that the specimen was not fully drained in test CID2. In a fully drained test, the excess (over back pressure) pore water pressure, is supposed to be zero. However, the excess pore water pressure generated in both the CID tests, as shown in Fig. 4.5c, indicates that a large non-uniformity of pore water pressures occurred in faster test CID2. This is because at any given axial strain, the volume change is smaller in faster test CID2 (see Fig. 4.5b), which leads to the generation of higher excess pore pressure. Although the excess pore water pressure starts to dissipate after about 6.5% axial strain, a significant excess pore water pressure of about 26kPa remains undissipated when shear band occurred. This indicates that a drained test conducted at fast rate may not remain fully drained. In fact, even in test CID1, an excess pore water pressure of about 14 kPa developed and about 9 kPa remained undissipated when shear band occurred. It indicates that fully drained condition is hard to achieve. It also shows that if a drained test is conducted at a fast shearing rate, the specimen may remain in partially drained condition. Higher excess pore water pressure reduces the effective confining pressure and results in lower drained shear strength.

The stress paths determined using top pore water pressure (fully drained) in tests CID1 and CID2, are compared in Figures 4.5d. It shows that the faster test CID2 shows failure at a stress-state much below the failure envelope. However, the stress paths determined using mid-height pore water pressure (i.e. excess pore water pressure is accounted for), as shown in Fig. 4.5e, show significantly different trend. Fig. 4.5e shows that the failure in test CID2 occurred at a stress-state close to the failure envelope, although still below the failure envelope. It shows that on increase in axial strain rate, undissipated pore pressure increases and it reduces the mean effective stress (p') and leads to a reduction of drained shear strength as the failure conditions in terms of effective stresses are reached earlier. It indicates that the actual stress state within the specimen could be significantly different from the idealized

fully drained condition. It can only be determined using pore water pressure measurement at mid-height when double drainage is used.

Fig. 4.5a: Stress-strain curves in CID tests conducted at different shearing rates

Fig. 4.5b: Volume change in two drained tests

Fig. 4.5c: Excess pore pressure in drained tests conducted at different rates

Fig. 4.5d: Stress paths determined using top pore pressure in drained tests

Fig. 4.5e: Stress paths determined using mid pore pressure in drained tests

It is important to mention that, as shown in Table 3.2 in Chapter 3, the acceptable time to failure for drained test using isotropic consolidation data (BS1377, 1990; Head, 1986) was 1910 mins, which gives an axial strain rate of 0.013 mm/min for an assumed failure axial strain of 25%. It can be seen from the above discussion that a rate of 0.01 mm/min resulted in large non-uniformity of pore pressure and underestimation of the drained shear strength and effective friction angle. Hence, use of isotropic consolidation data for choosing the axial strain rate for CID tests may not be the best method for undisturbed Singapore marine clay. Also, as discussed in section 4.3.2, the selection of rate is dependent upon the chosen axial strain at failure. Considering the change in axial strain at failure with the change in axial strain rate (Fig. 4.5a), it will be more realistic to conduct the drained tests with pore pressure measurement (preferably at mid-height, to avoid increase in shearing time on end measurement with single drainage). The rate can be adjusted during the test if the excess pore pressure is found to increase in an unacceptable manner.

From the results of the drained tests CID1 and CID2, it can be seen that drained tests are affected significantly by rate effect. Increase in axial strain rate in drained tests

may result in significantly lower drained shear strength and lower failure envelope, which is opposite of the trend in undrained test, where undrained shear strength increased with axial strain rate. However, the two trends are related in that in both drained and undrained tests, higher pore water pressure results in lower shear strength. Also, both drained and undrained tests showed early failure in faster tests due to large non-uniformity of pore water pressure.

4.7 RATE EFFECT ON ANISOTROPIC CONSOLIDATION

One anisotropically consolidated undrained test (CK₀U or CAU), CAU1, was conducted on undisturbed Singapore marine clay to obtain an additional point for establishing the failure envelop. The specimen was K₀-consolidated at an axial strain rate of 0.005 mm/min. An interesting observation was that a shear band developed in specimen during K₀-consolidation, as indicated by x-sign in Fig. 4.6a. The K₀-consolidation was still continued till the desired stress level of 190 kPa. Upon shearing again, the specimen failed after short time, which was obviously due to the already developed shear band in the specimen.

Fig. 4.6a: Stress-paths of CAU tests determined using top pore pressure

To examine the probable causes of the early failure, another test, CAU2, was conducted in which the K_0 -consolidation of specimen was started at a slower rate of 0.0015 mm/min. The stress paths determined using the top pore water pressure (Fig. 4.6a) shows no significant difference between the two tests. However, a careful comparison of the stress paths determined using mid-height pore water pressure (Fig. 4.6b) in the two tests, CAU1 and CAU2, indicated that the cause of early failure in test CAU1 could be the higher non-uniformity of pore water pressure in test CAU1 at the start of K_0 -consolidation (see Fig. 4.6c). Fig. 4.6c shows that the excess mid-height pore pressure was about 10 kPa higher in test CAU1, which led to lower mean effective stress. It can be observed from Fig. 4.6c that the effective stress path in test CAU1 reached very close to the failure envelope at the start of K_0 -consolidation due to high excess mid-height pore water pressure. When the stress level is low, a small change in pore water pressure can lead to a large change in q/p' and it can be seen from Fig. 4.6d that q/p' was significantly high in test CAU1 at the start of K_0 consolidation. This could have led to the development of shear band within the specimen in test CAU1. On the other hand, the stress path in test CAU2 did not reach as close to the failure envelope (Fig. 4.6b) and K_0 -consolidation and subsequent shearing could be completed without any problem.

The results of the two CAU tests indicate that the rate of shearing could significantly affect the pore water pressure distribution within the specimen and may result in early failure. The use of mid-height pore water pressure is essential for an accurate determination of the pore water pressure and stress-strain responses of the soil.

Fig. 4.6b: Effective stress-paths of CAU tests determined using mid pore pressure

Fig. 4.6c: Comparison of excess pore pressures at the mid-height during K_0 consolidation

Fig. 4.6d: Variation of q/p' determined using mid pore pressure in CAU tests

4.8 DISCUSSIONS

4.8.1 Rate-Effect on Undrained Shear Strength and Stress-Strain Behaviour

Triaxial compression tests were conducted on isotropically consolidated specimens of undisturbed Singapore marine clay to study the effect of loading rate and loading mode on its undrained behaviour. Several interesting trends emerge from the test results. The undrained shear strength of Singapore marine clay increases with the increase in shearing rate in both load-controlled and deformation-controlled loading modes. Similar observation has been made for many other clays (Gibson and Henkel, 1954; Richardson and Whitman, 1963; Singh & Mitchell, 1969; Berre and Bjerrum, 1973; Akai et al., 1975; Vaid and Campanella, 1977; Lefebvre and LeBoeuf, 1987; Vaid, 1988; Lacasse, 1995; Sheahan et al., 1996; Zhu et al., 1999; Teachavorasinskun et al., 2002). However, only about 5% increase in undrained shear strength was observed per log cycle increase in axial strain rate, which is lesser than that observed for most other clays (8% to 15% increase in s_u per log cycle of axial strain rate).

The change in undrained shear strength with the change in shearing rate was found to be caused by the difference between the pore pressures in tests conducted with different shearing rates. In both, load-controlled and deformation-controlled tests, pore water pressure was found to decrease with the increase in shearing rate, which resulted in higher mean effective stress and higher peak deviator stress. Thus, depending upon the pore pressure generated during the test, the effective stress paths tend to reach the failure envelope (at maximum obliquity) at different deviator stresses. However, early failure, characterized by shear band formation and rapid drop in deviator stress, occurs in faster deformation-controlled tests and in load-controlled tests.

Lefebvre and LeBoeuf (1987) reported that the pore pressure increased with decrease in strain rate but the effective friction angle at maximum obliquity was unaffected by rate-effect for three types of *destructured* clays (Olga Clay, Grande Baleine Clay and B-6 Clay) by. However, Lefebvre and LeBoeuf (1987) did not observe significant difference in pore pressure for *structured* clay and attributed the pore pressure differences to the measurement errors. The results presented in this chapter are based on proper pore pressure measurement and show that the difference in pore pressure generation at different rates is a real phenomenon for a structured (undisturbed) soil also. Whether the same is true for reconstituted Singapore marine clay needs to be studied, especially considering the evidence of significant differences between the rate-effects on the behaviour of structured and destructured clays for other clays (Lefebvre and LeBoeuf, 1987). Also, any effects of non-homogeneity of undisturbed samples on the testing results can be eliminated in reconstituted clay.

The pore pressure measurements at the top and bottom of the specimen were found to over predict the pore water pressure as compared to the mid-height pore water pressure in tests conducted at the shearing rates faster than 0.005 mm/min. This could be due to the higher end friction at rough end platens in faster tests. Similar observation has earlier been made by Barden and McDermott (1965) and Fourie and Dong (1991) in undrained compression test conducted at different rates. Both these studies compared the tests conducted using similar rates with free ends and rough ends. It was shown that the end measurements of pore pressures were overpredicted on using the rough ends at faster tests compared to that measured with free-ends,

while mid-height pore pressures were comparable in tests conducted with free ends and rough ends even at faster rates. The large difference between the pore pressures measured at the top, bottom and the mid-height of the specimen in faster tests, makes the estimation of effective stresses difficult and inaccurate.

Another important observation made for deformation-controlled tests was that while significant pre-failure strain softening occurred in slower tests, there was lesser pre-failure strain-softening in faster tests and the specimen failed before the effective stress state could reach the failure envelope. Also, the specimen in slow test failed at larger axial strain and after large bulging while specimens in faster tests failed at smaller axial strain with the formation of shear band. In load-controlled tests, the specimen in all the tests failed with the formation of shear band before the effective stress state could reach the failure envelope. This appears to be a result of larger non-uniformity of pore water pressure distribution within the specimen at higher shearing rates. Due to the non-uniformity of pore pressures, the effective stress states at some points within the specimen reach the failure state earlier than the others and the shear band develops. In load-controlled tests, the non-uniformity of pore pressures occurs near the peak deviator stress at all shearing rates due to the onset of instability. Due to this reason shear band was found to occur in all the load-controlled tests slightly before the effective stress-state could reach the failure envelope.

4.8.2 Effect of Loading Mode

The stress-strain behaviour and pore pressure variations in deformation-controlled and load-controlled tests up to the peak deviator stress were similar. The peak deviator stress was a function of the time taken in reaching the peak and not of the loading mode. When the time taken in reaching the peak was similar, load-controlled and deformation-controlled tests showed similar stress-strain and pore water pressure response. However, the post peak behaviours were different. Instability, characterized by rapid increase in axial strain and axial strain rate, occurred in load-controlled tests near the peak deviator stress and resulted in brittle failure and an abrupt drop in deviator stress. Instability occurred because of the tendency of the loading system to increase the load even after the peak deviator stress. Instability occurred slightly before the effective stress reached the failure envelope. Slow

deformation-controlled tests show pre-failure strain softening and gradual drop in deviator stress and instability does not occur. This shows that in field conditions, where most of the loading conditions are load-controlled, instability may occur when the foundation is based on clay and will lead to a rapid failure of the structure. It may happen before the effective stress state reaches the failure state (which is generally determined from deformation-controlled tests). It is also notable that in load-controlled loading conditions, the imminent failure may not be known until the onset of instability because the axial strain and axial strain rate increase quite steadily before the onset of instability. Pore pressure may not give an indication as pore pressures in deformation-controlled and load-controlled tests are quite similar until the peak deviator stress. Therefore, where the actual field loading is load-controlled, load-controlled undrained tests should also be conducted while determining the strength and stress-strain parameters.

4.8.3 Rate-Effect on Drained Behaviour

The shearing rate was found to also affect the drained behaviour of marine clay. The drained shear strength decreased with the increase in axial strain rate from 0.002 mm/min to 0.01 mm/min, which was due to the non-uniform distribution of pore water pressure in faster drained test, which reduces the effective confining stress and leads to a lower peak deviator stress. Thus pre-mature failure may occur in drained tests conducted at faster rate, thus reducing the effective friction angle and shear strength. The currently used method (BS1377, 1990; Head, 1986) for calculating the time to failure in drained test for clay may underpredict the time to failure and result in lower drained shear strength and lower friction angle. Therefore, it is necessary to conduct the drained tests with proper pore pressure measured to ensure that the strain rate is suitable to allow acceptable uniformity of pore pressures within the specimen.

The differences between pore pressures at different shearing rates, which appear to be the main cause of the rate effect, in addition to the mineralogy etc., could be related to the creep in the structure of the soil as well as to the low permeability of marine clay. While creep may lead to higher overall pore pressure in slower tests, low permeability leads to non-homogeneity of pore pressures within the specimen in

the tests conducted at faster rates. Some differences might also have been caused by the inherent non-homogeneity of undisturbed soil samples. A study, similar to the one presented in this chapter, on reconstituted marine clay, which is destructured and more homogeneous, will throw some more light on the possible causes of rate-effect. For this reason, a similar study was carried out on reconstituted Singapore marine clay and its results will be presented in Chapter 6.

4.9 CONCLUSIONS

Results from the undrained tests on undisturbed Singapore marine clay were presented in this chapter. Behaviour of Singapore marine clay was found to be affected by the shearing rate and shearing mode. The main conclusions can be summarized as follows.

1. The undrained shear strength increases with the increase in rate of deformation (deformation-controlled tests) and rate of load increment (load-controlled tests). A linear relationship exists between the undrained strength and logarithmic change in shearing rate. Undrained shear strength increases by an average of 6% due to one log cycle increase in strain rate.
2. The pre-peak stress-strain behaviour and peak deviator stress are not affected significantly by the loading mode when comparable shearing rates are used by keeping the time taken to reach the peak deviator stress to be the same in both load-controlled and deformation-controlled tests.
3. Instability, characterized by rapid increase in axial strain and axial strain rate, was found to occur in load-controlled tests before the effective stress states reached the failure envelope. Deformation-controlled test, which is commonly used for determining the strength and stress-strain behaviour in practice, does not capture the instability behaviour.
4. The pore water pressure generated in undrained tests increases with the decrease in shearing rate in both load-controlled and deformation-controlled

- tests. This could be due to the additional pore pressure generated as a result of undrained creep in slower tests.
5. The effective friction angle at maximum obliquity is lower for the deformation-controlled tests conducted at rates faster than those determined using BS1377 (1990). Also, the effective friction angle at maximum obliquity is lower in load-controlled test. The specimens in faster deformation-controlled tests and in load-controlled tests fail by shear band formation slightly after the peak deviator stress. This appears to be related to the non-uniformity of pore pressure in these tests. This is also indicated by the fact that the specimen in slow deformation-controlled test, in which pore pressure is uniform throughout the specimen, fails by bulging after large strain-softening.
 6. The pore water pressure distribution within the specimen becomes non uniform when the axial strain rate is higher than the maximum permissible axial strain rate in both undrained and drained compression tests. This is due to the low permeability of clay, which makes the equalization of pore pressure more difficult with the increase in axial strain rate. Large difference between the pore pressures measured at the top, bottom and mid-height of the specimen makes the precise estimation of effective stresses difficult. The test may become less meaningful at faster rates of shearing, especially when effective stress parameters are required. Also, the specimen in tests conducted at faster rates in both load-controlled and deformation-controlled tests failed before the effective stress could reach the failure envelope. This appears to be due to the larger non-uniformity of pore pressures in faster tests.
 7. Measurement of mid-height pore water pressure is essential to estimate the uniformity of pore water pressure within the specimen, especially in faster deformation-controlled tests and load-controlled tests, when tests are conducted on a low permeability soil.
 8. The method suggested by BS1377 (1990) for choosing the time to failure for clay based on the isotropic consolidation data is adequate for undrained

compression tests in terms of ensuring the equalization of pore water pressure within the specimen. Conducting the tests at shearing rates faster than the calculated leads to large non-uniformity of pore pressures and early formation of shear band within the specimen. However, use of BS1377 (1990) only predicts the time to failure and the selection of rate depends upon the chosen axial strain at failure. Considering the fact that the axial strain at failure changes with the change in axial strain rate, it will be more realistic and useful to conduct tests with proper pore pressure measurement and adjust the shearing rate according to the observed non-uniformity of pore pressure distribution.

9. The behaviour of undisturbed marine clay in drained compression tests is affected by the axial strain rate. A test conducted at fast rate under-predicts the drained shear strength and does not represent the truly drained behaviour. This is because the fast shearing rate results in incomplete drainage and large excess pore pressure, which in turn leads to lower mean effective stress and lower deviator stress. Therefore, drained compression tests must be conducted with proper pore pressure measurement to make sure that the excess pore pressure within the specimen are reasonably low.

It may be pointed out that the non-homogeneity of undisturbed samples could have influenced the results obtained from different tests. Therefore, it is necessary to conduct a study of rate-effect on the reconstituted form of the undisturbed Singapore marine clay. Reconstituted samples are more homogeneous and identical; hence the sample variability can be significantly reduced. The results from study of rate effect on reconstituted marine clay will be presented in Chapter 6.

CHAPTER 5

UNDRAINED CREEP AND INSTABILITY IN UNDISTURBED SINGAPORE MARINE CLAY

5.1 INTRODUCTION

One of the objectives of this research program was to study the instability behaviour of cohesive soils, which is one among the three major time-effects on clay, namely; rate effect, creep and stress relaxation. As defined in section 2.4.1 in Chapter 2, instability refers to a phenomenon in which large plastic strains are generated rapidly due to the inability of the soil element to sustain a given load or stress, including small perturbations (Lade 1994). As discussed in Chapter 2, during triaxial compression tests on sand, when lateral confinement is reduced while maintaining the axial load constant, the specimen may become unstable well before the effective stress state reaches the failure envelope (Lipinski, 2001; Chu and Leong 2002). An instability line and a 'zone of potential instability' can be defined, as shown earlier in Fig. 2.27 in Chapter 2. Whether such a zone exists and pre-failure instability occurs in clay is not yet known.

As shown in section 4.4.1 in Chapter 4, instability occurred in load-controlled CIU tests slightly before the effective stress state reached the effective failure envelope determined from deformation-controlled CIU and CID tests. In a similar way, instability may occur in undrained creep tests since creep also takes place in a load-controlled condition. As discussed in Chapter 2, when clay is subjected to undrained creep at a constant load below failure, excess pore water pressure is generated (Walker, 1969). This leads to the reduction of mean effective stress and movement of

effective stress state towards the failure line, as schematically shown earlier in Fig. 2.26 in Chapter 2. Axial strain rate resulting from creep appears to initially reduce in most of the cases, as shown in Fig. 5.1 (Sheahan, 1994). At lower deviator stress level, axial strain rate may continue to decrease or become constant at a small value, as shown by the lowest curve in Fig. 5.1. If the deviator stress level is sufficiently high, the axial strain rate increases again (transition at point A or B) and creep rupture occurs (Arulanandan et al, 1971; Finn and Shead, 1973). However, creep rupture is preceded by instability, characterized by increase in axial strain rate after initial reduction to a transient minimum strain rate, as shown by points 'A' and 'B' in Fig. 5.1. However, the term, 'instability' for clay, is not used in the same way as for sand. Although there have been numerous studies on undrained creep behaviour of clay, those have mainly focused on time-dependent variation of axial strain and axial strain rate (Casagrande and Wilson, 1951; Mitchell et al, 1968; Singh and Mitchell, 1969; Walker, 1969; Finn and Shead, 1973; Campanella and Vaid, 1974; Akai et al, 1975; Vaid and Campanella, 1977; Wu et al, 1978; Kavazanjian and Mitchell, 1980; Mesri et al, 1981; Graham et al, 1983; Kuhn and Mitchell, 1993; Sheahan, 1994; Liingard et al, 2004; Augustesen et al, 2004). Oka et al (1994) mathematically showed that there exists a critical stress ratio (q/p'), η_c , in effective stress space, which separates stable region from unstable region, as shown earlier in Fig. 2.26 in Chapter 2. This concept is similar to the concept of pre-failure instability in sand. However, experimental results, that support this concept, were not shown by Oka et al (1994). The effective stress changing process and the instability behaviour of clay under creep have seldom been studied. It is still not known whether the instability will occur at a pre-failure state. One of the reasons for omission of discussion on effective stresses in undrained creep tests in the previous studies is that, the pore water pressure within the specimen could not be measured with accuracy near rupture (Finn and Shead, 1973). Therefore, there is a need to study the undrained creep behaviour of clay with proper pore pressure measurement so that the effective stress change process during undrained creep and near the rupture may be understood.

This chapter presents the results of undrained creep tests on undisturbed Singapore marine clay. Both single-stage and multi-stage undrained creep tests were conducted.

In addition to the time-dependent variation of axial strain and axial strain rate, special attention was paid to the variation of pore water pressure, which could be measured more reliably with the use of a mid-height pore water pressure transducer in addition to the pore water measurements at the top and bottom of the specimen. Effective stress-paths and failure states during undrained creep tests were compared with the effective stress paths and failure envelope determined from conventional tests. Existence of pre-failure instability was investigated.

Fig. 5.1: Schematic representation of the variation of log axial strain rate with time (after, Sheahan, 1994)

5.2 UNDRAINED CREEP BEHAVIOUR OF UNDISTURBED SINGAPORE MARINE CLAY

During an undrained creep test, the specimen was first consolidated isotropically under an effective confining pressure of 200 kPa, and then it was sheared undrained till a selected deviator stress level was reached. The load corresponding to this deviator stress level was held constant and specimen was allowed to creep under undrained conditions. A consolidation pressure of 200 kPa was chosen to compare the behaviour in undrained creep tests with CIU tests described in Chapter 4, since most of the results presented in that chapter were on specimens consolidated under 200 kPa. Since in most of the undrained creep tests, the desired deviator stress levels

were achieved rapidly (within 1 to 4 minutes), it was expected that the specimen would have a strength similar or slightly higher than that determined from fastest CIU test. Therefore, the chosen deviator stress level for undrained creep was a certain percentage of the maximum peak deviator stress (132 kPa) obtained from fastest CIU test (CIU3 – 0.5 mm/min), as discussed in Chapter 4.

If the specimen did not show any instability after a certain duration of creep (usually between 2000 min. to 5000 min.) and axial strain rate was found to become small (usually between 0.00002 mm/min to 0.0001 mm/min), it was sheared further under undrained condition to a higher deviator stress level and left again for undrained creep. This procedure was repeated till the specimen became unstable and failed. The instability was identified as the point at which the axial strain rate started to increase rapidly (Chu and Leong, 2002; Chu et al, 2003). The effective stress states (mainly q/p' ratio) at the onset of instability and failure were compared with the failure envelope obtained from CIU tests (as determined earlier in Fig. 4.1a in Chapter 4).

A total of nine (9) undrained creep tests, namely CR1 to CR9, were conducted, which included both single stage and multi-stage undrained creep tests. Tests CR1 and CR2 were single stage creep tests, in which specimen were sheared at 0.005 mm/min until desired deviator stress level for creep was reached. In rest of the tests, specimen was rapidly sheared (within 2-3 minutes) to a desired creep deviator stress level. Undrained creep was allowed to take place at constant load rather than constant deviator stress due to the limitations of data-logging software. It may be pointed out that in most of the earlier studies of undrained creep the deviator stress during undrained creep was held constant by small increments of load to account for the increased cross-sectional area (Augustesen et al, 2004). However, Vaid and Campanella (1977) conducted both constant-stress and constant-load creep tests on Haney clay and concluded that the behaviour was similar in the constant stress and constant load creep. Therefore, the behaviour in constant-load creep observed in this study may not differ much from the behaviour in constant stress creep.

Pore water pressures were measured at the top, bottom and middle of the specimen throughout the loading and creep process in all the tests. Since the response of mid-

height probe is quick (Hight, 1982; Fourie and Dong, 1991; Sheahan et al, 1996), it is able to capture the change in pore water pressure in the near failure stages, when axial strain rate is high. A summary of the nine undrained creep tests, with effective stress ratios (q/p') at the start of creep in different stages of creep is presented in Table 5.1. q/p' values at the onset of instability, deviator stress levels at the beginning of the stage in which instability occurred and the ratios $[(q/p')/(q/p')_f]$ at the beginning of the stage in which instability occurred are also listed. Here $(q/p')_f$ is equal to 0.986 and is the maximum value of q/p' in CIU tests, which was obtained in test CIU1 (0.005 mm/min).

Table 5.1: Summary of undrained creep tests

Test	p_0'	(q/p') at start of creep stages					q/p' at instability	(q/p')/(q/p') _f at the beginning of instability stage [#]	Deviator stress level, (q/q _m) [^] in instability stage
		stage 1	stage 2	stage 3	stage 4	stage 5			
CR1	200	0.781	-	-	-	-	1.033	0.792	0.864
CR2	200	0.553	-	-	-	-	na	na	na
CR3	200	0.502	-	-	-	-	na	na	na
CR4	200	0.579	0.737	0.888	-	-	0.900	0.900	0.833
CR5	200	0.382	0.486	0.569	0.691	0.841	0.924	0.853	0.788
CR6	200	0.558	0.747	0.786	-	-	0.801	0.797	0.800
CR7	200	0.620	0.767	-	-	-	0.767	0.778	0.775
CR8	200	0.517	0.667	0.744	0.781	0.976	0.981	0.990	0.914
CR9	200	0.607	0.828	-	-	-	0.918	0.840	0.887

[#] $(q/p')_f = 0.986$ is the maximum value of q/p' (at max. σ_1'/σ_3') in CIU tests (obtained from CIU1)

[^] $q_m = 132\text{kPa}$ is the maximum peak deviator stress (from CIU3) amongst all CIU tests

Following sections will present the results of some of the undrained creep test separately for the sake of clarity of figures. A discussion on the typical trends observed will be made subsequently with reference to the earlier studies on undrained creep.

5.2.1 Test CR2

Results of test CR2 are presented in Figs. 5.2a to e. Results of test CIU1 (specimen sheared at 0.005 mm/min) are also plotted in Figs. 5.2a and 5.2b for comparison. To follow an effective stress path similar to that in test CIU1, the specimen in test CR2 also was sheared at 0.005 mm/min to point 1 (Fig. 5.2a) with q/p' of 0.553 and was left for undrained creep. The axial strain increased (Fig. 5.2b) and the effective stress path moved towards the failure envelope (Fig. 5.2a) due to the increase in pore water pressure (Fig. 5.2c). However, the pore water pressure (Fig. 5.2c) became constant after about 4000 minutes of creep and hence there was little further change in the mean effective stress. The axial strain rate (Fig. 5.2d) also reduced to a very small value of about 0.00002 %/min to 0.00003 %/min and appeared to be reducing further. Due to this, the axial strain (Fig. 5.2e) increased very slowly. After about 5500 minutes of creep, the effective stress state reached a point 1 having q/p' of 0.553.

The variation of pore water pressure and axial strain rate in stage 1 indicated that the instability may not occur at the current deviator stress level ($q/p' = 0.553$). Therefore, the process of creep was terminated and the specimen was sheared again under an undrained condition at 0.005 mm/min. The specimen showed a stiffer response on re-shearing as evident from $q-p'$ and $q-\varepsilon_1$ curves in Figs. 5.2a and 5.2b, although, the peak deviator stress and failure envelope on re-shearing were the same as those in test CIU1. This test showed that the instability may not occur at very low deviator stress level and the effective stress and strain conditions become stable with little further change at the end of such a creep stage.

Fig. 5.2a: Stress path in test CR2 compared with test CIU1

Fig. 5.2b: Stress-strain curves in test CR2

Fig. 5.2c: Variation of pore pressure with time in tests CR2

Fig. 5.2d: Variation of axial strain rate with time in test CR2

Fig. 5.2e: Variation of axial strain with time in test CR2

5.2.2 Test CR3

Test CR3 was conducted under the same test conditions as test CR2 except that the shearing methods were different. While in test CR2, the shearing was at 0.005 mm/min to reach the required deviator stress level for creep, the shearing was rapid in test CR3. This was done to examine the effect of method of shearing (to achieve the desired deviator stress level) on the undrained creep behaviour. The results of test CR3 are shown in Figs. 5.3a to e. Results of test CIU1 are also plotted in Figs. 5.3a and 5.3b for comparison. The specimen was sheared rapidly to a point ‘a’ (Fig. 5.3a) with q/p' of 0.502 within 2-3 minutes and left for undrained creep. The axial strain (Fig. 5.3b) increased due to the creep-induced deformation. The effective stress path moved towards the failure envelope (Fig. 5.3a) due to the large increase in pore water pressure (Fig. 5.3c). However, the pore water pressure (Fig. 5.3c) and axial strain rate (Fig. 5.3d) became constant after about 3000 minutes of creep and remained almost constant thereafter. After about 5800 minutes of creep, the effective

stress state became stable at point a' (Fig. 5.3a) and axial strain (Fig. 5.3e) increased very slowly. It indicated that the instability may not occur. The test was terminated at this point.

Fig. 5.3a: Effective stress path in test CR3

Fig. 5.3b: Stress-strain curve in test CR3

Fig. 5.3c: Variation of pore pressure in test CR3

Fig. 5.3d: Variation of axial strain rate with time in test CR3

Fig. 5.3e: Variation of axial strain with time in test CR3

The results of test CR3 are compared with the results of test CR2 in Figs. 5.4a and 5.4b. Although initial deviator stresses (deviator stress levels) were the same in tests CR2 and CR3, the specimen in test CR2 reached a higher q/p' of 0.553 at point 1 while CR3 reached a lower q/p' of 0.502 at point 'a' at the beginning of creep. This was due to lesser pore water pressure in test CR3 than that in test CR2 (point 'a' compared to point 1 in Fig. 5.4b). Nevertheless, both tests did not show any instability and the effective stress state at the end of creep was the same (point a' and 1') in both. The effective stress states at the end of creep were also the same in the two tests. It indicates that the method of loading used to reach a desired creep deviator stress level does not affect the creep behaviour significantly, when the deviator stress level is low.

Fig. 5.4a: Comparison of stress paths in tests CR2 and CR3

Fig. 5.4b: Comparison of pore pressure variations in tests CR2 and CR3

5.2.3 Test CR1

Results of test CR1 are presented in Figs. 5.5a to f. Results of test CIU1 are also plotted for comparison in Figs. 5.5a and 5.5e. As shown in Fig. 5.5a, the specimen was sheared at 0.005mm/min from an isotropic consolidated state ($p_0' = 200$ kPa) to a point 1 with q/p' of 0.781 and left for undrained creep at constant load. Pore water pressure increased gradually during the creep, as shown by curve 'A' in Fig. 5.5b. As a result, the mean effective stress reduced and the effective stress path moved towards the failure envelope (Fig. 5.5a). The specimen became unstable at point 'I' with q/p' of 1.033 which is close to the failure envelope obtained from CIU and CID tests. The instability was identified from axial strain vs. time and axial strain rate vs. time curves as indicated by point I in Figs. 5.5c and 5.5d. Fig. 5.5c shows that the axial strain increased steadily during creep and a rapid increase started at point I. Similarly, the axial strain rate (Fig. 5.5d) started to increase rapidly at point I. Thus, the point I, at which a rapid increase in axial strain and axial strain rate started, was identified as the point of onset of instability. Failure occurred with the formation of shear band at point 'SB' with q/p' of 1.039 (Fig. 5.5a). Both onset of instability and failure occurred close to the failure envelope as seen from Fig. 5.5a. Shear band was identified by the sudden drop in q and q/p' . This method of detection of shear band was discussed earlier in Chapter 4.

It can be observed from the plot of pore water pressure vs. time (curve A) in Fig. 5.5b that the pore water pressure started increasing rapidly at point I. However, it was observed that while top and mid-height pore water pressure kept increasing, the bottom pore water pressure dropped abruptly after instability. Thus the difference between bottom and mid-height pore pressures (curve B in Fig. 5.5b) increased rapidly. This shows that a large non-uniformity of pore water pressure occurred within the specimen after the onset of instability and stress-strain parameters measured after the instability may not be reliable.

The stress-strain curve (Fig. 5.5e) shows that about 6.5% axial strain developed during creep from point 1 to point I. Magnitude of deviator stress reduced with the progress of creep due to the increase in cross sectional area. Rapid drop in deviator stress was recorded after the shear band formation. The drop was due to the

formation of shear band as well as due to the inability of the system to maintain the axial load constant after the specimen had become unstable.

Fig. 5.5a: Stress path in test CR1 compared with test CIU1

Fig. 5.5b: Variation of pore pressure and pore pressure distribution in test CR1

Fig. 5.5c: Variation of axial strain with time in test CR1

Fig. 5.5d: Variation of axial strain rate with time on linear scale in test CR1

Fig. 5.5e: Stress strain curve in test CR1

It is necessary to compare the conventional analysis method for creep rupture tests with the method adopted in this thesis. All the earlier studies, as listed in the introduction to this chapter, show the axial strain rate vs. time plot on a double-log scale (Fig. 5.1). The axial strain rate vs. time relationship for test CR1 is plotted on a log-log scale in Figs. 5.5f for comparison with the linear plot of Fig. 5.5d. It is observed that the data presented here follow the typical trend on a log-log scale (Fig. 5.5f). On the log-log scale the point of transient minimum (at 140 minutes) appears to be the point of instability. However, the point of onset of instability (from Fig. 5.5d) is significantly different (at 630 minutes) from the point of transient minimum strain rate (Singh and Mitchell, 1969; Finn and Shead, 1973, Campanella and Vaid, 1977). Therefore, the conventional method of plotting axial strain vs. time relationship on log-log plot may not be suitable for determining the point of onset of instability. It also shows that the point of transient minimum strain rate may not be the point of onset of instability.

Fig. 5.5f: Variation of axial strain rate with time on a log-log scale in test CR1

5.2.4 Test CR9

The results of two-stage creep test CR9 are presented in Figs. 5.6a to 5.6e. The results of test CIU1 are also plotted in Fig. 5.6a and 5.6d for comparison. In the first stage (Fig. 5.6a), the specimen was sheared rapidly to a point 1 with a q/p' of 0.607. Undrained creep developed for 5600 minutes and the effective stress state moved from point 1 to 1' without any instability. The deviator stress level was then increased to point 2 with q/p' of 0.828. After about 2500 minutes of creep, instability occurred at point I (Fig. 5.6a). q/p' at instability was 0.856, which was close to the failure envelope. Shear band occurred at point SB, which was on the failure envelope.

Variations of axial strain and axial strain rate are shown in Figs. 5.6b and Fig. 5.6c. Axial strain increased steadily until instability occurred and increased rapidly after the instability. In stage 1, in which instability did not occur, axial strain rate continuously reduced to reach very small value. In stage 2, strain rate initially decreased at the start of creep. At the onset of instability, the strain rate increased

rapidly and shear band occurred. Transient minimum strain rate in stage 2 occurred before the instability. However, the minimum strain rate remained nearly constant over a large duration in stage 2 and it may not be defined by a single point.

The stress-strain curve of test CR9 (Fig. 5.6d) shows that large axial strain occurred after the instability. Sudden drop in deviator stress was observed at failure due to the formation of shear band and inability of the system to maintain the load constant. In stage 1, in which the instability did not occur, the pore water pressure (Fig. 5.6e) became almost constant after the initial increase for about 2000 minutes. However, in stage 2, the pore water pressure increased continuously. After the onset of instability, a rapid increase in pore water pressure occurred. Pore water pressures at the top and mid-height of the specimen dropped abruptly at failure, indicating large non-uniformity within the specimen.

Comparison of observed behaviours in tests CR1 and CR9 shows that the pore water pressure increases continuously in a stage in which instability is about to occur, while in a stage in which instability does not occur, the pore water becomes constant after some duration of creep.

Fig. 5.6a: Stress path in test CR9

Fig. 5.6b: Variation of axial strain with time in test CR9

Fig. 5.6c: Variation of axial strain rate with time in test CR9

Fig. 5.6d: Stress-strain curve in test CR9

Fig. 5.6e: Variation of mid pore pressure with time in test CR9

5.2.5 Test CR5

Test CR5 was a multi-stage creep test. Results of this test are presented in Figs. 5.7a to e. Results from test CIU1 are also plotted in Figs. 5.7a and 5.7b for comparison. The behaviour of the specimen in this test was similar to that in test CR9. In the first four stages, in which instability did not occur, the effective stress state (Fig. 5.7a) moved towards the failure envelope and became stable at points 1', 2', 3' and 4' respectively. In stage 5, instability occurred at point I, which was close to the failure envelope, and failure occurred at the failure envelope. Stress-strain curve (Fig. 5.7b) shows an increase in axial strain in each creep stage. It also shows that higher axial strain occurred during creep at higher deviator stress levels and very large axial strain occurred after the instability. Variation of pore water pressure (Fig. 5.7c) was slightly different from that in tests CR1 and CR9. Although, overall pore water pressure increased at the end of each stage, pore water pressure increased to a high value then decreased again in each stage (see Fig. 5.7c, between points 1-1', 3-3', 4-4'). The pore water pressure increased rapidly after the instability and was accompanied by large non-uniformity of pore pressure (see pore pressure difference plot in Fig. 5.7c). The axial strain rate (Fig. 5.7d) appears to be continuously decreasing after the initial increase in all the stages. The minimum axial strain rate increased with the increase in deviator stress level despite longer duration of creep in later stages (as compared with stages 2, 3, 4 and 5). This observation is consistent with the earlier studies on undrained creep of clay (Vaid and Campanella, 1977; Finn and Shead, 1973; Augustesen et al, 2004). Axial strain (Fig. 5.7e) continuously increased during creep and a rapid increase was observed after the onset of instability.

The results suggest that the onset of instability occurs close to the failure envelope and is accompanied by a rise in pore pressure and large non-uniformity of pore pressure within the specimen. The results are similar to those obtained from tests CR9 and CR1.

Fig. 5.7a: Stress path and instability line in test CR5

Fig. 5.7b: Stress-strain curve in test CR5

Fig. 5.7c: Variation of mid pore pressure with time in test CR5

Fig. 5.7d: Variation of axial strain rate with time in test CR5

Fig. 5.7e: Variation of axial strain with time in test CR5

5.2.6 Test CR8

Results of the multi-stage creep test CR8 are shown in Figs. 5.8a to 5.8e together with the results from test CIU1 in Figs. 5.8a and 5.8b. The observed behaviour was similar to that observed in tests CR9 and CR5. Instability occurred when the effective stress state reached the failure envelope (Fig. 5.8a). The deviator stress level was increased at this stage to see the effect of increase in load after the instability. The pore water pressure (Fig. 5.8c), axial strain (Fig. 5.8d) and axial strain rate (Fig. 5.8e) increased rapidly due to the instability and increase in deviator stress level. The failure occurred after the failure envelope. The variation of axial strain rate (Fig. 5.8e) shows that in stage 4, the axial strain rate increased very gradually after the transient minimum strain rate. In fact, it is difficult to establish the point at transient minimum strain rate since the axial strain rate remains almost constant at minimum value over a significant duration in stage 4.

Fig. 5.8a: Stress path in test CR8

Fig. 5.8b: Stress-strain curve in test CR8

Fig. 5.8c: Variation of mid pore pressure with time in test CR8

Fig. 5.8d: Variation of axial strain with time in test CR8

Fig. 5.8e: Variation of axial strain rate with time in test CR8

5.2.7 Test CR6

Test CR6 was also a multi-stage undrained creep test. The results of this test were similar to the results of the tests discussed above with one significant difference. The instability was observed to occur at a point before reaching the failure envelope, as shown in Fig. 5.9a. However, the failure occurred at an effective stress state, which was very close to the failure envelope.

Two more multistage creep tests, CR4 and CR7, were also conducted using various deviator stress levels during creep stages. The results were similar to those presented above.

The results of the undrained creep tests on undisturbed Singapore marine clay were presented in this chapter. A number of important observations were made from the results. When the clay specimen is left for undrained creep at a constant load, the pore water pressure increases (see Figs. 5.2c, 5.3c, 5.4b, 5.5b, 5.6e, 5.8c) and the effective stress state moves closer to the failure envelope, as shown in Figs. 5.2a, 5.3a, 5.4a, 5.5a, 5.6a, 5.7a, 5.8a and 5.9a. This behaviour is well understood theoretically (Finn & Shead, 1973; Augustesen et al, 2004). However, the effective stress paths in undrained creep tests have seldom been shown in literature, except in a few studies (Walker, 1969). The results presented in this chapter attempted to analyze the results of undrained creep tests in terms of effective stress variations.

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5.3.1 Variation of Axial Strain and Axial Strain Rate

In this study, it was observed that the axial strain increases slowly with time during undrained creep of undisturbed marine clay when deviator stress levels is low (see, for example, creep stages 1, 2 & 3 in Fig. 5.8d). In the undrained creep stage in which the instability occurs, a sharp turn in ϵ_1 -t curve is observed at the onset of instability. The point of onset of instability can be identified in a manner illustrated in Fig. 5.10a. Tangents can be drawn to the two sections of the ϵ_1 -t curve, namely, the section before the onset of instability and the section after the onset of instability. The point, on which a line drawn to bisect the angle formed by the intersection of the two tangents intersects the ϵ_1 -t curve, can be identified as the point of instability, I.

Fig. 5.10a: Identification of point of onset of instability

Variation of axial strain rate versus time has been conventionally plotted on a log-log scale (as shown earlier in Fig. 5.1) in most of the previous studies of undrained creep (e.g. Mitchell et al, 1968; Singh and Mitchell, 1969; Walker, 1969; Finn and Shead, 1973; Campanella and Vaid, 1974; Akai et al, 1975; Vaid and Campanella, 1977; Wu et al, 1978 etc.). At a creep deviator stress level ($q/q_{\max(\text{CIU})}$) at which rupture occurs the axial strain rate is found to reach a transient minimum strain rate (Finn

and Shead, 1973), after which axial strain rate increases and rupture occurs, as shown earlier in section 2.4 of Chapter 2. The points of transient minimum strain rate lie on a straight line on a log-log scale as shown in Fig. 5.10b (from Finn and Shead, 1973).

Fig. 5.10b: The line of transient minimum strain rates in undrained creep tests (Finn & Shead, 1973)

To compare the results presented in this chapter with the above observation, the axial strain rate vs. time curves have been plotted on a log-log scale in Figs. 5.10c and 5.10d. When deviator stress level $(q/q_{\max(\text{CIU})})^\dagger$ and effective stress conditions $[(q/p')/(q/p')_{\text{failure}(\text{CIU})}]^\ddagger$ are suitable (see section 5.3.3 for the suitable conditions for instability to occur), the axial strain rate initially decreases at the start of creep until a minimum value (transient minimum strain rate) is reached (Fig. 5.10c) and then starts to increase gradually (also see, for example, 5.5f). Instability occurs after some duration of gradual increase in axial strain rate and the axial strain rate starts to increase rapidly after that. Two straight lines have been drawn through the possible

[†] $q_{\max(\text{CIU})}$ is the peak deviator stress obtained in fastest CIU test CIU3 (1 mm/min)

[‡] $(q/p')_{\text{failure}(\text{CIU})}$ is the effective stress ratio at failure in CIU tests

points of transient minimum strain rate in Fig. 5.10c. This figure suggests that there may not be a sharp increase in axial strain rate and it is difficult to define the point of transient minimum strain rate accurately for undisturbed Singapore marine clay. Thus the points at the transient minimum strain rate may not lie on a straight line for this clay. Also, the value of minimum strain rate does not appear to have a consistent relationship with the deviator stress level (Fig. 5.10c). For example, the deviator stress level (0.887) is higher in test CR9 than the deviator stress level (0.788) in test CR4, nevertheless, the minimum rate is higher in test CR4. It is different from the earlier observations (Finn and Shead, 1973, Singh and Mitchell, 1968; Campanella and Vaid, 1974) that the transient minimum strain rate increases with the increase in deviator stress level.

Fig. 5.10c: Variation of axial strain rate in the failure stage of creep in undrained creep tests

As shown by the encircled parts of the curves, there was a time-interval near the transient minimum strain rate in which the axial strain rate was almost constant or increased very slowly. The turn in strain rate vs. time curve was not as sharp as observed in earlier studies such as that shown in Fig. 5.10b (Finn and Shead, 1973).

It shows that significant secondary creep, during which the axial strain rate is almost constant, may occur during undrained creep tests on undisturbed Singapore marine clay. In a field loading condition, secondary creep may be taken as an indication of the imminent instability.

Fig. 5.10d: Variation of axial strain rate in the failure stage of creep in undrained creep tests showing intervals of almost constant strain rate

When deviator stress level $(q/q_{\max(\text{CIU})})$ and effective stress conditions $[(q/p')/(q/p')_{\text{failure}(\text{CIU})}]$ are unsuitable, i.e. in creep stages in which instability does not occur, axial strain rate (Fig. 5.10e) continuously reduces throughout the creep process (also see, for example, stages 1, 2 and 3 in Fig. 5.8e). Axial strain rate does not appear to reach a constant value, thus indicating that the secondary creep (Campanella and Vaid, 1974; Sheahan, 1995; Augustesen et al, 2004) does not occur in the creep stages in which instability is not imminent. This figure shows an almost linear decrease in axial strain rate on a log-log scale. Similar observation has earlier been made in some earlier studies (Mitchell et al, 1968; Finn and Shead, 1973; Campanella and Vaid, 1974; Akai et al, 1975; Wu et al, 1978; Kavazanjian and Mitchell, 1980; Mesri et al, 1981; Sheahan, 1994; Augustesen et al, 2004).

Fig. 5.10e: Variation of axial strain rate in non-failure stages of undrained creep

5.3.2 Variation of Pore Water Pressure

The results presented in this chapter showed that if the deviator stress level was low or if the effective stress state was far away from the failure envelope, the pore water pressure increased only at the start of creep and became almost constant after some time (see Fig. 5.8c, for example). Thus, the effective stress-states ceased further movement and instability did not occur. However, when the deviator stress level was high enough and effective stress state was not very far from the failure envelope, the pore water pressure kept rising and at the onset of instability, pore pressure increased rapidly (see Figs. 5.7c, 5.8c etc.). Similar to the observation made in Chapter 4 for the CIU tests conducted at fast rates, large non-uniformity of pore water pressure occurred within the specimens after the onset of instability, which was indicated by large difference between the pore water pressures at top, bottom and mid-height of the specimen (see Fig. 5.7c, 5.8c). Due to the non-uniformity of pore pressure it became difficult to accurately determine the effective stress state at failure.

5.3.3 Onset of Instability

For sand, the onset of instability is studied with respect to the effective stress failure envelope (Lade, 1994; Chu and Leong, 2002; Chu et al, 2003). As discussed in the introduction to this chapter and shown in Fig. 2.27 in Chapter 2, a zone of potential instability may be defined for sand in effective stress space. If the effective stress state of the soil element is below this zone, instability will not occur. However, if the effective stress state is within the zone of potential instability, instability may occur. To examine whether the similar concept can be applied to undisturbed marine clay, the effective stress states at the beginning of those creep stages, in which instability did not occur, effective stress states at the beginning of stages in which instability occurred and the points of onset of instability in undrained creep and load-controlled CIU tests are plotted in Fig. 5.11a. The effective failure envelope, as determined from CIU tests, is also plotted for comparison. Following observations can be made from Fig. 5.11a

1. an 'instability line' can be defined which passes through the lowest effective stress state at which instability occurred,
2. a 'zone of potential instability' can be defined which is bound by the failure envelope and the instability line,
3. when the effective stress state at the beginning of any creep stage was below the zone of potential instability, specimen remained stable and instability did not occur and
4. when the effective stress state at the beginning of a creep stage was within the zone of potential instability, instability occurred.

It can also be observed from Fig. 5.11a that the effective stress states at the onset of instability in load-controlled CIU tests were within the zone of potential instability.

Fig. 5.11b shows the effective stress paths of two CIU and one CAU test conducted on undisturbed marine clay with respect to the instability line and the zone of potential instability. It should be noted that the instability line coincides with the envelope of peak deviator stresses for the CIU and CAU tests. Therefore, the zone of potential instability is the zone bound by the envelope of peak deviator stresses and

the failure envelope obtained from conventional CIU and CAU tests. Whether or not instability will occur is governed by the effective stress state at the start of undrained creep state. Since the onset of instability always results in the failure of the specimen, it is as important to know the location of the instability line as the location of failure envelope in the effective stress state. Hence, it will be more conservative to use the instability line as the criteria for failure rather than the effective failure envelope when problem of undrained creep is critical since the failure may be imminent when the effective stress state is within the zone of potential instability.

It should also be mentioned that the instability line is the envelope of peak deviator stresses and zone of potential instability may be significant only if the peak occurs well before the failure. This means that the significant pre-failure strain softening should take place for the zone of potential instability to be significant.

Fig. 5.11a: Zone of potential instability and instability line for undisturbed marine clay

Fig. 5.11b: Comparison of instability line and the envelope of peak deviator stresses in CIU tests

5.3.4 Comparison with the Threshold Deviator Stress Level

Previous studies have shown that there exists a threshold deviator stress level, also referred to as the upper yield strength, below which creep rupture or failure will not take place (Singh and Mitchell, 1968; Finn and Shead, 1973; Campanella and Vaid, 1974). When the deviator stress level during undrained creep is higher than the threshold deviator stress level, failure or creep rupture occurs. Threshold deviator stress level is the lowest deviator stress level during creep at which the failure occurs and is expressed as the percentage of the peak deviator stress (q_m) obtained from conventional deformation-controlled undrained tests. However, there exists a dilemma in choosing the value of q_m since q_m is rate dependent and higher the shearing rate, higher will be the value of peak deviator stress obtained in CIU test. This issue has not been addressed in the published literature on undrained creep. To analyze the data from undrained creep tests, as presented in this chapter, peak deviator stress of 132kPa (q_m), which was obtained from the fastest CIU test CIU3 (1 mm/min) was chosen as reference for calculating the deviator stress levels in various undrained creep stages. Table 5.1 (see section 5.2) lists the deviator stress levels at the start of undrained creep stages in which instability occurred in various tests. It

can be seen that the lowest deviator stress level at which instability occurred, or threshold deviator stress level, was $0.775q_m$ (102 kPa) for undisturbed Singapore marine clay. Failure did not occur during creep at deviator stress levels below 0.775. The threshold deviator stress level and the maximum deviator stress (q_m) are shown in Fig. 5.12 together with all the stable effective stress states in undrained creep tests. It can be observed from this figure (see encircled points) that there were many effective stress states in which the deviator stress level was higher than the threshold deviator stress level, still the instability did not occur. It can be seen that all such stable effective stress states were below the zone of potential instability. Since it has been shown in section 5.3.3 that the effective stress state must be within the zone of potential instability for instability to occur, it can be concluded that the governing condition for the instability to occur is that the effective stress state should be within the zone of potential instability. A deviator stress level, higher than the threshold may not necessarily lead to failure unless the effective stress state is suitable. Also, considering the fact that the determination of deviator stress level is strain-rate dependent while the failure envelope is not dependent upon the strain rate, it will be more reliable to study the undrained creep behaviour in terms of effective stress conditions (q/p' with respect to the $(q/p')_{\text{failure}}$).

Fig. 5.12: Stable stages compared with threshold deviator stress level

From the above discussion of the results of undrained creep tests, it can be concluded that a instability line and a 'zone of potential instability' can be defined for undisturbed Singapore marine clay, similar to the sand (Lipinsky, 2001; Chu and Leong, 2002). The instability line coincides with the envelope of peak deviator stresses in CIU tests. The zone of potential instability is the zone bound by the instability line and the failure envelope determined from CIU tests. For instability to occur, the effective stress state at the beginning of creep must be within this zone. Instability will not occur at effective stress states below the zone of potential instability even if the deviator stress level is higher than the threshold deviator stress level. The observation that the instability will not occur if the effective stress state is below the instability line has been observed for sand (Lade, 1994; Lipinsky, 2001). However, Leong and Chu (2002) have shown that creep can bring the stress state into the zone of potential instability even if the initial effective stress state is slightly below the instability line. Among the undrained creep tests conducted on undisturbed Singapore marine clay, creep was not found to bring the stress state into the zone of potential instability. Instability occurred only when the effective stress state at the start of creep was already on the instability line or inside the zone of potential instability. However, if much longer time (in the order of 20000 to 30000 minutes) time is allowed for undrained creep in a state when the effective stress state is slightly below the zone of potential instability, instability may probably occur. More undrained creep tests, with much longer time duration allowed for each creep stage, are required to come to a definite conclusion in this regard.

The discussion above shows that due attention should be paid to the onset of instability in clays. During undrained creep tests, it is the onset of instability which is more critical than the failure or creep rupture itself, because instability occurs before the failure envelope and leads to the failure. Since instability was found to occur before the effective stress state reaches the failure envelope, choosing the failure envelope for design effective friction angle may lead to an unconservative design, especially when the problem of undrained creep is foreseen. This is particularly important for slope stability problems, deep excavations and structures built on shallow foundations in clayey soils. For example, highway embankments built over deposits of clayey soils may fail due to increase in pore pressure due to creep well before the effective friction angle at failure is mobilized. Duncan and Buchignani

(1973) had attributed the failure of an underwater slope in San Francisco Bay Mud to undrained creep, where mobilized friction angle at failure was lesser than that determined from laboratory tests. Similarly, Rowe and Hinchberger (1998) had also reported an increase in pore pressure due to undrained creep at constant load, which indicated progressive failure of Sackville test embankment.

5.4 CONCLUSIONS

Undrained creep tests were conducted on isotropically consolidated specimens of undisturbed Singapore marine clay and the results were compared with the results of conventional undrained compression tests. Special emphasis was given to the proper measurement of pore water pressure to determine the effective stress variation during undrained creep. Following conclusions can be made.

1. The pore water pressure rises during undrained creep in clay and leads to the movement of effective stress state towards the failure envelope (e.g. Fig. 5.8a). If the pore water pressure becomes constant after some increase in its value during creep, it is an indication that instability may not occur under given stress conditions. However, if the pore water pressure is found to be continuously increasing, it indicates that instability, characterized by rapid increase in axial strain and axial strain rate may occur soon.
2. An 'instability line' can be defined for undisturbed marine clay (see Fig. 5.11a). When the effective stress state is below this line, instability does not occur. The instability line coincides with the line or envelope passing through the peaks of effective stress paths of CIU and CAU tests.
3. A 'zone of potential instability' can be defined for undisturbed marine clay, which is bound by the instability line and the effective failure envelope of the soil (see Fig. 5.11a), where the effective failure envelope is the envelope of maximum q/p' values recorded in CIU and CAU tests. When the effective stress state of soil element is in the zone of potential instability, instability may occur and lead to the failure of the specimen. Since the instability line coincides with the envelope of peak deviator stresses obtained in CIU tests,

the envelope of peak deviator stress is as important as the large strain failure envelope for soils showing strain softening.

4. During undrained creep, instability will only occur if the initial effective stress state is within the zone of potential instability. If the effective stress state is below the zone of potential instability, instability will not occur even if the deviator stress is higher than the threshold deviator stress level required for causing failure in creep. Therefore, the location of the effective stress state in the zone of potential instability may be a necessary condition for instability to occur. However, for sand, undrained creep can bring the effective stress state within the zone of potential instability and may cause instability. Whether the same is true for clay needs to be further studied.
5. In an undrained creep stage, in which instability does not occur, the axial strain rate continuously reduces and does not reach a constant value, indicating that only primary creep occurs during such stages. In the undrained creep stages, in which instability occurs, the axial strain rate initially decreases until a transient minimum strain rate is reached. The axial strain appears to remain constant or starts to increase slowly after the transient minimum strain rate. This appears to be the phase of secondary creep. Onset of instability occurs after a significant time-interval has passed after the transient minimum strain rate is reached. Secondary creep may be used as an indication of the imminent instability for Singapore marine clay.
6. Undrained creep can cause the failure of the specimen at a deviator stress much lower than that obtained from conventional CIU tests. Lowest value of q at which failure occurred was only 77.5% (test CR7) for undisturbed Singapore marine clay.

Finally, it can be concluded that the concept of pre-failure instability during undrained creep, which is well established for sand, is applicable to the undisturbed marine clay. A study on reconstituted marine clay will be useful to examine the undrained creep behaviour further. The results of undrained creep tests on reconstituted marine clay will be presented in Chapter 6.

CHAPTER 6

EFFECT OF LOADING RATE, LOADING MODE AND UNDRAINED CREEP ON THE STRESS-STRAIN BEHAVIOUR OF RECONSTITUTED MARINE CLAY

6.1 INTRODUCTION

It was observed in Chapter 4 and Chapter 5 that the stress-strain behaviour of undisturbed marine clay is affected significantly by the loading rate, loading mode and undrained creep. Time to failure (in terms of shearing rate or time given for creep) was found to affect the pore pressure generation, which in turn affected the strength and overall stress-strain behaviour of the undisturbed marine clay in different loading rates, loading modes and undrained creep. The difference in pore pressure could be related to the time dependent creep or to the different response of the structure of undisturbed clay under different loading rates and loading modes. Also, some differences, such as pre-mature failure of specimen in load-controlled and deformation-controlled CIU and CID tests, could be caused by the natural variability and non-homogeneity of the undisturbed samples. Therefore, it is important to study the behaviour of reconstituted marine clay under different loading rates and loading modes. Reconstituted specimens are less influenced by the effects of structure, variability and non-homogeneity, thus leaving the time as the dominating factor contributing to the effects of loading mode and loading rate.

Also, it is well known that the general stress-strain behaviour of reconstituted clay may be significantly different from the behaviour of undisturbed clay due to the destruction of structure (fabric and bonding) of natural soil during remoulding

(Mitchell, 1970; Bishop, 1971; Graham and Li, 1985; Burland, 1990; Burland et al., 1996; Amorosi et al., 2001). Lefebvre and LeBoeuf (1987) concluded that the effect of strain rate on the stress-strain behaviours of structured and destructured clay were fundamentally different. On the other hand, the effect of shearing rate on the behaviour of reconstituted Haney clay was found to be similar to the effect on the behaviour of undisturbed clay (Vaid, 1988).

To examine the effect of loading rate and loading mode and general stress-strain behaviour of reconstituted clay, undrained compression, drained compression and undrained creep tests were conducted on reconstituted Singapore marine clay. The marine clay was reconstituted by mixing it thoroughly with water to form a slurry. The slurry was then consolidated into soil using a consolidation tank. The tests included deformation-controlled and load-controlled CIU and CID tests at different shearing rates and two multi-stage undrained creep tests.

In this chapter, the effect of loading rate and loading mode on the undrained behaviour of reconstituted Singapore marine clay is studied first followed by the drained behaviour. The behaviour in undrained creep tests is studied thereafter. Finally, a comparison of the behaviours of undisturbed marine clay and reconstituted marine clay is made. A discussion on the observed behaviour is made subsequently.

6.2 FAILURE ENVELOPE

The failure envelope for the reconstituted Singapore marine clay was determined using four undrained tests and two drained tests as shown in Fig. 6.1a. The four undrained tests (RCU1, RCU8, RCU15 and RCU16) were conducted on specimens isotropically consolidated under effective confining pressures of 200, 300, 450 and 600 kPa respectively. The two drained tests, RCD3 and RCD9, were conducted on specimens isotropically consolidated under effective confining stresses of 200 kPa and 300 kPa respectively. A non-linear failure envelope (Fig. 6.1a) was obtained from these tests. Non-linearity of failure envelope for cohesive and cemented soils is a well-reported phenomenon (Lo and Morin, 1972; Burland et al., 1996; Baker, 2004; Asghari et al., 2003). The shape of the initial part of the failure envelope could not be ensured as no tests were conducted on the specimens consolidated under very

low confining stresses. However, the effective stresses at failure in the test results presented later in this chapter were all in the zone above the stress path of test RCU1 (Fig. 6.1a) and initial shape of the failure envelope does not affect the interpretation of the results. The stress-strain curves for the four undrained and two drained tests are presented in Fig. 6.1b. The stress-strain curves for undrained tests are almost flat at the peak. A brittle type of failure occurred when shear band occurred. In drained tests, a sharp peak occurred. This was followed by shear band and rapid drop in deviator stress. Further discussion on drained and undrained behaviour of this clay will be made later in this chapter.

It may be recalled that it was established in Chapter 5 that the instability line for undisturbed marine clay is the line passing through the peaks of effective stress paths for different CIU tests. In the case of reconstituted clay, the failure occurs at the peak deviator stress. Hence the failure envelope should also be the instability line. This point will be verified later in section 6.7 during the discussion on undrained creep behaviour of reconstituted marine clay.

Fig. 6.1a: Failure envelope for reconstituted marine clay

Fig. 6.1b: Stress-strain curves in CIU and CID tests

6.3 EFFECTS OF SHEARING RATE ON THE UNDRAINED BEHAVIOUR IN DEFORMATION-CONTROLLED TESTS

Tests, similar to those conducted on undisturbed marine clay, were conducted on reconstituted marine clay to study the strain rate effect. Four deformation-controlled tests, RCU1, RCU2, RCU3 and RCU4 were conducted at strain rates of 0.005 mm/min, 0.05 mm/min, 0.2 mm/min and 0.5 mm/min respectively. A summary of load-controlled and deformation-controlled undrained tests is presented in Table 6.1. It should be mentioned that the maximum permissible value of axial strain rate (BS1377, 1990) was 0.004 mm/min assuming a failure axial strain of 7% (see Appendix A).

6.3.1 Strength and Stress-Strain Behaviour

Figures 6.2a to 6.2h present the results of the deformation-controlled undrained tests conducted on the specimens consolidated under an effective confining pressure of 200 kPa, which is the same as that used for undisturbed marine clay. It can be observed from the stress-strain curves shown in Fig. 6.2a that the peak deviator

stress, q_m , increased with the increase in strain rate. The variation of peak deviator stress with strain rate is shown in Fig. 6.2b. The peak deviator stress increased linearly with the logarithmic increase in strain rate. An average increase of 5% in peak deviator stress occurred for one log-cycle increase in strain rate. This percentage is lower than that observed for most other reconstituted clays. For example, an average increase of about 10% in undrained shear strength per log cycle of strain rate (varying from 0.002 to 1%/min) was observed by Richardson and Whitman (1963) for remoulded Mississippi river valley clay. Same percentage increase (10%) was also observed (at strain rates varying from 0.0008 to 2%/min) by Lefebvre and LeBoeuf (1987) for three types of destructured clays (Grande Baleine Clay, Olga Clay and B-6 Clay) and by Sheahan et al. (1996) for re-sedimented Boston Blue clay (at strain rates varying from 0.0008 to 0.8%/min). It has been observed, for most of the reconstituted clays that the undrained shear strength increased by 10% to 20% with a ten-fold increase in increase in strain rate (Akai et al., 1975; Vaid, 1988; Zhu et al., 1999). It may be pointed out that the increase in undrained shear strength per log cycle of strain rate was 6% (section 4.3.1 in Chapter 4) for undisturbed Singapore marine clay, which is close to the increase for reconstituted clay as shown here.

Table 6.1: Summary of undrained tests on reconstituted marine clay

Test	rate	units	w- pre test (%)	w- post test (%)	q_m (kPa)	c_u (kPa)	q/p'_{max} (mid)	ϕ' at failure [^] (mid)	ϕ' at peak* (mid)	ϕ'_{av} at peak [#] (top & base)
RCU1	0.005	mm/min	51.5	46.5	103.1	51.5	0.920	23.5	23.0	23.0
RCU2	0.05	mm/min	51.7	46.8	108.2	54.1	0.931	23.8	21.7	21.8
RCU3	0.2	mm/min	50.7	46.9	112.9	56.5	0.853	22.3	21.1	21.1
RCU4	0.5	mm/min	50.6	46.8	114.6	57.3	0.873	22.3	21.3	22.6
RCU5	0.25	N/min	51.5	47.1	111.2	55.6	0.928	23.7	22.9	22.8
RCU6	2.5	N/min	51.0	47.3	118.9	59.5	0.907	23.2	22.0	22.5
RCU7	25	N/min	50.9	46.9	123.2	61.6	0.876	22.5	21.5	22.5

w ~ % water content; q_m ~ peak deviator stress; q/p'_{max} ~ maximum stress ratio; c_u ~ undrained shear strength;

[^] ϕ' at failure is the effective friction angle at maximum σ_1'/σ_3' (or maximum q/p')

* ϕ' at peak is the effective friction angle at maximum $(\sigma_1' - \sigma_3')$ (or maximum q)

[#] ϕ'_{av} at peak is the average of effective friction angles (at maximum $\sigma_1' - \sigma_3'$) calculated using σ_1' and σ_3' estimated from pore pressures at top and base of the specimen

As shown in Fig. 6.2a, the reduction of deviator stress is gradual after the peak deviator stress, indicating that the pre-failure strain-softening is small in reconstituted marine clay. Shear band was observed to occur after large axial strain at the points indicated on the curves in Fig. 6.2a. When the shear band occurred, the axial load and deviator stress dropped sharply. This has been found to be typical behaviour of many reconstituted clays (Burland, 1990). Curves for tests RCU2 and RCU3 indicate that the deviator stress reaches an almost constant value after the drop, indicating that the residual state has been reached.

Fig. 6.2a: Stress-strain curves of deformation-controlled CIU tests

Fig. 6.2b: Variation of peak deviator stress with logarithmic increase in strain rate

6.3.2 Variation of Pore Water Pressure

The variation of mid-height pore water pressure is shown in Fig. 6.2c. It can be observed that the magnitude of pore water pressure increased as the strain rate decreased and the highest pore water pressure was generated in the slowest test RCU1 (0.005 mm/min). In other words, pore water pressure increased with the increase in time of shearing, which is due to the generation of pore pressure caused by undrained creep (Lefebvre and LeBoeuf, 1987). The non-uniformity of pore water pressure distribution within the specimens increased with the increase in strain rate, as shown in Figs. 6.2d to 6.2g using the plots of pore water pressures measured at the top, bottom and middle of the specimen in each test. While the pore pressures measured at the top, bottom and mid-height were similar in the slowest test RCU1 (Fig. 6.2d), the difference between bottom and mid pore pressures increased with increasing rate (Fig. 6.2e and 6.2f). A large difference of about 20 kPa was observed between the bottom and mid-height pore pressures in the fastest test RCU4 (0.5 mm/min) (Fig. 6.2g). These observations are consistent with the observations made for the undisturbed marine clay in Chapters 4.

Fig. 6.2c: Variation of mid pore pressure in deformation-controlled undrained tests

Fig. 6.2d: Pore pressure variation in test RCU1

Fig. 6.2e: Pore pressure variation in test RCU2

Fig. 6.2f: Pore pressure variation at top, bottom and mid-height of specimen in test RCU3

Fig. 6.2g: Pore pressure variation at top, bottom and mid-height of specimen in test RCU4

6.3.3 Effective Stress Paths and Failure

The effective stress paths plotted using the pore water pressure measured at the mid-height of the specimen are shown in Fig. 6.2h. The effective stress paths were affected significantly by the strain rate. It is because as the strain rate increased, the amount of pore water pressure generated reduced (see Fig. 6.2c) thus the mean effective stress increased. Therefore, the stress paths of faster tests (RCU3 and RCU4) are shifted towards the right hand side, thus indicating the expansion of the yield surface. However, all the effective stress paths reached the failure envelope determined in Fig. 6.1a earlier. The effective stress paths of the tests conducted at faster rate reach the failure envelope at a higher deviator stress. The uniqueness of effective failure envelope at maximum obliquity has been earlier reported by Lefebvre and LeBoeuf (1987) and Sheahan et al. (1996). Lefebvre and LeBoeuf (1987) attributed it to a lowering of failure envelope at higher strain rate. The results presented above show that this could be due to the curvature of the effective failure envelope for Singapore marine clay.

Fig. 6.2h: Effective stress-paths determined using mid-height pore pressure in deformation-controlled undrained tests on reconstituted marine clay

6.3.4 Selection of Proper Strain Rate in Deformation-Controlled Tests

It is important to mention here that the calculated time to failure of 1760 minutes and the maximum permissible axial strain rate of 0.004 mm/min were obtained (see Appendix A) for undrained tests on reconstituted marine clay as calculated based on the volume change vs. time curve in isotropic consolidation (Head, 1986; BS1377, 1990). In this calculation, axial strain at failure (ϵ_f) is assumed to be 7% because peak deviator stress was reached (Fig. 6.2a) at about this axial strain. However, Fig. 6.2a shows that the failure occurred at 11% to 16%, which will give maximum permissible strain rates of 0.006 and 0.009 mm/min respectively. Therefore, it is difficult to choose the value of ϵ_f and hence the maximum permissible strain rate for reconstituted marine clay. Nevertheless, pore pressure was reasonably uniform in test RCU2 (max. difference of 8 kPa between bottom and middle pore pressure), which was conducted at 0.05 mm/min (see Fig. 6.2e), a rate 5 to 12 times higher than the estimated maximum permissible rates of 0.009 mm/min and 0.004 mm/min. It shows that the method given by BS1377 can still be used reliably for undrained test on

reconstituted Singapore marine clay. Whether the same is true for other clays, needs to be verified by pore pressure measurement at the mid-height.

6.4 EFFECT OF SHEARING RATE ON THE UNDRAINED BEHAVIOUR IN LOAD-CONTROLLED TESTS

Three load-controlled tests, namely RCU5, RCU6 and RCU7, were conducted at load increment rates of 2.5 N/min, 0.25 N/min and 25 N/min respectively to study the effect of load increment rate on the undrained shear strength of reconstituted marine clay. The results of the three load-controlled tests are presented in Figures 6.3a to 6.3h.

6.4.1 Strength and Stress-Strain Behaviour

The stress-strain curves and the variation of peak deviator stress with the load increment rate are shown in Figs. 6.3a and 6.3b respectively. Peak deviator stress was reached at about 5% axial strain (Fig. 6.3a). The drop in deviator stress after peak was gradual; indicating that the strain softening was small. Rapid drop in q occurred after the formation of shear band. The shear band, in general, occurred at the maximum q/p' and was identified by the rapid drop in load and q/p' .

It can be observed from Figs. 6.3a and 6.3b that the peak deviator stress, q_m , increased with the increase in load increment rate. A linear increase in peak deviator stress (Fig. 6.3b) was observed with logarithmic increase in axial strain rate. The peak deviator stress, q_m , increased by an average of 6% (with respect to q_m in slowest test, RCU5) due to one log cycle increase in load increment rate.

Fig. 6.3a: Stress-strain curves for load controlled undrained tests

Fig. 6.3b: Variation of peak deviator stress with logarithmic increase in load increment rate

6.4.2 Variation of Pore Water Pressure

Variation of mid-height pore water pressure is shown in Figures 6.3c. It clearly shows that the mid-height pore water pressure reduced as the load increment rate increased, which is consistent with the observations made for deformation-controlled tests. It can be seen from the plots of pore pressures measured at the top, bottom and mid-height of the specimen in the three tests in Figs. 6.3d, 6.3e and 6.3f that the non-uniformity of pore water pressure within the specimen increased with the increase in load increment rate. This point is further illustrated by the plot of difference between bottom and mid pore pressure measurements in the three tests, as shown in Fig. 6.3g. It shows that a difference of about 12 kPa occurred between the bottom and mid-height pore pressures in the fastest test RCU7 (25 N/min).

Fig. 6.3c: Variation of mid pore pressure in load controlled undrained tests

Fig. 6.3d: Variation of pore pressures measured from top, bottom and mid-height of specimen in test RCU5

Fig. 6.3e: Variation of pore pressures measured from top, bottom and mid-height of specimen in test RCU6

Fig. 6.3f: Variation of pore pressures measured from top, bottom and mid-height of specimen in test RCU7

Fig. 6.3g: Comparison of non-uniformities of pore pressure in tests RCU5, RCU6 and RCU7

6.4.3 Effective Stress Paths and Failure

The effective stress paths determined using mid-height pore water pressures in the three load-controlled tests are shown in Fig. 6.3h. It shows that the stress paths are shifted towards the right hand side as the load increment rate increases, because of lesser excess pore water pressure in faster tests, which leads to an increase in mean effective stress, p' . However, the effective stress paths of all the three tests reach the same failure envelope. This observation is similar to the observation in deformation-controlled tests. Hence the observed increase in peak deviator stress with increase in strain rate is consistent with the failure in effective stress space and shows the uniqueness of the failure envelope for reconstituted marine clay, irrespective of the strain rate. Failure occurs when the effective stress state within the specimen reaches the failure envelope. However, in the tests conducted at faster rate, the stress path reaches the failure envelope at a higher mean effective stress and higher deviator stress. Thus, shearing at faster rate is equivalent to shearing of a specimen consolidated under higher effective confining stress.

Fig. 6.3h: Effective stress paths based on pore water pressure measured at mid-height in load controlled undrained tests

6.4.4 Summary of the Observations in Load-Controlled Tests

The observations in load-controlled tests were similar to those in deformation-controlled tests. The undrained shear strength increases and excess pore water pressure decreases with the increase in load increment rate. Higher non-uniformity of pore water pressure within the specimen is developed in the faster tests compared to the slow tests. The failure envelope remains unaffected by rate of load increment.

6.5 EFFECT OF LOADING MODE ON THE UNDRAINED BEHAVIOUR OF RECONSTITUTED MARINE CLAY

To study the effect of loading mode on the stress-strain behaviour of reconstituted marine clay, the results of load-controlled (LC) and deformation-controlled (DC) tests are plotted together in Figs. 6.4a to 6.4g. Load-controlled tests are shown using broken lines while deformation-controlled tests using continuous lines in these figures. For the sake of clarity, only two pairs of tests are shown [RCU1 (DC) is compared with RCU5 (LC) and RCU4 (DC) is compared with RCU7 (LC)]. Other tests showed similar trends.

Before the comparison of results, it may be mentioned that to keep a pair of load-controlled and deformation-controlled tests comparable, load increment rate in load-controlled test was chosen in such a way that the time taken to reach the peak deviator stress will be the same as the time taken in deformation-controlled test as shown in Fig. 6.4a for tests RCU4 (DC) and RCU7 (LC). It was assumed that the peak deviator stress in load-controlled test will be the same as that obtained in deformation-controlled test.

From the stress-strain curves shown in Figs. 6.4b-(i) and 6.4b-(ii), it can be noted that the load-controlled tests attained higher peak deviator stress than the comparable deformation-controlled tests. The difference between the peak deviator stresses in load-controlled and deformation-controlled tests can also be seen from Fig. 6.4c. In this figure, the peak deviator stresses in load-controlled and deformation-controlled tests are plotted versus axial strain rate, where the average axial strain rate in load-

controlled test was calculated by dividing the axial strain at peak deviator stress by the time taken in reaching the peak. Fig. 6.4c shows that the peak deviator stresses in load-controlled test were up to 10 kPa (about 8%) higher than the peak deviator stress in comparable deformation-controlled test. Similar observation was reported by Lacasse (1995) for Haga clay, for which constant rate of stress loading gave as much as 20% higher undrained shear strength than the constant rate of strain loading, when the time to failure was only 10 minutes in both types of tests.

Fig. 6.4a: Comparison of loading time to peak q in comparable load-controlled and deformation-controlled tests

Fig. 6.4b-(i): Comparison of stress-strain curves in LC and DC tests, RCU1 & RCU5

Fig. 6.4b-(ii): Comparison of stress-strain curves in LC and DC tests, RCU4 and RCU7

Fig. 6.4c: Comparison of peak deviator stresses obtained in LC and DC tests

The variation of pore water pressures in load-controlled and deformation-controlled tests are compared in Fig. 6.4d-(i) and 6.4d-(ii). It can be seen that the pore water pressure was lesser in load-controlled tests than that in comparable deformation-controlled tests.

It should be pointed out that for undisturbed marine clay, the comparable load-controlled and deformation-controlled tests showed similar shear strengths and similar pore pressures. One possible explanation for the difference between peak deviator stresses in load-controlled and deformation-controlled tests for reconstituted marine clay can be given with the help of the pore water pressure and axial strain rate variation as shown in Figs. 6.4d and 6.4e. It can be seen from Fig. 6.4e that the axial strain rate in a load-controlled test (e.g. RCU5) becomes higher than corresponding deformation-controlled test (e.g. RCU1) much before the peak deviator stress is reached. Correspondingly, the later part of the increase in deviator stress takes place in shorter time in load-controlled test. Hence the pore water pressure is lower (see Fig. 6.4d) in load-controlled test (RCU5 & RCU7) due to lesser time for creep induced pore pressure to develop. Smaller pore pressure leads to increase in mean effective stress (see Fig. 6.4g) and higher peak deviator stress.

Fig. 6.4d-(i): Comparison of pore pressure variation in LC and DC tests, RCU1 & RCU5

Fig. 6.4d-(ii): Comparison of pore pressure variation in LC and DC tests, RCU4 and RCU7

Fig. 6.4e-(i): Comparison of strain rate variations in tests RCU1 & RCU5

Fig. 6.4e-(ii): Comparison of strain rate variations in tests RCU4 & RCU7

Richardson and Whitman (1963) had reported an increase in deviator stress and corresponding decrease in pore pressure due to a sudden increase in strain rate during undrained shearing of remoulded Mississippi river valley clay. The difference between load-controlled and deformation-controlled tests as observed here appears to be caused by similar phenomenon. Therefore, higher strength in load-controlled tests than deformation-controlled tests is a result of rate-effect on the pore pressure variation.

Fig. 6.4f: Variation of axial strain rate with time in LC and DC tests

The effective stress paths of load-controlled and deformation-controlled tests are compared in Fig.6.4g. It is observed that all the tests reached the same failure envelope. It shows that the failure envelope is unique for tested reconstituted marine clay regardless of loading mode.

To summarize, load-controlled tests give higher undrained shear strength than corresponding deformation-controlled tests. Lesser pore water pressure was developed in load-controlled tests than in deformation-controlled tests, which is due to the higher axial strain rate before peak in load-controlled tests than in deformation-controlled tests. Thus, the higher shear strength in load-controlled test is a result of rate effect on pore pressure and hence the effective stresses. The undrained shear strength increases linearly with logarithmic increase in shearing rate in both load-controlled and deformation-controlled tests.

Fig. 6.4g: Comparison of stress paths in load controlled and deformation-controlled tests

6.6 DRAINED BEHAVIOUR OF RECONSTITUTED MARINE CLAY

To study the effect of loading rate and loading mode on the drained behaviour of reconstituted marine clay a total of 9 CID tests, RCD1 to RCD9, comprising of both load-controlled and deformation-controlled tests at varying shearing rates were conducted. Four tests, RCD1 to RCD4, were conducted on specimens consolidated under 200 kPa while five tests, RCD5 to RCD9, were conducted on specimens consolidated under 300 kPa effective confining stress. In all the tests, drainage was allowed from both top and bottom of the specimen. A mid-height pore water pressure probe was used to measure undissipated pore water pressure at the mid-height of the specimen. In a truly drained test, complete dissipation of pore water pressure should occur. However, it is known that if the shearing rate in drained test is not slow enough, complete drainage will not take place and excess pore water pressure will be generated within the specimen (Bishop and Henkel, 1962; Newson et al., 1997). However, the effect of excess pore water pressure on the drained shear strength of clay is not fully understood. The results shown in Figures 6.5a to 6.8d will throw more light on this aspect of the drained behaviour of clay. A summary of all the drained tests on reconstituted marine clay is given in Table 6.2.

Table 6.2: Summary of drained tests on reconstituted marine clay

Test	rate	units	w-pre test (%)	w-post test (%)	p_0	q_m (kPa)
RCD1	0.01	mm/min	52.2	38.1	200	187
RCD2	0.182	N/min	51.5	39.4	200	170
RCD3	0.002	mm/min	49.3	37.9	200	206
RCD4	0.054	N/min	49.2	38.5	200	212.4
RCD5	0.26	N/min	51.3	35.5	300	245.4
RCD6	0.11	N/min	50.6	35.6	300	281
RCD7	0.06	N/min	49.3	34.7	300	293
RCD8	0.01	mm/min	50.2	35.6	300	252
RCD9	0.0035	mm/min	49.6	35.2	300	296

6.6.1 Effect of Shearing Rate on the Drained Behaviour in Deformation-Controlled Tests

The stress-strain curves of two pairs of deformation-controlled CID tests on reconstituted marine clay are shown in Fig. 6.5a and 6.5b. Specimens in tests RCD1 and RCD3 (Fig. 6.5a) were consolidated under effective confining stress of 200 kPa while those in RCD8 and RCD9 (Fig. 6.5b) were consolidated under 300 kPa. Figs. 6.5a and 6.5b show that the faster tests, RCD1 (0.01 mm/min) and RCD8 (0.01 mm/min) attained lower peak deviator stress than corresponding slow tests, RCD3 (0.002 mm/min) and RCD9 (0.0037 mm/min) respectively. One of the explanations for this behaviour can be given based on the variation of volume change and excess mid-height pore water pressures as shown in Figs. 6.5c to 6.5f. The non-uniformity of pore water pressure increased significantly with the increase in axial strain rate in both the sets of drained tests (Fig. 6.5e and 6.5f), while volumetric strain decreased with the increase in axial strain rate (Fig. 6.5c and 6.5d). This means that a drained test conducted under fast shearing rate is actually not fully drained, or in other words, is partially undrained, because pore water pressure can not fully dissipate. This leads to a reduction in drained shear strength. The peak deviator stress reduced by 9% and 15% respectively in tests RCD1 and RCD8 compared to the tests RCD3 and RCD9. It may be noted that the excess pore water pressure drops rapidly after the formation

of shear band, which shows the migration of pore water within the specimen due to non-homogeneous deformation caused by shear band formation.

The results discussed above indicate that the drained tests conducted at fast shearing rates are not truly drained tests and may not be representative of drained behaviour. A proper drained test must be conducted with pore pressure measurement to ensure that the excess pore pressure does not increase beyond the acceptable limit of 4% of the effective confining stress (BS1377, 1990).

Fig. 6.5a: Stress-strain curves in deformation-controlled CID tests with $p_0 = 200$ kPa

Fig. 6.5b: Stress-strain curves in deformation-controlled CID tests with $p_0 = 300$ kPa

Fig. 6.5c: Volume change in deformation-controlled CID tests with $p_0 = 200$ kPa

Fig. 6.5d: Volume change in deformation-controlled CID tests with $p_0 = 300$ kPa

Fig. 6.5e: Excess mid pwp in deformation-controlled CID tests with $p_0 = 200$ kPa

Fig. 6.5f: Excess mid pwp in deformation-controlled CID tests with $p_0 = 300$ kPa

6.6.2 Effect of Shearing Rate on the Drained Behaviour in Load-Controlled Tests

Five load-controlled drained tests, RCD2, RCD4, RCD5, RCD6 and RCD7, were conducted at load increment rates of 0.182, 0.054, 0.26, 0.11 and 0.06 N/min respectively. Tests RCD2 and RCD4 (Figs. 6.6a and 6.6c) were conducted on specimens consolidated under 200 kPa while tests RCD5, RCD6 and RCD7 (Figs. 6.6b and 6.6d) were conducted on specimens consolidated under 300 kPa effective confining pressure.

Stress-strain curves of the two sets of load-controlled drained tests are shown in Figs. 6.6a and 6.6b. It can be observed that the peak deviator stress reduced with the increase in load increment rate. This is similar to the effect of shearing rate in deformation-controlled drained tests.

The explanation for the reduction in drained shear strength with the increase in load increment rate is similar to that given for deformation-controlled tests and can be given using Figs. 6.6c, 6.6d and 6.6e. It can be seen that lesser volume change (Fig. 6.6c and 6.6d) occurred in faster tests and larger non-uniformity of pore water pressure (Fig. 6.6e) developed in faster load-controlled tests RCD4, RCD6 and RCD7. This resulted in partially undrained condition and hence reduced shear strength.

Variation of axial strain rate with time is shown in Fig. 6.6f. The axial strain rate increased continuously throughout the test. The non-uniformity of pore water pressure continuously increased (Fig. 6.6e) as a result. Onset of instability was found to occur at the peak deviator stress. At the onset of instability, the axial strain rate increased rapidly (Fig. 6.6f) and specimen collapsed accompanied by further increase in pore water pressure (Fig. 6.6e). This shows that the test is not fully drained towards the end of load-controlled tests.

Fig. 6.6a: Stress-strain curves in deformation-controlled CID tests with $p_0 = 200$ kPa

Fig. 6.6b: Stress-strain curves in deformation-controlled CID tests with $p_0 = 300$ kPa

Fig. 6.6c: Volume change in load controlled CID tests with $p_0 = 200$ kPa

Fig. 6.6d: Volume change in load controlled CID tests with $p_0 = 300$ kPa

Fig. 6.6e: Excess mid-height pore pressure in two representative load-controlled CID tests

Fig. 6.6f: Variation of axial strain rate with time in load-controlled CID tests

6.6.3 Effect of Loading Mode on the Drained Behaviour

Two pairs of comparable deformation-controlled and load-controlled drained tests, conducted on specimens consolidated under 200 kPa (RCD3 and RCD4) and 300 kPa (RCD7 and RCD9) are compared in Figs. 6.7a to 6.7f. These were the tests conducted at relatively slower rates, which developed lesser non-uniformity of pore pressure. The load increment rates for load-controlled tests were chosen in such a way that the time required to reach the expected peak would be the same as that required in corresponding deformation-controlled test.

The stress-strain curves in Figs. 6.7a and 6.7b show that the comparable load-controlled and deformation-controlled tests (e.g. RCD3 and RCD4) reached almost the same peak deviator stress. A comparison of volume change behaviours (6.7c and 6.7d) shows that the total volumetric strains at peak q were similar in comparable load-controlled and deformation-controlled tests. The excess pore pressures (Fig. 6.7e and 6.7f) at peak deviator stress were less than 20 kPa (6% to 9% of the peak deviator stress) in all the four tests compared here. These observations show that if the chosen strain rate is such that the excess pore pressure is small, the behaviours in load-controlled and deformation-controlled tests will be similar.

Fig. 6.7a: Comparison of stress-strain curves in DC and LC CID tests with $p_0 = 200$ kPa

Fig. 6.7b: Comparison of stress-strain curves in DC and LC CID tests with $p_0 = 300$ kPa

Fig. 6.7c: Comparison of volumetric strains in LC and DC drained tests with $p_0 = 200$ kPa

Fig. 6.7d: Comparison of volumetric strains in DC and LC drained tests with $p_0 = 300$ kPa

Fig. 6.7e: Comparison of excess mid pore pressures in LC and DC drained tests with $p_0 = 200$ kPa

Fig. 6.7f: Comparison of excess mid pore pressures in DC and LC drained tests with $p_0 = 300$ kPa

6.6.4 Effect of Loading Rate and Loading Mode on the Failure Envelope

To compare the effective stress-states at failure in load-controlled and deformation-controlled tests, the failure points are shown in Fig.6.8a. In this plot, excess pore water pressure has not been considered in calculating the mean effective stress. The load-controlled-tests are indicated by dark markers while deformation-controlled tests are shown by blank markers. Stress paths in both load-controlled and deformation-controlled tests conducted at faster rates are unable to reach the failure envelope determined from CIU tests. At faster rates, specimens in both load-controlled and deformation-controlled tests failed at deviator stresses well below the failure envelope. This pre-mature failure is due to the large non-uniformity of pore pressure which resulted in partially drained conditions. The effective stresses are unreliable in faster tests due to the large non-uniformity of pore pressures. At slower rates, the effective stress states in both load-controlled and deformation-controlled tests reached close to the failure envelope, although slightly below the failure envelope for some tests.

Fig. 6.8a: Comparison of failures in DC and LC tests

To examine the possible causes of early failure, the effective stress paths in various drained tests are plotted in Fig. 6.8b, with mean effective stress calculated using the mid-height pore water pressure. It is noteworthy that the peaks of all the stress paths, including those of fast tests (RCD6 and RCD8), reached very close to the failure envelope. This shows that the effective stress state at the mid-height of the specimen reached the failure envelope at smaller deviator stress due to the reduction in effective confining stress (p') caused by undissipated pore water pressure. It should be pointed out that the failure still seems to be governed by the effective failure envelope determined from properly conducted tests. It is also observed that the effective stresses at failure, as determined using mid-height pore pressure, in tests conducted at slow rates (e.g. RCD3 and RCD4) lie closer to the failure envelope than those determined using the applied back pressure.

Fig. 6.8b: Comparison of drained stress paths determined using mid-height pore pressure

It should be mentioned that the time to failure in drained test, as determined theoretically (BS1377, 1990) using the volume change data from isotropic consolidation stage of the reconstituted marine clay, is 6800 minutes (see Appendix A). A comparison of time to failure in load-controlled and deformation-controlled drained tests with the theoretical time to failure is shown in Fig. 6.8c through q vs. time plots. It shows that the time intervals to failure were 5200, 6400, 8400 and 8700

minutes respectively in tests RCD6, RCD9, RCD4 and RCD3. Test RCD6 attained a lower peak than the comparable test RCD9 while comparable tests RCD4 and RCD3 attained similar peaks. The excess pore pressures in these tests are shown in Fig. 6.8d. It can be seen that the undissipated pore pressure at failure was less than 20 kPa in tests RCD9, RCD4 and RCD3, in which time to failure was close to or greater than the theoretical time to failure. However, in test RCD6, in which the time to failure was much shorter (by 1600 minutes) than the theoretical time (6800 minutes), the undissipated pore pressure at failure was 30 kPa and led to lower peak deviator stress (Fig. 6.8c). These observations indicate that currently used method (Head, 1986; BS1377, 1990) of choosing the time to failure in drained tests is adequate. However, some pore pressure remains undissipated even when the time to failure is longer than the theoretical value and it leads to small reduction in drained shear strength. If the undissipated pore pressure at the mid-height can be measured and taken into account while calculating the effective stresses, it will give more accurate estimate of the failure envelope in drained tests. Therefore, measurement of pore water pressure during drained test should be done in routine laboratory testing. It gives an idea of undissipated pore pressure and helps to prevent the use of high strain rates, which may cause high non-uniformity of pore pressure and underestimation of drained shear strength and effective failure envelope.

Fig. 6.8c: Comparison of deviator stress variation in load-controlled and deformation-controlled CID tests

Fig. 6.8d: Excess pore pressure measured at mid-height in CID tests

To summarize, the observations on the loading rate and loading mode effects on the drained behaviour of reconstituted marine clay show that the drained behaviour is significantly affected by the loading rate and loading mode when tests are conducted at faster rates. At rates faster than those determined using isotropic consolidation data (BS1377, 1990), large pore pressure remains undissipated and leads to a reduction of mean effective stress and peak deviator stress. Hence, if the rate of shearing in drained test is not chosen carefully, it will lead to a behaviour of soil which is not truly drained and may seriously underestimate the drained shear strength and failure envelope. Load-controlled and deformation-controlled drained tests give similar drained shear strengths and failure envelope when conducted at suitably slow rates. However, some pore pressure remains undissipated at slow rates also. Mid-height pore pressure can be used to estimate the actual effective stresses within the specimen and hence avoids underestimation of failure envelope or friction angle. The method given in BS1377 (1990) for choosing the time to failure in drained tests on reconstituted clay is adequate, however, measurement of mid-height pore pressure should be done in routine testing to prevent any error in choosing the strain rate and to ensure that non-uniformity of pore pressure is within the limits.

6.7 UNDRAINED CREEP BEHAVIOUR OF RECONSTITUTED MARINE CLAY

Studies of undrained creep behaviour of clay have been conducted on both undisturbed clays (Walker, 1969; Arulanandan et al., 1971; Duncan and Buchignani, 1973; Finn and Shead, 1973; Vaid and Campanella, 1977; Lin and Wang, 1998) and reconstituted clays (Casagrande and Wilson, 1951; Akai et al., 1975; Wu et al., 1978; Kavazanjian and Mitchell, 1980; Mesri et al., 1981; Sheahan, 1995). The general stress-strain-time behaviours during undrained creep are similar in undisturbed and reconstituted clays. However, a comparative study of undrained creep on undisturbed and reconstituted samples of the same clay has not been reported in literature. Considering the fact that the effects of loading mode on the stress-strain behaviours were different (see section 6.5) for undisturbed and reconstituted marine clays, it is necessary to examine whether the undrained creep behaviours could also be different.

Two multi-stage undrained creep tests, RCR1 and RCR2, were conducted on reconstituted marine clay. A summary of the test conditions in the two undrained creep tests is shown in Table 6.2. In each stage the specimen was rapidly loaded to a pre-determined deviator stress level. The specimen was then left for undrained creep. The stress ratios (q/p') at the start of each creep stage in tests RCR1 and RCR2 are shown in Table 6.2. Stress ratios at instability are also presented in the table. The results of the two undrained creep tests, RCR1 and RCR2 are shown in Figures 6.9a to 6.11b. The results are compared with the results of CIU tests on reconstituted marine clay.

Table 6.3: Summary of undrained creep tests RCR1 and RCR2

Test	p_0' (kPa)	(q/p') at start of creep stages								q/p' at instability
		stage 1	stage 2	stage 3	stage 4	stage 5	stage 6	stage 7	stage 8	
RCR1	200	0.740	0.912	0.923	0.971	-	-	-	-	0.972
RCR2	200	0.424	0.550	0.656	0.731	0.763	0.832	0.895	0.980	0.979

6.7.1 Test RCR1

Results of test RCR1 are presented in Figs. 6.9a to 6.9e. Results of tests RCU1 and RCU7 are also shown for comparison in Figs. 6.9a and 6.9b. There were four creep stages in test RCR1. The specimen was sheared rapidly for increasing the deviator stress level to start a stage of creep. The changes during the short duration (less than 1 minute to 3 minutes) of increase in deviator stress level are shown in the figures by dotted lines at the end of each creep stage. In stage 1, the specimen was rapidly loaded in undrained condition to point 1 with q/p' of 0.740 (Fig. 6.9a) and left for undrained creep. About 5% axial strain (6.9b) occurred during this stage of creep. Pore water pressure increased during the undrained creep (Fig. 6.9c), due to which the effective stress path (Fig. 6.9a) moved towards the failure envelope (determined from CIU tests) and became stable at point 'a' after 4000 minutes of creep. The increase in axial strain was gradual (Fig. 6.9d) and the axial strain rate (Fig. 6.9e) reduced to a very small value at point 'a'.

Thereafter, the deviator stress level was increased to reach point 2 with q/p' of 0.912 (stage 2), which was on the failure envelope. It may be noted from Fig. 6.9a that the effective stress path moved away from the failure envelope during creep in this stage. It was due to the reduction in pore water pressure in this stage as seen between points 2 and b in Fig. 6.9c. This is in contrast with the observations for undisturbed marine clay, in which the overall pore water pressure was always found to increase in any stage of creep. Very small axial strain occurred in stage 2 (about 0.4% from 2 to b in Fig. 6.9b) and axial strain increased very gradually (Fig. 6.9d). The axial strain rate (Fig. 6.9e) in stage 2 again decreased to a small value although the value was greater than the axial strain rate at the end of stage 1.

On increasing the deviator stress level to a point 3 with q/p' of 0.923 (stage 3), which again lay on the failure envelope (Fig. 6.9a), there was no significant movement of stress path and the specimen remained stable. There was a slight decrease in pore water pressure in this stage (Fig. 6.9c, point 3 to c). The axial strain rate again reached a small value at point c (Fig. 6.9e).

When the deviator stress level was again increased to reach a point 4 with q/p' of 0.971 (stage 4), which was slightly above the failure envelope, instability occurred within 300 minutes of creep at point I (Fig. 6.9d) and was identified by the rapid increase in axial strain (Fig. 6.9d) and axial strain rate (Fig. 6.9e). The effective stress path moved towards the failure envelope from its left side (Fig. 6.9d) and failure, identified by shear band formation, occurred at the failure envelope (point F).

The stress-strain curve (Fig. 6.9b) indicates that most of the axial strain took place in the first stage, which resulted in drop in deviator stress. Very small additional axial strain took place in stages 2 and 3. In stage 4, onset of instability occurred and it was accompanied by rapid increase in axial strain. Shear band occurred at about 12% axial strain. It appears that the specimen became stronger after creep in stage 1.

Pore water pressure variation (Fig. 6.9c) indicates that most of the pore water pressure developed at the start of creep in the first stage and became almost constant at the end of creep albeit some fluctuations. In stages 2 and 3, the pore water pressure reduced slightly. In stage 4, the pore water pressure dropped abruptly after the onset of instability. The pore water pressure after instability varied in inconsistent way as shown in Fig. 6.9c using pore pressure difference between bottom and mid pore pressure. It was a result of non-uniform deformation due to high strain rate.

Fig. 6.9a: Stress path for test RCR1 showing no pre-failure instability

Fig. 6.9b: Stress-strain curve for test RCR1 compared with CIU test

Fig. 6.9c: Variation of mid pore pressure with time in test RCR1

Fig. 6.9d: Variation of axial strain with time in for test RCR1

Fig. 6.9e: Variation of axial strain rate with time in test RCR1

6.7.2 Test RCR2

Results of test RCR2 are shown in Figs. 6.10a to 6.10e. There were 8 creep stages in test RCR2 (Table 6.2). The start of stages 1 to 8 is shown by numbers 1 to 8 in Figs. 6.10a to 6.10e. Fig. 6.10a shows that the effective stress path in stage 1 recorded the maximum movement towards the failure envelope. The effective stress path was found to stabilize after some duration of creep in stages 1 to 7. Stress states were away from the failure envelope at the end of stages 1 to 5 and specimen remained stable. The stress-state was near the failure envelope at the end of stage 6; however, the specimen remained stable. At the end of stage 7, the stress-state reached slightly beyond the failure envelope (point g) and increasing pore water pressure (Fig. 6.10c) and axial strain rate (Fig. 6.10e) were noticed. When deviator stress level was again increased to a point 8 with stress ratio of 0.980, instability occurred, indicated by rapid increase in axial strain (Fig. 6.10d) and axial strain rate (Fig. 6.10e). The instability occurred immediately after the increase in deviator stress level. It was followed by failure, indicated by shear band formation. This shows that the instability occurred at or slightly beyond the failure envelope. The possible reasons for the apparent occurrence of instability after the failure envelope are explained in section 6.7.3.

The stress-strain curve shown in Fig. 6.10b, when compared with the stress-strain curve in test RCR1 (Fig. 6.9b), indicates that lower axial strain occurs at low deviator stress levels (RCR2). There was only about 1% axial strain in each of the stages 1 to 5 despite nearly the same duration of creep of about 3000 mins in each stage. Larger axial strain occurred only in stages 6 and 7 when deviator stress level was increased significantly. It may be noted that the maximum deviator stress attained in test RCR2 was higher than that attained in test RCU7 and RCR1. This indicates that the specimen might have become stronger with longer duration of creep.

The variations of axial strain and axial strain rate with time, as shown in Figs. 6.10d and 6.10e, were similar to those earlier observed in test RCR1 (see Figs. 6.9d and 6.9e). Pore water pressure variation in test RCR2, as shown in Fig. 6.10c, was also quite similar to that in test RCR1 (see Fig. 6.9c). In general, net pore water pressure

increased with the increase in the duration of creep but the increase was very gradual. In stage 8, in which instability occurred, the pore water pressure increased rapidly. However, an abrupt drop in pore water pressure was recorded after the onset of instability, indicating the development of non-uniformity of pore water pressure after the instability. It is also indicated by the rapid increase in difference between bottom and mid pore pressure as shown in Fig. 6.10c. This implies that the determined effective stresses after the onset of instability will be erroneous.

Fig. 6.10a: Stress path of test RCR2, showing no pre-failure instability

Fig. 6.10b: Stress-strain curve for test RCR2

Fig. 6.10c: Variation of mid pore pressure with time in test RCR2

Fig. 6.10d: Variation of axial strain with time in test RCR2

Fig. 6.10e: Variation of axial strain rate with time in test RCR2

6.7.3 Comparison of Results from Tests RCR1 and RCR2

The above results from tests RCR1 and RCR2 indicate that instability also occurred in reconstituted marine clay similar to the undisturbed clay. Fig. 6.11a shows the effective stress states at instability, as determined using mid-height pore pressures, along with the effective stress states (at the end of creep stages) in which instability had not occurred. The effective stress states at failure from CIU tests are also shown for comparison. It shows that the instability apparently occurred at a stress-state slightly after the failure envelope in both the undrained creep tests. However, Fig. 6.11b, which shows the effective stress states at instability and failure determined using bottom pore pressure, shows that all the states lie within the failure envelope determined from bottom pore water pressure. The large difference between effective stress states determined from bottom and mid pore pressures reflects the significant non-uniformity of pore pressure that occurs in the undrained creep tests at the onset of instability (also see Figs. 6.9c and 6.10c). Figs. 6.11a and 6.11b also show that the onset of instability and failure in undrained compression tests on marine clay is governed by a band of failure, as indicated in Fig. 6.11b. It is difficult to accurately determine the governing effective stress state at instability and failure. Since the effective stress states at the onset of instability are very close to the failure envelope, the zone of potential instability (similar to that defined for undisturbed marine clay) may be very small and may not be useful to define for reconstituted marine clay. For practical use, it would be conservative to choose the failure envelope as conventionally defined.

Fig. 6.11a: Effective stress states (determined from MID-HEIGHT pore pressure) at instability in tests RCR1 and RCR2

Fig. 6.11b: Effective stress states (determined from BOTTOM pore pressure) at instability in tests RCR1 and RCR2

6.8 COMPARISON OF THE BEHAVIOURS OF RECONSTITUTED AND UNDISTURBED MARINE CLAYS

The behaviour of reconstituted marine clay can be compared with that of undisturbed marine clay as follows.

6.8.1 Comparison of Undrained Compression Behaviours

General Stress-Strain Behaviour

The undrained compression behaviours of reconstituted and undisturbed marine clays are compared in Figs. 6.12a to 6.12e. For both clays, the results of the load-controlled and deformation-controlled undrained compression tests conducted at slowest and comparable shearing rates are plotted together. Other tests also show similar trends. In this section, the general stress-strain behaviour will be compared before the comparison of rate and mode effects on the behaviour of the two clays is made.

Stress-strain curves, as shown in Fig. 6.12a, show that the stress-strain behaviours of undisturbed and reconstituted clays are quite different. Firstly, the peak deviator stress is higher in undisturbed clay, indicating that the undrained shear strength is lower for reconstituted marine clay. This is a well known phenomena exhibited by most of the cohesive soils (Holtz and Kovacs, 1981; Parry and Wroth, 1981; Atkinson, 1993) and is caused by the destructuration effect. Secondly, while the peak deviator stress occurs at a small axial strain of 1.5 to 3% followed by strain softening in the undisturbed clay (Fig. 6.12a), reconstituted clay shows an almost constant peak deviator stress over a large axial strain rate. Strain softening is small in reconstituted clay and shear band develops before the specimen can soften to any significant degree.

It can be seen from Fig. 6.12b that the pore water pressure generated during shearing is higher in undisturbed clay than that in reconstituted clay. However, the manner in which pore pressure changes is similar, i.e. pore pressure continuously rises during shearing and rate of development of pore pressure continuously decreases. Pore pressure finally tends to reach a constant value, although it is not that obvious in case of undisturbed marine clay. (Apparently constant pore pressure towards the end of

load-controlled test, CIU4, on undisturbed marine clay is due to onset of instability and short duration of deformation after peak, which does not allow pore pressure to develop.) Larger pore water pressure is generated due to the compression of soil skeleton as well as breaking of bonds (structure) within the soil skeleton in undisturbed clay. In reconstituted clay, it seems that pore pressure due to the compression of soil skeleton reaches a limiting value. It also shows the potential for higher pore pressure generation during creep due to the similar mechanism in undisturbed clay. Higher undrained shear strength of undisturbed clay, despite higher pore water pressure, also points to the fact that the structure of soil (fabric and bonding) contributes significantly towards the shear strength of the soil. From the comparison of pore pressure variation in undisturbed and reconstituted marine clay, it can be said that the study of reconstituted clay may not give true indication of the behaviour of undisturbed clay in similar testing conditions.

From the comparison of the effective stress paths in Fig. 6.12c, it appears that the failure envelope is slightly higher for undisturbed marine clay than the failure envelope for reconstituted clay. However, the difference is very small. It shows that the failure envelope was not affected significantly by the remoulding of undisturbed clay.

Fig. 6.12a: Comparison of stress-strain curves in undrained tests on undisturbed and reconstituted clay

Fig. 6.12b: Comparison of pore water pressure variation in undrained tests on undisturbed and reconstituted clay

Fig. 6.12c: Comparison of stress-paths in undrained tests on undisturbed and reconstituted marine clay

Effect of Loading Rate and Loading Mode

Effect of loading rate and loading mode on the behaviour of undisturbed and reconstituted marine clay is illustrated well by Figure 6.12d, which plots the variation of peak deviator stresses versus loading rates in load-controlled and deformation-controlled tests for the undisturbed and reconstituted marine clays. The three lines in Fig. 6.12d seem to be parallel, which indicates that the proportion of increase in peak deviator stress with increase in shearing rate is identical in the two types of clays. It was observed that the increase in peak deviator stress was about 6% per log cycle increase in axial strain rate for undisturbed marine clay, while it was 5% for reconstituted marine clay. Thus the percentage increase in strength is similar in both clays. However, undisturbed marine clay has higher undrained shear strength despite higher water content (54%) than the water content of reconstituted marine clay (47%). This indicates that the natural structure of the undisturbed marine clay could be contributing significantly to the shear strength.

It is notable from Fig. 6.12d that while load-controlled and deformation-controlled tests conducted at comparable rates reach similar peak deviator stresses for undisturbed soil, load-controlled tests show much higher strength than corresponding deformation-controlled tests for reconstituted clay. One explanation for this could be as follows. It was mentioned in Section 6.5 that the higher strength in load-controlled undrained tests on reconstituted marine clay is caused by the higher strain rate at peak q in load-controlled test, which reduces the pore pressure generation, increases the mean effective stress and deviator stress. For undisturbed clay, while the high strain rate near peak q in load-controlled test tries to increase the deviator stress, the tendency of undisturbed clay specimen to soften due to the breaking of the structure after peak q tries to reduce the deviator stress, i.e. tries to soften. The softening tendency takes precedence and does not allow the deviator stress to increase after peak in load-controlled tests on undisturbed marine clay. Also, it can be observed that the pore water pressures (see Fig. 6.12b) up to the peak deviator stress are similar in load-controlled and deformation-controlled tests on undisturbed clay. On the other hand, for reconstituted clay (Fig. 6.12b) the pore pressure is higher in deformation-controlled test than the pore pressure in load-controlled test. Correspondingly, the shear strength is lower in deformation-controlled test.

From the above comparison of the behaviours of reconstituted and undisturbed marine clay, it appears that the natural structure of undisturbed marine clay affects the stress-strain behaviour of the clay and the destructured clay has lower shear strength. However, the effect of destructuration on the ‘percentage change in shear strength due to the increase in shearing rate’ is small. This shows that the change in the pore pressure caused by the rate-effect is the primary cause of the rate-effect on the undrained shear strength. The change in pore pressure due to the change in strain rate could be related to the permeability and ease of equalization of pore pressures within the soil. It implies that a clayey soil with high permeability, in which quick pore pressure equalization is possible, may show little or no rate-effects. This is examined in Chapter 7 using the undrained compression and extension tests on Kaolin, which is highly permeable.

Fig. 6.12d: Comparison of variation of peak deviator stress with axial strain rate in undrained tests on undisturbed and reconstituted clay

6.8.2 Comparison of Drained Compression Behaviours

The behaviours of the reconstituted marine clay and undisturbed marine clay in drained compression tests are compared in Figs. 6.13a to 6.13c using the slowest tests (CID1 and RCD1), both conducted at the same axial strain rate of 0.002 mm/min and on the specimens consolidated under the same effective confining stress of 200 kPa.

It can be seen from Fig. 6.13a that the undisturbed marine clay (test CID1) showed a much stiffer behaviour than the reconstituted clay at the beginning of shearing. However, after the plastic yielding at point Y, the stress-strain curve turned and followed the same q - ε_1 curve as that followed by reconstituted marine clay (RCD1). Thus, after yielding, undisturbed soil starts to behave like reconstituted soil due to the destructuration of soil skeleton. This can be further understood using the plot of logarithm of vertical effective stress (σ_1') vs. axial strain as shown in Fig. 6.13b. It shows a sharp yielding point after which similar $\log \sigma_v' - \varepsilon_1$ curves are followed by the two soils.

It can also be seen from Fig. 6.13a that both the drained tests reached almost the same peak deviator stress. This again shows that after the plastic yielding, undisturbed soil starts to behave like reconstituted soil due to the destruction of the structure of soil during drained shearing. Hence, both the soils will reach the same failure envelope.

Fig. 6.13c shows the variation of volumetric strain in the two drained tests. It can be seen that the volumetric strain is much smaller in reconstituted marine clay than that in undisturbed marine clay. This is due to the fact that the undisturbed soil is more compressible than the reconstituted soil and has higher initial water content (54.5%) than the reconstituted clay (47%). The final water contents at failure were similar in the drained tests (42% in undisturbed specimen and 40.5% reconstituted clay).

Fig. 6.13a: Comparison of stress-strain curves in drained tests on undisturbed and reconstituted marine clay

Fig. 6.13b: Comparison of stress-strain curves in drained tests on undisturbed and reconstituted marine clay

Fig. 6.13c: Comparison of volumetric strains in drained tests on undisturbed and reconstituted marine clay

6.8.3 Comparison of Undrained Creep Behaviours

Typical undrained creep behaviours of undisturbed and reconstituted marine clay are compared in Figs. 6.14a to 6.14e. A comparison of effective stress states at the end of creep stages in which instability did not occur is shown in Fig. 6.14a for both undisturbed and reconstituted clays. The effective stress states at the onset of instability in undisturbed and reconstituted marine clays are shown in Fig. 6.14b. The failure envelope and potential instability line for undisturbed marine clay is also shown. For undisturbed clay, Fig. 6.14a shows that almost all (except one) of the stable effective stress states lay outside the zone of potential instability while Fig. 6.14b shows that the points at the onset of instability were spread across the zone of potential instability. This implies that instability occurred when the effective stress state reached into the zone of potential instability. For reconstituted clay, Fig. 6.14a shows that there were many stable effective stress states inside the zone of potential instability while Fig. 6.14b shows that the instability in the two undrained creep tests occurred slightly after the failure envelope. These observations from 6.14a and 6.14b

show that the reconstituted marine clay is less susceptible to pre-failure instability than undisturbed marine clay. This indicates that the zone of potential instability does not exist or could be very small for reconstituted Singapore marine clay. The existence of a well defined zone of potential instability for the undisturbed marine clay and a possible non-existence of the zone for reconstituted marine clay indicates that pre-failure strain softening before failure may be necessary for the zone of potential instability to exist and hence the pre-failure instability to occur. For the undisturbed marine clay, for which significant pre-failure strain softening occurs after the peak deviator stress and before the shear band develops, pre-failure instability is significant. On the other hand; the reconstituted marine clay, for which the pre-failure strain softening before the occurrence of shear band is small (see section 6.3.1), does not show any significant pre-failure instability.

Typical pore water pressure variations in undrained creep tests on reconstituted and undisturbed marine clays are shown in Fig. 6.14c. Fig. 6.14c shows that the pore water pressure increases with increasing creep duration in both types of clays. However, there is larger fluctuation in pore pressure in reconstituted marine clay, which could be because of the lower permeability of reconstituted clay, due to which the equalization of pore pressure is more difficult than in undisturbed soil. Fig. 6.14c also shows that the magnitudes of pore pressures are similar in the tests on undisturbed marine clay and reconstituted marine clay. This is in contrast with the observation that the pore pressure was lesser in reconstituted clay during CIU tests (see Fig. 6.12b). This shows that the undrained creep has larger effect on pore pressure for reconstituted marine clay.

The stress-strain curves in undisturbed and reconstituted marine clay are compared in Fig. 6.14d. It shows that the deviator stress at failure in reconstituted clay was higher than the deviator stress at failure in undisturbed marine clay. This indicates that the strength of reconstituted marine clay specimen increased due to the creep. This indicates that under the sustained loading, thixotropic hardening may be taking place in reconstituted marine clay.

The variation of axial strain (Fig. 6.14e) and axial strain rate (6.14f) are also quite similar in the two types of clays with the exception that there was larger fluctuation in axial strain rate and the creep strain rate is higher in reconstituted marine clay. This could again be due to the larger fluctuation in mean effective stress in reconstituted clay (due to the fluctuation in pore pressure). One example of this is shown by encircled points in Figs. 6.14c and 6.14e. When pore pressure increases (Fig. 6.14c), the mean effective stress decreases, due to which the load tends to decrease. To keep the load constant, strain rate (6.14e) increases.

Fig. 6.14a: Comparison of stable effective stress states during unundrained creep tests on undisturbed and reconstituted marine clay

Fig. 6.14b: Comparison of effective stress states at the onset of instability in undisturbed and reconstituted marine clay

Fig. 6.14c: Comparison of pore pressure variation in undrained creep tests on undisturbed and reconstituted marine clay

Fig. 6.14d: Comparison of pore pressure variation in undrained creep tests on undisturbed and reconstituted marine clay

Fig. 6.14e: Comparison of variation of axial strain in undrained creep tests on undisturbed and reconstituted marine clay

Fig. 6.14f: Comparison of variations of axial strain rate in undrained creep tests on undisturbed and reconstituted marine clay

6.9 DISCUSSIONS

Results of deformation-controlled and load-controlled undrained and drained tests conducted at different shearing rates on reconstituted marine clay were presented in this chapter. Results of the undrained creep tests were also presented. A comparison of the behaviours of reconstituted and undisturbed marine clays was also made.

6.9.1 Relationship between the Rate-Effect and Pore Pressure Variation in Undrained Tests

It was observed that the peak deviator stress and hence the undrained shear strength increased with the increase in shearing rate in both load-controlled and deformation-controlled tests. There was a linear increase in peak deviator stress with logarithmic increase in shearing rate as shown earlier in Fig. 6.12d. The peak deviator stress increased by an average of 5% per log cycle increase in shearing rate. This can be explained based on the observation that the pore water pressure decreased with the increase in shearing rate in both deformation-controlled (see Fig. 6.2c) and load-

controlled tests (see Fig. 6.3c). Since the pore pressure reduces with the increase in shearing rate and thus leads to an increase in mean effective stress, shearing at different rates is equivalent to the shearing from different initial effective mean effective stresses. Higher mean effective stress leads to the higher peak deviator stress. However, the failure envelope of the soil is more or less unaffected. Therefore, the effective stress states at failure in tests conducted at faster rates reach the failure envelope at a higher point (see Fig. 6.4g). The increase in pore water pressure, and thus the decrease in mean effective stress in slower tests, could be related to the creep induced pore water pressure. In slow tests, the soil skeleton also creeps progressively during the shearing process and larger pore water pressure is developed. This fact can be further supported by the observed pore water pressure increase at constant load during undrained creep tests (see Fig. 6.14c).

The rate-effects observed in relation to the undrained compression of clay had long been related to the pore water pressure differences (Richardson and Whitman, 1963; Zhu et al., 1999). However, it was suspected by some authors (Lefebvre and LeBoeuf, 1987) that the apparent decrease in pore water pressure due to the increase in shearing could be due to the improper measurement of pore water pressure. Many studies on rate-effect did not discuss the variation of pore water pressure (Casagrande and Wilson, 1951; Vaid and Campanella, 1977; Graham et al., 1983; Shibuya et al., 1995; Lo Presti et al., 1999) or based their conclusions on pore water pressure measurement from only one end or middle and one end only (Richardson and Whitman, 1963; Akai et al., 1975; Lefebvre and LeBoeuf, 1987; Sheahan et al., 1996; Teachavorasinskun et al., 2002). The study presented in this thesis used pore water pressures measured at the top, bottom and middle of the specimen and hence obtained a better estimate of the pore water pressure variation with shearing rate. It has been confirmed from undrained tests on both undisturbed and reconstituted marine clays that the inverse relationship between shearing rate and pore water pressure exists.

6.9.2 Loading Mode Effect vs. Loading Rate Effect

When load-controlled tests and deformation-controlled tests were conducted in such a way that the time taken to reach the peak deviator stress would remain the same in

both types of tests, the load-controlled tests resulted in up to 8% higher peak deviator stress than corresponding deformation-controlled tests (see Fig. 6.4c). This could again be related to the rate effect on the pore water pressure. The pore water pressure in load-controlled tests was lower than the pore water pressure in deformation-controlled tests, which was caused by the higher axial strain rate in load-controlled tests. Higher strain rate did not allow the creep related pore water pressure to develop fully and led to higher mean effective stress, which caused the shear strength to increase in such a way that the effective stress state at failure was on the failure envelope (see Fig. 6.4g). This shows that the effect of loading mode, similar to the rate-effect, is a result of the changes in pore pressure due to the change in rate and the uniqueness of the failure envelope of the soil in effective stress space.

6.9.3 Relationship between the Loading Rate and Loading Mode Effects and Pore Pressure in Drained Compression

The effect of loading rate and loading mode on the drained behaviour was also studied. It was observed that the reconstituted marine clay in isotropically consolidated drained tests is affected by the loading rate and loading mode. The peak deviator stress in drained test was found to reduce by 10% to 15% due to only 5-fold increase in strain rate (see Figs. 6.6a and 6.6b) and it also led to a lower failure envelope. It is because of higher undissipated pore water pressure (see Fig. 6.6e), or in other words, due to the higher non-uniformity of pore water pressure in the CID tests conducted at faster rates of shearing due to insufficient time for drainage. Incomplete drainage with increase in shearing rate was also reflected as reduced magnitude of volumetric strain with increase in shearing rate. Therefore results obtained from fast drained compression tests are not reliable. Reduction in drained shear strength of clay with decrease in time to failure has been earlier reported by Gibson and Henkel (1954). It was observed that when effective stress conditions are measured using the undissipated pore pressure at the mid-height of specimen, the failure was found to occur at the failure envelope determined from CIU tests. This shows that the failure in drained test also is governed by the effective failure envelope determined from properly conducted CIU tests.

It was also observed that currently accepted method of selecting the theoretical time to failure in a drained test using the isotropic consolidation data gives good estimate of proper strain rate. However, some pore pressure always remains undissipated despite using a time to failure higher than the theoretical. This causes slight reduction in estimated drained shear strength and predicts lower failure envelope. It was observed that if the undissipated pore pressure measured at the mid-height is taken into account, a better estimate of failure envelope can be obtained.

When the CID tests were conducted at suitably low rates, the differences between corresponding load-controlled and deformation-controlled CID tests were small and both the drained shear strength and failure envelope remain largely unaffected by the loading mode (see Fig. 6.7a and 6.7b). This observation has not been reported earlier. It is useful to know that the drained shear strength and friction angles are not affected by the loading mode, since most of the field loading conditions are load-controlled type, while laboratory testing is done using deformation-controlled test.

The drained behaviour is affected by the loading mode when the tests are conducted at faster rates and a lower drained shear strength and failure envelope is obtained in load-controlled test. Similar observation was earlier reported by Lundgren et al. (1968) for CID tests on laboratory compacted kaolin, in which drained shear strength was found to be lower in stress controlled tests. However, such tests are not the representative of true drained behaviour. The stress path determined using excess pore water pressure was found to reach very close to the failure envelope determined from undrained test. This shows that the failure envelope determined from properly conducted drained and undrained tests governs the failure even in apparently improperly conducted tests. However, measurement of mid-height pore pressure is essential for better estimation of failure envelope.

It is evident that the drained shear strength and effective friction angle will be underestimated if higher rates of shearing are used in CID tests. This may seem to be more conservative for design purpose. However, the strength and friction angles obtained from an unreasonably high strain rate test will lead to over conservative design and corresponding increase in cost.

6.9.4 Undrained Creep Behaviour

The undrained creep behaviour of reconstituted marine clay was slightly different from the undrained creep behaviour of undisturbed marine clay. Although the pore pressure was found to increase with increase in time for creep, similar to the undisturbed soil, the instability occurred on failure in reconstituted marine clay. This is different from the observation of pre-failure instability in undisturbed clay, for which instability line (a line passing through the peaks of effective stress paths of CIU tests) and a zone of potential instability (bound by instability line and failure envelope) can be defined. Since strain softening is very small and the envelope of peaks is very close to the failure envelope, non-existence of pre-failure instability in reconstituted clay shows that the strain-softening is a necessary condition for the zone of potential instability to be significant and hence the pre-failure instability to occur.

6.10 CONCLUSIONS

Results from CIU tests conducted on reconstituted marine clay specimens under different loading rates and loading modes were presented in this chapter. Following conclusions can be made from these results.

1. The peak deviator stress and hence the undrained shear strength of reconstituted marine clay increases with the increase in shearing rate in both load-controlled and deformation-controlled test. The increase in peak deviator stress is linear with the logarithmic increase in shearing rate in both load and deformation-controlled tests. An increase of about 5% in peak deviator stress occurs per log cycle increase in shearing rate. Similar observation (6% increase in peak deviator stress per log cycle increase in strain rate) was earlier made for undisturbed soil. It shows that the remoulding may not affect the magnitude of rate-effect significantly. However, the difference in the water contents of reconstituted and undisturbed marine clays could also have played a significant role.

2. The increase in shear strength due to the increase in shearing rate is consistent with the failure in terms of effective stresses. Pore water pressure reduces with the increase in shearing rate, thus increasing the mean effective stress. Correspondingly, the deviator stress increases so that the failure occurs at a higher point on the failure envelope.
3. The pore pressure variation during undrained shearing of reconstituted marine clay was found to be significantly affected by the shearing rate. The magnitude of pore pressure decreased with the increase in shearing rate. Considering the fact that the pore pressure increases at constant load during undrained creep tests, it can be concluded that the higher pore pressure in slower tests is caused by simultaneous undrained creep during shearing together with the pore pressure developed due to the shearing of the specimen.
4. The undrained shear strength in load-controlled tests was found to be up to 8% higher than the undrained shear strength in deformation-controlled tests for reconstituted marine clay, even when the time taken to reach the peak deviator stress was the same. Correspondingly, the pore pressure was found to be lesser in load-controlled test. This observation is different from the observation for undisturbed clay, in which loading mode did not affect the undrained shear strength. The behaviour of reconstituted marine clay could be different because the strain rate in load-controlled test becomes higher than the strain rate in deformation-controlled test much before the peak deviator stress, which causes reduced time for any creep induced pore pressure to develop and hence a reduction in overall pore pressure. The reduction in pore pressure causes an increase in mean effective stress and corresponding increase in deviator stress, so that the failure envelope remains the same. Thus the mode effect is consistent with the failure in terms of effective stresses. In the case of undisturbed marine clay, the pore pressures in comparable load controlled and deformation-controlled tests were similar and hence the shear strengths were also similar in the two loading modes. It appears that the tendency of the specimen for strain softening after the peak

deviator stress negates the effect of increased strain rate in load-controlled tests on undisturbed marine clay.

5. The drained shear strength and effective friction angle are affected by the shearing rate when tests are conducted at fast shearing rate (faster than that determined using the method given in BS1377). This is due to the non-uniformity of pore pressure within the specimen, caused by incomplete drainage at fast shearing rates. Thus, the drained tests conducted at fast tests are not the representative of true drained behaviour.
6. The drained tests conducted at proper shearing rate, as determined using isotropic consolidation data, give fairly accurate estimate of the drained shear strength and friction angle, although there is slight underprediction of time to failure. Some pore pressure always remains undissipated and a more accurate determination of effective friction angle can only be made if the pore pressure is measured at the mid-height of the specimen.
7. Drained shear strength and friction angle are not affected by loading mode if tests are conducted sufficiently slowly (at the same or slower rates than that determined according to BS1377). This could not be verified from tests on undisturbed clay because only one drained test was conducted at proper rate on that clay. What happens in drained tests conducted at rates much slower rates than that determined using BS1377, needs to be verified.
8. The drained shear strength and friction angle are similar for reconstituted and undisturbed marine clay (despite higher water content of undisturbed marine clay), if obtained from properly conducted drained tests at adequately slow shearing rates. It was observed that the volume change during drained shearing was much lower for reconstituted marine clay compared to the undisturbed clay. Thus the water contents at peak deviator stress in the two clays were similar.
9. Undrained creep behaviour of reconstituted marine clay is different from the behaviour of undisturbed marine clay. While pre-failure instability may occur

in undisturbed marine clay, the instability was found to occur on failure for reconstituted marine clay. Zone of potential instability is very small or may not exist at all for reconstituted marine clay. Comparison of the undrained creep behaviours of reconstituted and undisturbed marine clays indicated that the pre-failure strain-softening could be a necessary condition for a significant zone of potential instability to be defined and hence the pre-failure instability to occur.

Finally, it may be mentioned that the effects of loading rate and loading mode appear to be related to the changes in pore water pressure for both undisturbed and reconstituted marine clay. Both of these clays have low permeability (approximately 7×10^{-9} and 4×10^{-9} m/s respectively for undisturbed and reconstituted marine clay), which appears to be one of the main factors causing pore pressure differences. It will be useful to examine the effect of loading rate and loading mode on more permeable clay to determine the extent to which pore pressure equalization governs the rate and mode effects. For this reason, a similar study was conducted on Kaolin, which has a significantly higher permeability (2×10^{-5} m/s) and is presented in the next Chapter.

CHAPTER 7

STRESS-STRAIN BEHAVIOUR OF KAOLIN UNDER DIFFERENT LOADING RATES AND LOADING MODES

7.1 INTRODUCTION

The results presented in Chapters 4 and 6 showed that the stress-strain behaviours of undisturbed and reconstituted Singapore marine clay were affected by the loading rate and loading mode. While structure of soil appeared to play a role in this behaviour, the change in pore water pressure response of the soil upon the change in loading rate and loading mode appeared to be the primary cause of the rate and mode effects, since even the destructured soil (reconstituted clay) was affected significantly by the loading rate and loading mode. Since both undisturbed and reconstituted marine clays have low permeability ($k = 7 \times 10^{-9}$ and 4×10^{-9} m/s respectively), pore pressure equalization does not take place easily and that may be the cause for the changes in pore pressure with varying rates. Whether the pore pressure differences and hence the rate effect is caused mainly by the permeability of the soil can be verified by conducting the tests on a clay with comparatively higher permeability. To achieve this objective, triaxial tests were conducted on reconstituted Kaolin specimens ($k = 2 \times 10^{-5}$ m/s) under various loading rates and loading modes. In addition to the compression tests under different loading modes and loading rates, some extension tests were also conducted at varying strain rates. All the tests were conducted on normally consolidated Kaolin specimens. The tests conducted included isotropically consolidated undrained (CIU or CU) tests under different loading rates

and loading modes, isotropically consolidated drained (CID or CD) tests to establish the failure envelope of the soil and few constant stress drained (CSD) tests. Some deformation-controlled extension tests were also conducted to study the effect of deformation rate on the undrained behaviour of Kaolin in extension.

It is important to point out that Kaolin is often used to study the typical behaviour of clay (Lundgren et al., 1968; Fourie and Xiaobi, 1991; Shibuya et al., 1995; Penumadu and Chameau, 1997; Newson et al., 1997; Kurukulasuriya et al., 1999; Matesic and Vucetic, 2003; Prashant and Penumadu, 2005; Silva et al., 2006). However, the studies of rate effect and mode effect on the behaviour of Kaolin in CIU tests have been limited. Lundgren et al. (1968) showed that the peak deviator stress in stress-controlled UU and CID tests was lower than the peak deviator stress in strain controlled tests. Fourie and Xiaobi (1991) did not observe any significant effect of strain rate on the stress-strain curves in UU tests on Kaolin. Shibuya et al. (1995) and Matesic and Vucetic (2003) reported increase in secant modulus of Kaolin with the increase in strain rate while Penumadu and Chameau (1997) reported an increase in shear modulus and shear strength with the increase in radial strain rate in Pressuremeter tests. Thus, the effect of strain rate on the strength and modulus of Kaolin has been reported by some authors. However, a study of the loading mode effects in CIU tests on Kaolin has not been conducted.

This chapter will present the results of the study of loading rate and loading mode effects on the behaviour of commercial Kaolin clay, which had a plastic limit of 33.5%, liquid limit of 74% and specific gravity of 2.55. A summary of all the tests is presented in Table 7.1.

7.2 UNDRAINED COMPRESSION BEHAVIOUR OF KAOLIN

Isotropically consolidated undrained (CIU) tests were conducted on specimens consolidated under effective confining pressure of 300 kPa. Four deformation-controlled tests, KCU1 (0.01 mm/min), KCU2 (0.05 mm/min), KCU3 (0.1 mm/min) and KCU4 (1 mm/min) and two load-controlled tests KCU5 (0.3 N/min) and KCU6 (3 N/min) were conducted to study the effect of loading rate and loading mode on the

behaviour of Kaolin. Failure envelope for the soil is established in section 7.2.1. Results of the deformation-controlled tests are presented in section 7.2.2. A comparison of deformation-controlled and load-controlled tests is done in section 7.2.3.

Table 7.1: Summary of compression and extension tests on Kaolin

Test	rate	Units	w% pre test	w% post test	p_0	q_m	$(q/p')_{\max}$ top	$(q/p')_{\max}$ mid	ϕ' at peak* & failure [^]
KCU1	0.01	mm/min	58.4	50.6	300	182.4	1.130	1.134	28.3
KCU2	0.05	mm/min	59.3	52.2	300	187.5	1.073	1.079	26.9
KCU3	0.1	mm/min	59.3	51.4	300	179.6	1.121	1.120	27.5
KCU4	1	mm/min	58.9	51.2	300	188.1	1.095	1.090	27.2
KCU5	0.3	N/min	58.8	51.9	300	196.8	1.051	1.043	26.4
KCU6	3	N/min	59.2	51.8	300	191.8	1.047	1.05	26.5
KCU7	0.1	mm/min	57.9	53.2	210	131.3	1.015	1.011	25.5
KCU9	3	N/min	58.1	49.2	400	232.2	1.011	1.019	25.7
KCU10	1	mm/min	57.8	49.5	400	228	1.044	1.039	25.8
KACD1	0.04	mm/min	56.2	46.9	230	285	0.995	0.989	25.3
KCD1	0.05	mm/min	58.7	49.1	160	238.9	0.986	0.986	25.1
KCD2	0.05	mm/min	59.9	45.9	225	339.4	1.006	1.012	25.4
KCD3	0.05	mm/min	59.8	45.3	300	382.6	0.893	0.898	24.9
KCSD1	0.05	mm/min	58.6	48.5	400	-	1.064	1.130	-
KCSD2	0.05	mm/min	58.9	49.5	400	-	1.102	1.051	-
KUE1	0.01	mm/min	58.5	50.1	400	-234	-0.837	-0.838	-21.5
KUE2	0.1	mm/min	58.9	49.6	400	-229	-0.852	-0.845	-21.9
KUE3	1	mm/min	59.2	50.7	400	-235	-0.876	-0.859	-22.5
KDE1	0.03	mm/min	59.1	50.2	400	-260	-0.827	-0.824	-21.3
KCRE1	0.1	mm/min	59.5	52.2	400	-	-0.826	-0.826	-
KCSDE1	0.1	mm/min	59.3	48.7	400	-	-0.867	-0.867	-
KCSDE2	0.1	mm/min	58.9	48.5	400	-	-0.860	-0.893	-

w%: water content; q_m : peak deviator stress; q/p' max: maximum stress ratio

* ϕ' at peak is the effective friction angle at maximum $(\sigma_1' - \sigma_3')$ (or maximum q)

[^] ϕ' at failure is the effective friction angle at maximum σ_1'/σ_3' (or maximum q/p')

7.2.1 Failure Envelope for Tested Kaolin

Failure envelope, as shown in Fig. 7.1a, was determined using three undrained tests and four drained tests. The three isotropically consolidated undrained (CIU) tests,

KCU7, KCU1 and KCU9, were conducted on specimens consolidated under mean effective stresses of 210, 300 and 400 kPa respectively. Three isotropically consolidated drained tests, KCD1, KCD2 and KCD3 were conducted on Kaolin specimens consolidated under mean effective stresses of 160, 225 and 300 kPa respectively. One anisotropically consolidated drained test, KACD1, was also conducted on a specimen anisotropically consolidated up to 230 kPa mean effective stress. The effective stress paths in these drained and undrained tests, as shown in Fig. 7.1a, show that the failure envelope is curved at higher mean effective stresses. The failure envelope seems to be the same irrespective of drainage conditions. The shape of envelope at smaller mean effective stresses was not determined. This does not affect the interpretation of results of the tests in this chapter because all the tests were conducted on normally consolidated specimens with a consolidation pressure of higher than 200 kPa.

Fig. 7.1a: Comparison of effective stress paths in CIU tests with CID tests

The stress-strain curves of CIU and CID tests are plotted in Figs. 7.1b and 7.1c respectively. Drained tests reached their peak deviator stresses at much larger axial strains (greater than 20%) than that in undrained tests (8% to 10%). The peak deviator stress remained almost constant (Figs. 7.1b and 7.1c) over a large axial

strain near the peak and dropped rapidly after the shear band formation in the specimen. The shear band was accompanied by rapid drop in axial load (Fig. 7.1d) in both drained and undrained tests. The drop in axial load, and consequently a drop in q/p' , was used to identify the point of shear band formation. Strain softening before the shear band was small and most of the strain softening occurred after the shear band formation (see Figs. 7.1b and 7.1c). Volumetric strain variation with axial strain is plotted in Fig. 7.1e and it can be seen that the volumetric strain seems to become constant towards the end of the tests. The pore pressure variation in Fig. 7.1f shows that the pore pressure also becomes almost constant near the peak deviator stress.

The behaviour of Kaolin in CIU and CID tests is somewhat similar to the behaviour of reconstituted marine clay (see section 6.2 in Chapter 6). It was observed that both the clays do not show any significant strain softening before the shear band formation and the failure occurs slightly after the peak deviator stress is achieved. This is different from the behaviour of undisturbed marine clay (see section 4.2 in Chapter 4), which showed large strain softening before failure. Also, the stress-strain behaviour and pore pressure variations are similar as both Kaolin and reconstituted marine clay show almost constant deviator stress and pore water pressure over a large axial strain near the peak deviator stress.

Fig. 7.1b: Stress-strain curves in CID tests

Fig. 7.1c: Stress-strain curves in CIU tests

Fig. 7.1d: Detection of shear band from axial load vs. time curve

Fig. 7.1e: Volume change in CID tests

Fig. 7.1f: Pore water pressure variation in CIU tests

7.2.2 Effect of Shearing Rate on the Undrained Behaviour in Deformation-Controlled Tests

Stress-strain behaviour

The results of four deformation-controlled CIU tests are presented in Figures 7.2a to 7.2i. It should be mentioned that the maximum permissible strain rate calculated using volume change curves from isotropic consolidation stage (BS 1377, 1990) was 0.12 mm/min (see Appendix A).

The stress-strain curves shown in Fig. 7.2a indicate that the effect of shearing rate on the peak deviator stress is small. The peak deviator stress, in general, increased with the increase in axial strain rate, as shown in Fig. 7.2b. With the exception of test KCU3 (0.1 mm/min), which showed lowest peak deviator stress amongst the four undrained tests, the peak deviator stress, in general, increased by about 2% for every log cycle increase in axial strain rate.

It may be noted (Fig. 7.2a) that the peak deviator was reached at around the same axial strain in all the tests and q remained almost constant over a large axial strain, a typical behaviour of normally consolidated clay (Atkinson, 1993). There was only a slight drop in deviator stress (Fig. 7.2a) at larger axial strain, which was due to the increase in cross-sectional area. The shear band was found to occur in all the tests and was identified by sudden drop in load and hence the deviator stress (see Fig. 7.2a and 7.2c). This shows that the strain softening before the formation of shear band was small.

Observations made above are similar to those for deformation-controlled tests on reconstituted marine clay, as discussed in section 6.3.1. However, the percentage increase in the peak deviator stress was higher (5%) for reconstituted marine clay than that in Kaolin (2%). This indicates that the permeability of clay may be playing a major role in the observed rate effects on the behaviour of clay and less permeable soils may be affected more by rate effects.

Fig. 7.2a: Stress-strain curves for deformation-controlled CIU tests on Kaolin

Fig. 7.2b: Variation of peak deviator stress with axial strain rate in deformation-controlled tests

Fig. 7.2c: Detection of shear band from load-time curve

Variation of pore pressures

The uniformity of pore pressure within the specimen was found to be affected by the strain rate as seen in Figs. 7.2d and 7.2e, which show the difference between the top and mid pore pressures and the top and bottom pore pressures respectively. These figures show that large differences of up to 31 kPa developed between top and mid-height pore pressures at the beginning of faster tests (test KCU4 (1 mm/min)). However, the difference dropped rapidly after about 1.5% axial strain and equalized well before the peak deviator stress was reached.

Figs. 7.2d and 7.2e show that the non-uniformity of pore water pressure decreased with the decrease in shearing rate. It can be observed that, if only the top and bottom pore water pressures were measured, the difference (12 kPa from Fig. 7.2e) would not have been found to be as high as that observed between the top and mid pore water pressures (31 kPa from Fig. 7.2d). Since the maximum permissible strain rate calculated using BS 1377 (1990) was 0.12 mm/min and the maximum pore pressure difference in test KCU3 (0.10 mm/min) and slower tests is less than 10 kPa (5% of

peak q), it shows that the strain rate calculated using BS method is fairly accurate. This observation is similar to that made for reconstituted marine clay.

The variation of pore water pressure in the CIU tests is shown in Fig. 7.2f. The pore water pressure increased rapidly in the beginning of the tests but remained almost constant or increased only slightly after the peak deviator stress was attained. It can also be seen from Fig. 7.2f that lower pore water pressure developed in the tests KCU4 (1 mm/min) and KCU2 (0.05 mm/min), in which higher peak deviator stresses were obtained (see Fig. 7.2a, 7.2b and Table 7.1). Fig. 7.2g indicates that, in general, pore water pressures measured at all the three locations (top, bottom and mid-height) were lesser in tests exhibiting higher peak deviator stress. The change in undrained shear strength may be related to the change in pore water pressures generated at different rates, similar to the observation for undisturbed and reconstituted marine clays in Chapters 4 and 6. However, the results do not show a consistent trend. For example, tests KCU2 (0.05 mm/min) and KCU4 (1 mm/min) have similar pore pressures despite 20 times higher rate in test KCU4 (see Fig. 7.2f). Also, the effect of strain rate on the pore pressure and peak deviator stress is much smaller for Kaolin compared to that observed for marine clay.

Fig. 7.2d: Difference between the top and mid pore pressures in deformation-controlled CU tests

Fig. 7.2e: Difference between the top and bottom pore pressures in deformation-controlled CU tests

Fig. 7.2f: Variation of mid pore pressure with axial strain in deformation-controlled CU tests

Fig. 7.2g: Pore pressures at peak q (showing that q_m is proportional to pore pressure)

Effective stress paths and failure envelope

In Fig. 7.2h the effective stress paths for all the deformation-controlled tests discussed above are compared with the failure envelope determined in Fig. 7.1a. It indicates that the effect of the change in shearing rate on the failure envelope is small. The effective stress paths of the tests showing higher peak deviator stresses reached a higher point on the failure envelope, which is due to the reduction in mean effective stress caused by the decrease in pore pressure (Fig. 7.2f). This observation is consistent with the observations made for reconstituted and undisturbed marine clays.

It should be mentioned that the non-uniformity of pore pressures, as shown earlier in Figs. 7.2d and 7.2e, affects the interpretation of the results as illustrated in Fig. 7.2i using the effective stress paths obtained using top, bottom and mid-height pore pressure in the fastest test KCU4. It shows a shifting of effective stress paths determined using top and bottom pore pressures towards the right hand side due to

the higher pore pressures measured at the top and bottom. Similar phenomenon was observed for undisturbed and reconstituted marine clays.

Fig. 7.2h: Effective stress paths and failure envelop for deformation-controlled tests on Kaolin

Fig. 7.2i: Variation of stress path of test when determined using top, bottom and mid pore pressure measurements

7.2.3 Comparison of Undrained Behaviour in Deformation-controlled and Load-Controlled Tests

Two load-controlled CIU tests, KCU5 (0.3 N/min) and KCU6 (3 N/min) were conducted to study the effect of loading mode on the behaviour of clay. The results of the two tests are presented in Figs. 7.3a to 7.3i. Test KCU5 (0.3 N/min) was comparable with the test KCU1 (0.01 mm/min) and test KCU6 (3 N/min) was comparable with test KCU3 (0.1 mm/min). The load increment rate in any load-controlled test was chosen in such a way that the time taken to reach the peak deviator stress will be the same as that in comparable deformation-controlled test. The same method had been earlier adopted for tests on reconstituted and undisturbed marine clay also.

Variations of axial strain and axial strain rates

As discussed in Chapters 4 and 7, the major difference between the load-controlled and deformation-controlled tests is in the way the axial strain and strain rate vary with time in the two loading modes. This is shown in Figures 7.3a and 7.3b for Kaolin. While the axial strain (Fig. 7.3a) varied linearly and axial strain rate (Fig. 7.3b) was constant with time in deformation-controlled tests KCU1 and KCU3, the axial strain and axial strain rate increased continuously with time throughout the load-controlled tests KCU5 and KCU6. Also, the axial strain rate (Fig. 7.3b) was much smaller in the beginning of load-controlled tests than the axial strain rate in comparable deformation-controlled tests. It may be pointed out that it is difficult to identify the point of onset of instability in Figs. 7.3a and 7.3b because of the gradual change in axial strain and axial strain rate. This is different from the behaviour observed for marine clay in Chapters 4 and 6, where there was a sharp change in axial strain rate.

Fig. 7.3a: Variation of axial strain with time in load-controlled and deformation-controlled undrained tests

Fig. 7.3b: Variation of strain rate with time in LC and DC tests

Stress-strain behaviour

The stress-strain curves in load-controlled and deformation-controlled tests are compared in Fig. 7.3c. The stress-strain curves are similar till about 6% axial strain. However, the curves differ after that and the load-controlled tests attained higher peak deviator stress (by about 5%) than the peak deviator stress attained in comparable deformation-controlled tests. This observation is similar to the observation made for reconstituted marine clay. Fig. 7.3d shows the plot of peak deviator stresses versus axial strain rate in deformation-controlled test. The peak deviator stresses for load-controlled tests are also shown versus average axial strain rate, where average rate is the axial strain at peak divided by the time taken in reaching the peak. There is slight inconsistency in load-controlled tests in that the peak deviator stress in faster test (KCU6 – 1 mm/min) is lesser than the peak in slower test (KCU5 – 0.01 mm/min) by 2% of peak q (Fig. 7.3d). Correspondingly, pore pressure (Fig. 7.3e) was lesser in slower test. This is consistent with the general observation of decreased peak q with increasing pore pressure. The drop in peak q despite increase in strain rate could be due to the specimen variability.

Fig. 7.3c: Stress-strain curves of load-controlled and deformation-controlled tests showing higher peak in load-controlled tests

Fig. 7.3d: comparison of variation in q_m in LC and DC tests

Pore pressure variation

The pore water pressure variations in the load-controlled and deformation-controlled tests are compared in Fig. 7.3e. It can be seen that the pore water pressure developed in load-controlled tests was lesser than that developed in deformation-controlled tests. This could be the reason for higher deviator stress in load-controlled tests. This observation is consistent with the general observation of decreasing peak q with increasing pore pressure for undisturbed marine clay, reconstituted marine clay and Kaolin (deformation-controlled tests).

One possible reason for the higher pore pressure and lower deviator stress in deformation-controlled tests than in load-controlled tests could be higher undrained creep in deformation-controlled test as illustrated in Figs. 7.3f and 7.3g. It can be observed from Fig. 7.3f that the same increase of 32 kPa in deviator stress from 145 kPa to 177 kPa occurred in longer time (420 minutes) in deformation-controlled test than in load-controlled test (210 minutes). Correspondingly (Fig. 7.3g), larger pore pressure increase of 52 kPa occurred in deformation-controlled test than the 38 kPa increase in load-controlled test. This indicates that larger creep related pore pressure

developed in deformation-controlled test, which resulted in lesser mean-effective stress in deformation-controlled test (see Fig. 7.3i) and smaller peak deviator stress.

Fig. 7.3e: Change in mid-pore pressures showing lesser pore pressure in load-controlled tests

Fig. 7.3f: Illustration of creep as the possible reason for higher deviator stress in load-controlled test using q vs. time curve

Fig. 7.3g: Illustration of creep as the possible reason for higher pore pressure and lower q in deformation-controlled test

The non-uniformity of pore pressure between load-controlled and deformation-controlled tests is compared in Fig. 7.3h with the help of the plots of difference between the top and mid pore pressures. It can be seen that the pore water pressure was more uniform in load-controlled tests than deformation-controlled test in faster test, although the pore pressure equalizes quickly due to the high permeability of Kaolin. This observation is similar to those made for undisturbed and reconstituted marine clays and is because of lower initial deformation rate in load-controlled tests than the comparable deformation-controlled tests, as shown earlier in Fig. 7.3b.

Effective stress paths and failure envelope

The effective stress paths in the load-controlled and deformation-controlled tests are compared in Fig. 7.3i. The failure envelope was not significantly affected by the loading mode. It is observed that the effective stress paths of load-controlled tests reached the failure envelope at a higher deviator stress and higher mean effective stress (due to the lower pore pressure, see Fig. 7.3e). This trend is consistent with the observation for load-controlled tests on reconstituted marine clay.

Fig. 7.3h: Difference between top and mid pwp showing larger non-uniformity of pore pressure in DC tests

Fig. 7.3i: Failure envelopes for load-controlled and deformation-controlled tests

Summary of rate and mode effects

From the test results on Kaolin presented above, it has been observed that, in general, the peak deviator stress increases with the increase in shearing rate. However, the

change was much smaller for Kaolin compared to that observed earlier for marine clay. The increase in peak deviator stress per log cycle of strain rate was only 2% for Kaolin while it was 6% and 5% respectively for undisturbed and reconstituted marine clays respectively. The rate-effect on peak deviator stress appears to be smaller at rates slower than the maximum permissible strain rate determined from BS1377 (1990). The pore water pressure was found to be smaller in tests showing higher peak deviator stress. Correspondingly, the effective stress paths reached the failure envelope at a higher deviator stress and higher mean effective stress. This was found to be true for both load-controlled and deformation-controlled tests. Smaller effect of loading rate and loading mode on the behaviour of Kaolin shows that the rate-effect could be related to the permeability of soil. Kaolin, having higher permeability than marine clay, showed lesser rate effect than less permeable marine clays. However, other reasons, such as the mineralogy and structure could also be playing a role in it, which could not be established from the available data.

7.3 SHEARING RATE EFFECTS ON THE BEHAVIOUR OF KAOLIN IN EXTENSION TESTS

The effect of shearing rate on the stress-strain behaviour of Kaolin was also studied in triaxial extension tests through deformation-controlled tests conducted at varying strain rates. The tests included three undrained extension tests and one drained extension test. The results of these tests are presented in Figs. 7.4a to 7.4e. A comparison with the undrained compression tests is made in Figs. 7.5a to 7.5c. It should be mentioned that the load-controlled extension tests could not be carried out due to the limitation of the testing equipment.

Before discussing the results of extension tests on Kaolin, it should be pointed out that extension tests on undisturbed marine clay and reconstituted marine clay could not be conducted due to the limited number of undisturbed samples available. Since reconstituted samples were prepared from used undisturbed marine clay, additional samples for extension tests on reconstituted marine clay could not be prepared. A few extension tests were conducted only on Kaolin to get an indication of shearing rate effect on clay in extension and to guide the future research work.

7.3.1 Deformation-Controlled Extension Tests

Three isotropically consolidated undrained extension (CIUE) tests, namely, KUE1 (0.01 mm/min), KUE2 (0.1 mm/min) and KUE3 (1 mm/min) were conducted in deformation-controlled loading mode to study the rate-effect on the undrained extension behaviour of Kaolin. Figures 7.4a to 7.4e present the results of these three undrained extension tests.

From the stress-strain curves of the three CIUE tests, as shown in Fig. 7.4a, it is clear that there was no significant effect of deformation rate on the stress-strain behaviour of Kaolin in undrained extension. All the tests showed strain hardening behaviour and stress-strain curves of tests KUE1, KUE2 and KUE3 were similar. It was observed during the tests that the deviator stress kept increasing until the specimen failed with the formation of shear band. The pore water pressure variation, as shown in Fig. 7.4b, shows that there was no significant difference between the pore water pressures developed in the three tests. This may be the cause for not observing any significant rate-effect.

Stress paths determined using mid pore water pressure are shown in Fig. 7.4c. It can be seen that the effective stress paths in the three extension tests are quite similar. This is expected because the pore water pressures and stress-strain curves were also similar in the three tests. One reference drained test and one additional undrained test results are also plotted in Fig. 7.4c to show the curvature of failure envelope.

Fig. 7.4a: Stress-strain curves of extension tests on Kaolin

Fig. 7.4b: Mid pore pressure variation in extension tests on Kaolin

Fig. 7.4c: Stress-paths determined using mid pore pressure in extension tests

Pore pressures were non-uniform in the beginning of the faster tests, as shown in Fig. 7.4d. This caused shifting towards the right hand side of the effective stress path determined using the bottom pore pressure in faster tests, as shown in Fig. 7.4e. However, the pore water pressure distribution became uniform much before the failure (Fig. 7.4d) and all the effective stress paths (Fig. 7.4e) reached the failure envelope at the points close to each other.

To summarize, the undrained behaviour of Kaolin in extension seems to be not affected significantly by the deformation rate. The stress-strain curves and pore water pressure variations in CIUE tests conducted at different rates are similar. There are small differences between the stress-paths of the three CIUE tests due to the initial differences between pore water pressures. However, the failure envelope remains unaffected. Kaolin shows hardening behaviour during undrained extension and the failure occurs by shear band formation. The most likely cause of the insignificant rate-effect in Kaolin is its relatively high permeability and ease of equalization of pore water pressure within the specimen.

Fig. 7.4d: Difference between bottom and mid pwp in extension tests

Fig. 7.4e: Stress-paths determined using BOTTOM pore pressure in extension tests

7.3.2 Comparison of Behaviours of Kaolin in Compression and Extension Tests

The stress-strain behaviours of Kaolin in undrained compression and extension tests are compared in Figs. 7.5a to 7.5c. A comparison of effective stress-paths (Fig. 7.5a) shows that the failure envelope in compression has a higher slope than the failure envelope in extension, thus indicating that the effective friction angle will be higher in compression than in extension. However, the undrained shear strength seems to be similar in extension and compression for specimens consolidated under the same effective confining stress. Or, in other words, c_u/p_0' ratio for compression does not differ much from c_u/p_0' ratio for extension. For example (Fig. 7.5a), specimens consolidated under 400 kPa (KCU9 and KUE1) attained peak deviator stresses of 232 kPa and 230 kPa in compression and extension tests respectively. Similarly, the specimens consolidated under 300 kPa (KCU1 and KCUE4) attained peak deviator stresses of 185 kPa and 193 kPa respectively in compression and extension, which are close.

Fig. 7.5a: Comparison of effective stress paths in undrained compression and extension tests

Stress-strain curves in compression and extension tests are shown in Fig. 7.5b. It shows that while extension tests show continuous hardening behaviour till failure, compression tests show almost constant peak deviator stress over large axial strain near the peak q . The reason for hardening behaviour in extension is that the pore water pressure (Fig. 7.5c) in extension tests starts decreasing after increasing initially for some time. Decreasing pore water pressure leads to increased effective confining stress and hence the hardening.

Fig. 7.5b: Comparison of stress-strain curves in compression and extension tests

Fig. 7.5c: Comparison of pore pressure change in undrained compression and extension tests

To summarize, Kaolin did not show any significant rate-effect in extension. Since the rate effect on compression was also small, compression and extension behaviours are in agreement with each other. The most probable reason for the rate-effect on the behaviour of Kaolin being small is that Kaolin has a higher permeability (of the order of 10^{-5} cm/sec) than natural clay (of the order of 10^{-9} cm/sec). Due to this, pore water pressure developed during compression or extension test is quickly and uniformly distributed within the entire specimen. As a result, the mean effective stress remains unaffected by the shearing rate and hence the deviator stress also remains unaffected.

7.4 BEHAVIOUR OF KAOLIN IN CONSTANT STRESS DRAINED TEST

A constant stress drained (CSD) test is the one in which the effective confining stress is reduced while keeping the deviator stress and pore water pressure constant. This results in the movement of the effective stress path towards the failure envelope. The specimen may become unstable when the stress state is close to the failure envelope (Anderson and Reimer, 1995; Dai et al, 1999). To examine the instability behaviour of Kaolin in CSD test, two CSD tests (KCSD1 and KCSD2) were carried out in

compression and two in extension (KCSDE1 and KCSDE2). However, it may be pointed out that rather than keeping the deviator stress constant, load was held constant in this study, which resulted in a small reduction in deviator stress with the progress of test. The results of the four tests are presented in Figs. 7.6a to 7.6d. Results of one CIUC and one CIUE test are also plotted for comparison. Result of one undrained creep test in extension test is also shown.

It can be seen from Fig. 7.6a that in all the CSD tests, the effective stress paths moved towards the failure envelope with the reduction in mean effective stress. In this process, the axial strain rates (Fig. 7.6b) and axial strain (Fig. 7.6c) increased gradually. The instability occurred at effective stress states very close to the failure envelope as indicated in Fig. 7.6a for various CSD tests. Instability was identified as the point at which rapid increase in axial strain rate started to occur, as shown in Fig. 7.6b. Also, the axial strain (Fig. 7.6c) increased rapidly after the instability. Instability was followed by failure with the formation of shear band and rapid drop in deviator stress as shown in Figs. 7.6a and 7.6d. It is important to point out that the load could not be kept constant after the instability occurred. Similar trends were obtained in CSD tests conducted in both compression and extension. The results indicate that the instability occurred very close to the failure envelope (Fig. 7.6a) and pre-failure instability is not significant. This observation is similar to that for undrained creep in reconstituted marine clay.

The effective stress path in the undrained creep in extension (test KCRE1), as shown in Fig. 7.6a, shows that the stress state moved away from the failure envelope even though the effective stress state at the beginning of creep was very close to the failure envelope. This indicates that the specimen will not fail in undrained creep in extension on Kaolin. This is because Kaolin shows hardening behaviour near the peak and it has a tendency to dilate, due to which pore water pressure reduces and the effective stress path moves away from the envelope.

From the results of CSD tests, it is observed that the failure envelope is not affected by the type of effective stress path used for shearing and instability occurs at a state very close to the failure envelope. This observation for Kaolin serves to support the observation from reconstituted marine clay that the pre-failure instability may not occur in reconstituted clay. Since the strain softening before the shear band formation is small in reconstituted clay, the zone of potential instability will be small (see section 5.3.3 in Chapter 5). Hence, pre-failure instability will not be significant. Therefore, the behaviour of Kaolin in CSD tests indicates that pre-failure strain softening is a necessary condition for a significant zone of potential instability to exist and significant pre-failure instability to occur.

Fig. 7.6a: Effective stress path in CSD tests in undrained compression and extension

Fig. 7.6b: Variation of axial strain rate in CSD tests on Kaolin

Fig. 7.6c: Variation of axial strain rate in CSD tests on Kaolin

Fig. 7.6d: Stress-strain curves in CSD tests in undrained compression and extension

7.5 DISCUSSION

The data presented in this chapter showed that the effect of loading rate and loading mode is small for Kaolin. Nevertheless, the observations were in agreement with the overall trends observed from the tests on undisturbed and reconstituted marine clay. General trend of increasing peak deviator stress with increasing axial strain rate was observed in deformation-controlled compression tests on Kaolin, similar to that observed for undisturbed and reconstituted marine clays. The rate-effect appears to be related to the permeability and hence the ease of equalization of pore pressure within the clay. Permeability values for undisturbed marine clay, reconstituted marine clay and Kaolin were in the order of 7×10^{-9} cm/sec, 4×10^{-9} cm/sec and 2×10^{-5} cm/sec respectively. In terms of percentage increase in undrained shear strength, Kaolin, having highest permeability, showed smallest rate-effect (2% increase in peak q per log cycle of strain rate) and change was smaller at strain rates below the maximum permissible strain rate. Undisturbed marine clay, which has lower permeability than Kaolin, showed higher magnitude of increase (6%) in undrained shear strength per log cycle increase in strain rate. Reconstituted clay, which had the smallest permeability amongst the three soils, showed slightly lower rate effect (5%)

on undrained shear strength. This shows that undisturbed soil could be showing higher rate effect both due to both structure and low permeability. Reconstituted marine clay shows lesser rate-effect despite lower permeability, probably because of the loss of structure. These observations indicate that the permeability could be the main factor in determining the extent of rate effect.

The effect of shearing rate on the pore water pressure was similar for the three clays. The pore water pressure decreased with increase in rate, which is due to the lesser creep induced pore pressure in faster tests. The change in pore pressure and corresponding change in shear strength, both were small for Kaolin, which could be due to the high permeability and less creep-induced pore pressure in Kaolin.

For reconstituted marine clay and Kaolin, the effect of shearing rate and shearing mode on the failure envelope was small. Lesser pore water pressure developed in faster tests due to the shorter time for creep induced pore pressure. Due to this the mean effective stress increased and correspondingly peak deviator stress increased in such a way that effective stress state at failure lay on the failure envelope. Thus the rate effect could simply be due to different effective stress-paths followed during shearing due to different mean effective stresses to reach different points on the failure envelope. The specimens at faster rates behave like specimens sheared from a higher initial mean effective stress and reach higher points on the envelope. This is consistent with the failure in terms of effective stresses, implying that the failure envelope in q - p' effective stress space could be more or less unique for remoulded soil. However, for undisturbed marine clay, the specimens in faster tests showed early failure, implying that the failure occurred at an effective stress state before the failure envelope.

It was also observed for all the three soils that the measurement of top and bottom pore water pressure alone might not give correct indication of pore water pressure within the specimen, especially when the tests are conducted at rates faster than that maximum permissible strain rates. Pore pressures at the top and bottom of the specimen are more non-uniform and it is reflected as the difference between top or bottom and mid-height pore pressures. Mid-height pore water pressure was found to be more consistent and a better indicator of the pore water pressure distribution

inside the specimen. Mid-height pore water pressure measurement also gave a better idea of equalization of pore water pressure within the specimen than end measurements. This could be because of the use of rough ends (Rowe and Barden, 1964; Barden and McDermott, 1965). Rough ends cause non-uniformity of pore water pressure near the ends and hence make the measurement inaccurate, especially for tests conducted at fast rates of shearing (Fourie and Dong, 1991). Also, the response time of the pressure transducer installed at the ends of the specimen is longer than the mid-height pore water pressure transducer, which is directly in contact with the specimen and the end measurements lag behind the actual pore water pressure within the specimen (Hight, 1982; Sheahan et al, 1996). This causes significant error in end measurements in the tests conducted at faster rates. However, it may be pointed out that in the case of the Kaolin, larger non-uniformity of pore water pressure occurred only during the beginning stages of tests. The pore water pressures measured at the top, bottom and mid-height of the specimen were similar at the peak and failure, which is due to the ease of equalization of pore water pressure in Kaolin.

Another important observation was that the load-controlled tests were found to have higher undrained shear strength and lower pore water pressure than deformation-controlled tests for Kaolin, similar to the reconstituted marine clay. This is due to higher axial strain rate near the peak in load-controlled test than corresponding deformation-controlled test in both Kaolin and reconstituted marine clay. Higher axial strain rate results in smaller time for creep induced pore pressure to develop and hence lower overall pore pressure in load-controlled test. This leads to an increase in peak deviator stress at faster rates. In undisturbed soil, the tendency of soil to soften after peak due to breaking of structure does not let the deviator stress increase further in load-controlled test. In reconstituted marine clay and Kaolin, the tendency for strain softening before the shear band formation is small and peak deviator stress can increase due to the increase in strain rate.

The results of extension tests on Kaolin show that Kaolin may not be affected significantly by loading rate in undrained extension test. Both stress-strain and pore variation were found to be unaffected by the shearing rate in extension tests. This

could be due to the fact that Kaolin has significantly higher permeability than the marine clay and equalization of pore water pressure takes place easily. The undrained shear strength was found to be of similar magnitude in undrained compression and extension tests.

Results of constant stress drained (CSD) showed that the instability during the CSD tests occurs very close to the failure envelope. This shows that the pre-failure instability will be small. Similar observation was earlier made for reconstituted marine clay in undrained creep tests. These observations indicate that pre-failure strain-softening is a necessary condition for pre-failure instability to occur.

7.6 CONCLUSIONS

Results of the deformation-controlled and load-controlled undrained triaxial compression and extension tests conducted on normally consolidated Kaolin using different shearing rates have been presented in this chapter. Comparison between rate effect on undrained compression and extension behaviours of Kaolin was done. Results from a few CSD tests were also discussed. Some of the conclusions made from the observation made in this chapter and from the comparison with the behaviour of undisturbed and reconstituted Singapore marine clay are as follows.

1. The effect of shearing rate on the undrained behaviour of Kaolin is smaller than that observed for reconstituted and undisturbed marine clays. Only about 2% increase per log cycle increase in shearing rate was observed, which was lower than that observed for undisturbed marine clay (6%) and reconstituted marine clay (5%). Thus, permeability of the soil appears to be the major factor in causing the rate-effects.
2. However, it was consistently observed for Kaolin, similar to the other clays, that the reduction in undrained shear strength is accompanied by a corresponding increase in pore water pressure. This indicates an inverse relationship between the magnitude of pore pressure and peak deviator stress.

3. The failure envelope for Kaolin was unaffected by the shearing rate, which is similar to the observation made for reconstituted marine clay. The tests conducted at faster rates reach the failure envelope at higher mean effective stress and higher deviator stress. Thus the rate effect is a reflection of different effective stress paths followed at different rates for reconstituted clays.
4. Load-controlled tests on Kaolin showed slightly higher undrained shear strength than the deformation-controlled tests. Correspondingly, the pore water pressure developed in load-controlled tests is lesser than that in deformation-controlled tests. This appears to be due to higher axial strain rate near the peak deviator stress and smaller creep related pore pressure in load-controlled tests. This observation is similar to that observed for reconstituted marine clay.
5. Similar to the observations made for undisturbed and reconstituted marine clay, the measurement of mid pore water pressure in addition to the top or bottom pore water pressure gives a better idea of uniformity of pore water pressure within the Kaolin specimen than the measurement of pore water pressure at the top and bottom alone.
6. The c_u/p_0' ratio in extension tests was found to be similar to that obtained in compression test. However, the effective friction angle was lower in extension test.
7. Pre-failure instability may not occur in Kaolin in CSD tests. This is similar to the behaviour observed during undrained creep tests on reconstituted marine clay. The two observations show that destructured clay may not show pre-failure instability. Pre-failure strain softening appears to be a necessary condition for pre-failure instability to occur.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

An experimental study of the effects of shearing rate, loading mode and undrained creep on the stress-strain behaviour of three cohesive soils, namely undisturbed Singapore marine clay, reconstituted marine clay and Kaolin, has been carried out. The main conclusions from this study can be summarized as follows.

8.1.1 Effect of Loading Rate and Loading Mode on the Behaviour of Clay

1. Peak deviator stress (q_m) and hence the undrained shear strength, s_u , of all the three clays increased with the increase in shearing rate in deformation-controlled tests. A linear relationship exists between the peak deviator stress and axial strain rate (where axial strain rate is plotted on logarithmic scale) for undisturbed and reconstituted marine clay. The percentage increase in q_m was 6% and 5% respectively per log cycle increase in shearing rate for these two clays. For Kaolin, the increase in peak deviator stress with the increase in strain rate was smaller (about 2% increase in peak deviator stress per log cycle increase in strain rate).
2. The rate effect is caused by different pore water pressure response of soil to different shearing rates. A reduction in shearing rate results in increased testing duration, thereby resulting in an increase in pore water pressure and hence reduced mean effective stress, which causes the peak deviator stress to increase. Higher pore pressure at slower shearing rates is due to the undrained

creep over longer testing duration. This is supported by the pore water pressure increase observed during undrained creep tests.

3. The variation of axial strain, axial strain rate and load are different in load-controlled and deformation-controlled tests. In load-controlled test the axial strain increases at a progressively increasing axial strain rate. Instability, identified as a rapid increase in axial strain and axial strain is observed. In some of the load-controlled tests, the instability was found to occur at an effective stress state lower than the failure envelope of the soil. Therefore, conventional deformation-controlled tests may give a non-conservative estimate of the failure envelope and effective friction angle in cases where the actual field loading condition is that of load-controlled. Also, post-peak strain softening, which is observed in deformation-controlled tests on undisturbed marine clay, is not observed in load-controlled tests and the failure is rapid after the peak shear strength has been attained.
4. For undisturbed Singapore marine clay, load-controlled and deformation-controlled tests resulted in similar peak deviator stresses when the time to reach the peak deviator stress was kept the same. Correspondingly, the pore pressures were also similar in the two loading mode. This shows that the undrained shear strength depends upon the effective stress within the specimen. It also shows that the conventional deformation-controlled tests can be used to simulate a known load increment rate by keeping the time to failure to be the same for Singapore marine clay. Whether the same is true for other undisturbed soils needs to be studied.
5. For reconstituted marine clay, peak deviator stress was up to 8% higher in load-controlled tests than the deformation-controlled tests, when time to reach the peak were the same. Correspondingly, the pore water pressure was lesser in load-controlled tests. This is because the axial strain rate in load-controlled test became higher than that in deformation-controlled test well before the peak deviator stress was reached and resulted in smaller pore pressure in load-controlled test and a higher deviator stress.

6. For undisturbed marine clay, in faster deformation-controlled tests and load-controlled tests, the specimens failed before the effective stress path could reach the effective failure envelope, i.e. the effective friction angle at failure is lower in faster tests. The early failure was due to the higher non-uniformity of pore pressure at faster rates, which caused shear band to develop earlier and did not allow the pre-failure strain-softening to fully occur. For reconstituted marine clay and Kaolin, in which the pre-failure strain-softening is very small, the effective failure envelope was not affected significantly by the loading rate and loading mode. The undrained effective stress paths in faster tests reached the failure envelope at a higher deviator stress and higher mean effective stress due to the decrease in pore pressure with increase in strain rate.
7. For undrained compression tests in all the three clays, shearing rates equal to or lower than the maximum permissible strain rate determined using BS1377 (1990) result in more uniform pore pressure distribution within the specimen than that in tests conducted at faster rates. The non-uniformity of pore pressure within the specimens increased with the increase in shearing rate in both load-controlled and deformation-controlled tests. This shows that the method suggested by BS1377 (1990) can be reliably used for the estimation of suitable strain rate. However, in this method the axial strain at failure needs to be known before conducting the test. This causes a difficulty in choosing the correct shearing rate. It was found in this research that a more reliable method is to estimate a certain shearing rate and measure the top, bottom and mid-height pore pressure to monitor the non-uniformity of pore pressures and to verify or adjust the chosen shearing rate.
8. Formation of shear band is related to the shearing rate and non-uniformity of pore pressure. In tests conducted at shearing rates lower than the maximum permissible rate, pore pressure equalization was achieved and specimens failed by bulging, thus indicating that the deformation of specimen was uniform. In faster tests, large non-uniformity of pore pressure occurred and early shear band was found to occur, showing that the deformation of specimen was not uniform. Formation of shear band always resulted in rapid

drop in axial load and q/p' . This behaviour can be used to identify the shear band formation in triaxial tests.

9. Drained behaviour of Singapore marine clay is significantly affected by the shearing rate. Large non-uniformity of pore pressure developed in tests conducted at rates faster than the maximum permissible strain rate determined using BS1377 (1990). This resulted in partially drained condition which reduced the shear strength and lowered the failure envelope. For drained tests conducted at or below the maximum permissible strain rate, load-controlled tests and deformation-controlled tests resulted in similar drained shear strength and effective friction angle.

8.1.2 Undrained Creep Behaviour of Clay

1. During undrained creep at constant load in clay, the pore water pressure rises gradually and leads to the movement of effective stress state towards the failure envelope. If the pore water pressure becomes constant after some increase during creep, it is an indication that instability may not occur under given stress conditions. However, if the pore water pressure is found to be continuously increasing, it indicates that instability, characterized by rapid increase in axial strain and axial strain rate may occur soon.
2. A 'zone of potential instability' can be defined for undisturbed marine clay, which is bound by the instability line and the effective failure envelope. When the effective stress state of soil element is in this zone, instability may occur. Since the instability may occur at any point after the effective stress-state has crossed the instability line and reached inside the 'zone of potential instability', it can be concluded that the 'pre-failure instability' may occur during undrained creep in undisturbed clay, similar to that observed in sand. Creep rupture is a result of instability. When the effective stress state of the soil element is below the zone of potential instability, the specimen is stable. The instability line and failure envelope can be determined from conventional CIU tests. The line passing through the peaks of effective stress paths of several CIU tests coincides with the instability line.

3. During undrained creep tests on undisturbed marine clay, specimens failed at deviator stresses of only 77.5% to 91% of the peak deviator stress determined from conventional CIU tests. However, threshold stress level, or the upper yield strength, i.e. the minimum deviator stress during creep at which instability followed by creep rupture or failure will occur (apparently, $0.775q_m$ for undisturbed marine clay) may not be determined precisely for multi-stage creep. The specimen may remain stable even at a stress level higher than the threshold stress level if the effective stress state is below the zone of potential instability. The essential condition for instability to occur is that the effective stress state should lie above the instability line, or in other words, within the zone of potential instability.
4. Primary, secondary and tertiary creep phases exist for undisturbed Singapore marine clay. During undrained creep, if the effective stress conditions are not suitable to cause instability, axial strain continuously decreases, i.e. only primary creep occurs. If instability is about to occur during undrained creep, the axial strain reaches a minimum and remains almost constant for a significant length of time, i.e. secondary creep occurs. Tertiary creep, i.e. the phase of increasing axial strain rate leading to the failure of the specimen, always occurs after the secondary creep stage. Hence, the occurrence of secondary creep can be taken as an indication of imminent instability.
5. During the undrained creep tests on reconstituted marine clay and during constant stress drained tests on Kaolin instability occurred when the effective stress state reached the effective failure envelope. This confirms that the instability line is the line passing through the peaks of the effective stress paths because in case of these two clays, the strain softening was not significant and envelope of peak and failure were the same. It shows that if the failure envelope and the envelope of peaks are the same, pre-failure instability will not occur. Or in other words, pre-failure strain softening is a necessary condition for the pre-failure instability to occur.

8.2 PRACTICAL IMPLICATIONS

1. CIU tests in commercial laboratories are usually conducted at fast rates with pore pressure measurement at the base of the specimen only. This will result in unreliable values of c' and ϕ' . As shown in this thesis, fast rates lead to non-uniformity of pore pressure within the specimen and pore pressure at the end of specimen could either overestimate or underestimate the effective stress. Measurement of mid-height pore pressure should be made a routine practice in common triaxial testing, in commercial laboratories. This will also remove the errors caused by the uncertainties involved in choosing the strain rate due to the unknown axial strain at failure.
2. In load-controlled shearing, instability may occur at an effective stress state well below the failure envelope determined from conventional deformation controlled test. Thus, ϕ' values determined from deformation controlled tests will be unconservative. Therefore, in cases where field loading condition is load controlled, load controlled compression tests should also be conducted to determine the most suitable value of ϕ' for design.
3. Due consideration should be given to the possibility of pre-failure instability due to undrained creep in field loading conditions. In situations, where the problem of undrained creep is foreseen, it will be more conservative to use the effective friction angle at peak, ϕ_p (i.e. the slope of instability line) for design in place of ϕ' at maximum q/p' . This will ensure that the effective stress state of soil will not reach the zone of potential instability during undrained creep.
4. The undrained shear strength mobilized in field can be significantly reduced due to undrained creep. In case of Singapore marine clay, failure occurred at only 77.5% of the peak deviator stress determined from conventional CIU test. Therefore, in situations, where clay is expected to undergo undrained creep due to the chosen construction sequence, it will be advisable to conducted sensitivity analysis with reduced shear strength of clay.

8.3 LIMITATIONS OF THE STUDY

Despite every effort made to conduct the tests in ideal conditions with good quality samples, this study has its limitations and the results should be accepted with following limitations in mind.

1. The samples of undisturbed marine clay had inherent non-homogeneities as many of the samples were from different boreholes (although retrieved from same depth from close locations on same site). Therefore, magnitudes of some of the trends (e.g. effective friction angles) might have been caused by the non-identical samples.
2. Due to the time limitations, in some of the undrained creep tests, time given for undrained creep in some of the stages might not have been enough to bring the effective stress state inside the zone of potential instability. Longer duration could have given slightly different results.
3. The difference in initial water contents of the samples of the three soils could have caused some differences in the observed behaviours.

8.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Several important findings have been made in this thesis. However, due to the time-constraints, several important factors could not be studied. Following recommendations can be made for further research work.

1. Effect of loading mode should be studied for other undisturbed soils, which are found to be affected more by rate-effects. Since, in this study, the rate effects and mode effects were found to be related, it is possible that the behaviour of soils more sensitive to rate effects may be significantly affected by the loading mode. Similar research should ideally be conducted on better quality samples (e.g. block samples) to remove scatter in data caused by non-identical specimens.

2. Due to time-constraints extension tests could not be conducted on marine clay. Effect of loading rate and loading mode on the extension behaviour of marine clay should be conducted with proper pore pressure measurement.
3. Undrained creep behaviour of other undisturbed marine clays should be studied with special emphasis on proper pore pressure measurement and determination of effective stresses. Each stage in test should be given longer duration for creep (10000 to 20000 minutes) to examine whether creep can bring the effective stress state inside the zone of potential instability and cause instability. The concept of pre-failure instability should be examined for other undisturbed clays. Conditions, other than undrained creep and load-controlled shearing, such as constant stress drained shearing, should be examined to verify the existence of the 'zone of potential instability' and the 'instability line'.
4. Inclusion of a mid-height pore pressure transducer will increase the quality and reliability of test results and hence, it should be made an integral part of future researches on the behaviour of clay.

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Appendix -1

Calculation of acceptable strain rates using BS1377 : Part 8 : 1990

1. Undisturbed Singapore marine clay

A typical volume change vs. time curve during isotropic consolidation of undisturbed Singapore marine clay under 200 kPa is shown below in Fig. A1-1.

Fig. A1-1: Typical volume change vs. time curve during isotropic consolidation of undisturbed marine clay

From Fig. A1-1,

$$\sqrt{t_{100}} = 15 \Rightarrow t_{100} = 225 \text{ minutes}$$

$$\Rightarrow t_f = 8.5 \times 225 = 1912 \text{ minutes (for CD tests)}$$

$$\Rightarrow t_f = 2.1 \times 225 = 472 \text{ minutes (for CU tests)}$$

Therefore, axial displacement rates will be as follows:

$$\text{For CD tests, } d_{r(CD)} = \frac{0.25 \times 100}{1912} = 0.013 \text{ mm/min}$$

where, assumed axial strain at failure = 25%

$$\text{For CU tests, } d_{r(CU)} = \frac{0.025 \times 100}{472} = 0.005 \text{ mm/min}$$

where, assumed axial strain at failure = 2.5%

2. Reconstituted Singapore marine clay

A typical volume change vs. time curve during isotropic consolidation under 200 kPa for reconstituted marine clay is shown below in Fig. A1-2.

Fig. A1-2: Typical volume change vs. time curve during isotropic consolidation of reconstituted marine clay

From Fig. A1-2,

$$\sqrt{t_{100}} = 29 \Rightarrow t_{100} = 840 \text{ minutes}$$

$$\Rightarrow t_f = 8.5 \times 870 = 7150 \text{ minutes (for CD tests)}$$

$$\Rightarrow t_f = 2.1 \times 870 = 1760 \text{ minutes (for CU tests)}$$

Therefore, axial displacement rates will be as follows:

For CD tests, $d_{r(CD)} = \frac{0.22 \times 100}{7150} = 0.003 \text{ mm/min}$

where, assumed axial strain at failure = 22%

For CU tests, $d_{r(CU)} = \frac{0.07 \times 100}{1760} = 0.004 \text{ mm/min}$

where, assumed axial strain at failure = 7%

2. Kaolin

A typical volume change vs. time curve during isotropic consolidation under 200 kPa for Kaolin is shown below in Fig. A1-3.

Fig. A1-2: Typical volume change vs. time curve during isotropic consolidation of Kaolin

From Fig. A1-3,

$$\sqrt{t_{100}} = 4.4 \Rightarrow t_{100} = 19.4 \text{ minutes}$$

$$\Rightarrow t_f = 8.5 \times 19.4 = 165 \text{ minutes (for CD tests)}$$

$$\Rightarrow t_f = 2.1 \times 870 = 40 \text{ minutes (for CU tests)}$$

Therefore, axial displacement rates will be as follows:

$$\text{For CD tests, } d_{r(CD)} = \frac{0.21 \times 100}{165} = 0.13 \text{ mm/min}$$

where, assumed axial strain at failure = 21%

$$\text{For CU tests, } d_{r(CU)} = \frac{0.07 \times 100}{40} = 0.175 \text{ mm/min}$$

where, assumed axial strain at failure = 7%