

THE STRENGTH AND DEFORMATION BEHAVIOUR OF A RESIDUAL SOIL IN SINGAPORE



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July 2010

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A Thesis

**Submitted to the Nanyang Technological University
in partial fulfillment of the requirements for the degree of
Doctor of Philosophy**

July 2010

ACKNOWLEDGEMENTS

The research work presented in this report was carried out under the supervision of Associate Professor Chu Jian. I would like to express the most sincere gratitude for his inspiration, patience and guidance. His endlessly support and invaluable experience came handy throughout the whole year. His close monitoring and encouragement are especially appreciated.

I also extends my heartily appreciation to:

The technical staff of Geotechnics labs, especially Mr. Tay Soon Leung, Mr. Vincent Heng, Mr. Eugene Tan and Mr. Andy Koh, for their technical support during preparation of experimental set-up, also, for their kind and prompt assistance during the experiment.

My research colleagues in the School of Civil and Environmental Engineering, especially Manish, Darek, and Claudia, for their unreserved advice and sharing of knowledge about computer control program, setting up the machine and conducting the experiments.

My fellow research students, Guo Wei, Nie Wen and Shi Fan for sharing of knowledge on research topics and for assisting me in collecting information.

All my colleagues in my office in the School of Civil and Environmental Engineering, for being so friendly and lively, and thus making my stay in NTU a memorable event.

Nanyang Technological University, for granting the scholarship for this research program.

And finally, my parents and my friends, especially my beloved husband, for their encouragement on my research and for their continued love throughout my life.

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SUMMARY

Most residual soils in nature are highly structured or cemented depending on the degree of weathering and the formation processes. The shear strength and stress-strain behaviour of residual soil are known to be affected significantly not only by the initial porosity and stress history, but also by the bonds between particles. Although residual soil is commonly encountered during constructions in the tropical region, studies on its engineering properties are far from adequate. There is a lack of in-depth study to characterize the strength and deformation behaviour of intact residual soils in Singapore, especially under more representative testing conditions such as plane-strain conditions.

A comprehensive experimental study on the strength and deformation behaviour of a residual soil in Singapore is presented in this thesis. Large blocks of intact residual soil of Bukit Timah Formation were retrieved about 20 m below the ground surface at a tunneling project and used for the experiments. In order to study the inherent strength anisotropy, specimens cut in both vertical and horizontal directions were tested. The anisotropy behaviour of intact residual soil was also investigated under different major principal stress directions. The tests conducted included K_0 consolidated triaxial compression (CK_0UC), extension (CK_0UE), direct simple shear (CK_0DSS) and plane-strain (CK_0UPS) tests. Some constant-shear-drained (CSD) tests were also conducted to study the deformation and instability behaviour of the intact residual soil of Bukit Timah Formation. Based on the experimental results, the c_u/σ_{v0}' versus OCR relationship was established for each type of tests. The tests conducted on vertically and horizontally cut specimens did not show a strong strength anisotropy in terms of c_u/σ_{v0}' ratio. However, pronounced strength anisotropy both in terms of effective friction angle and undrained shear strength was observed from the tests with major principal stress rotation and plane-strain tests. It is observed from the testing results that, in general, under a given OCR the c_u/σ_{v0}' obtained from the CK_0UPS tests is the highest and this is followed by the CK_0UC tests. The CK_0UE tests give the lowest one, whereas the value obtained from CK_0UDSS tests is somewhere in between. The effective failure envelope is also affected by different types of tests. It is expected that these

data provide useful information for practical geotechnical design works in Singapore Bukit Timah residual soil.

To further study the cementation effect on the shear strength and deformation behaviour of bonded materials, an artificially cemented soil was tested under CK_0UC , CK_0UDSS and CK_0UE conditions to verify the observations made on the intact residual soil. It is noted that stronger cementation resulted in a much higher failure envelope, as well as c_u/σ_{10}' .

The deformation and instability behaviour of residual soil was investigated under CSD conditions. The study was focused on the compacted residual soil under plane-strain conditions to simulate soil and stress conditions beneath an embankment and earth dam using compacted fills. Under CSD testing conditions, instability characterized by a sudden increase in axial strain rate was observed for both NC and OC soils. For compacted soil, the initiation of instability was accompanied by a dilation and shear band formation. These phenomena are similar to the formation of hydraulic fracture. Therefore, the CSD tests may contribute to the understanding of the mechanism of hydraulic fracture. The condition for instability may be linked to the conditions for the occurrence of hydraulic fracture under plane-strain conditions.

The behaviour of intact residual soil under K_0 consolidation was studied using both triaxial and plane-strain apparatuses. Similar studies were also carried out for compacted residual soils and artificially cemented soil. A cementation index, I_c , was suggested to be used to quantify the cementation of bonded materials. It is concluded from the experimental results that the yielding stress was greatly affected by soil cementation. It is also observed that bonding of soil can be reflected by the variation of K_0 value. For intact residual soil and artificially cemented soil, the K_0 value attains a lower value first before it increases gradually, which could be a reflection of de-structuring process of bonded materials.

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LIST OF SYMBOLS

ν	:	Specific volume ($\nu = 1+e$)
ν'	:	Poisson's ratio
ϕ	:	Angle of internal friction in terms of effective stress
ε_1	:	Axial strain
ε_v	:	Volumetric strain
σ_1' / σ_v'	:	Effective vertical stress
$\sigma_1'{}_0$	:	Initial effective vertical stress at beginning of shearing
σ_2'	:	Effective intermediate stress
σ_3	:	Confining pressure/ Cell pressure
σ_3'	:	Lateral effective stress
σ_n	:	Normal stress
σ_p'	:	Pre-consolidation stress
ρ_{bulk}	:	Bulk Density
ϕ_{cs}	:	Angle of friction at critical state
ϕ_s'	:	Secant friction angle
Δu	:	Excess pore water pressure
A	:	Skempton's pore pressure parameter
A_f	:	Skempton's pore pressure parameter at failure
a_f	:	The Henkel pore pressure parameter at failure
b	:	Relative magnitude of the intermediate principal stress
c'	:	Cohesion
c_u / s_u	:	Undrained shear strength
$c_u / \sigma_1'{}_0$	:	Undrained shear strength ratio
e	:	Void ratio
e_0	:	Initial void ratio
E_{tan} / E_t	:	Tangential stiffness
I_c	:	Cementation index
p'	:	Mean effective stress, $(\sigma_1' + 2\sigma_3')/3$

p'_0	:	Mean effective stress at beginning of shearing
P_f	:	Initiation pressure for hydraulic fracture
q	:	Deviator stress ($\sigma'_1 - \sigma'_3 = \sigma_1 - \sigma_3$)
q/p'	:	Effective stress ratio
q_0	:	Deviator stress at beginning of shearing
q_{peak}	:	Peak deviator stress
R	:	$R = \frac{\sigma'_1}{\sigma'_3}$
T_0	:	Coupled stress
τ	:	Shear strength
CIU	:	Isotropically consolidated undrained test
CK ₀ UC	:	K_0 consolidated undrained compression test
CK ₀ UDSS	:	Direct simple shear test
CK ₀ UE	:	K_0 consolidated undrained undrained extension test
CK ₀ UPS	:	K_0 consolidated undrained undrained Plane-strain test
CSD	:	Constant-shear-drained test
CSL	:	Critical state line
CSSM	:	Critical state soil mechanics
DPVC	:	Digital pressure volume controller
LVDT	:	Linear Variable Differential Transformer
M	:	Slope of the critical state line
NC	:	Normal consolidated
NCL	:	Normal consolidation line
OC	:	Overconsolidated
OCR	:	Over-consolidation ratio
PWP / u	:	Pore water pressure
SEM	:	Scanning electron microscope
SPT	:	Standard Penetration Test
UU	:	Unconsolidated-undrained test
VST	:	Vane shear test

CHAPTER 1

INTRODUCTION

1.1 Background

Residual soil is formed by the decomposition of rocks, which is one of the most predominate geological formations in Singapore. It is highly structured in nature and may also be cemented depending on the degree of weathering. The structural nature of the soil affects its mechanical behaviour. For example, the strength and deformation behaviour of residual soils may be highly anisotropic. The stability and deformation of geotechnical structures such as retaining walls and tunnels are affected by the strength and deformation behaviour of soil. The strength and deformation of soil are affected by how the stresses are applied to the soil. Although the stresses are normally applied under plane-strain and three dimensional conditions in the field, the soil properties are normally measured under axisymmetric conditions using a triaxial cell.

The previous researches on residual soils have covered topics such as the effect of weathering during soil formation (Vaughan & Kwan, 1984), characterizing the engineering properties (Winn et al., 2001; Viana da Fonseca, 2003; Hight & Leroueil, 2003; Rahardjo et al., 2004; Zhang et al., 2007), investigating the effects of soil structure on the engineering properties (Leroueil & Vaughan, 1990; Nagaraj et al., 1998), analyzing the compressibility behaviour (Nagaraj et al., 1998) and also evaluating the collapse behaviour (Rao & Revanasiddappa, 2006; Huat et al., 2008). However, the majority of laboratory studies on residual soils were carried out on small scale of intact residual soils or compacted residual soils, and tested under conventional triaxial or oedometer conditions. There is a lack of in-depth study

reported in the literature to characterise the strength and deformation behaviour of the intact residual soil of Bukit Timah granite in Singapore under generalised stress conditions. Therefore, a comparative study was carried under axisymmetric and plane-strain conditions. A Series of K_0 consolidated triaxial compression (CK_0UC), extension (CK_0UE), direct simple shear (CK_0DSS) and plane-strain (CK_0UPS) tests were performed on large block of intact residual soil samples. In order to study the strength anisotropy of the intact residual soil, large block samples were cut in both vertical and horizontal directions for testing.

Instability of loose granular materials under undrained conditions has been well understood and established (Lade et al., 1988; Lade & Pradel, 1990; Lade, 1992, 1993, 1994; Leong, 2001; Chu et al., 2003). It has also been demonstrated by many researchers that the granular materials can become unstable under drained or strain path controlled conditions (Lade et al., 1987, 1988; Chu, 1991; Chu et al., 1993, 2003; Chu & Leong, 2001 and Loke, 2004). Although a number of studies have been focused on the occurrence of instability, most of them were conducted on sand, except Zhu & Anderson (1998). The instability behaviour of other materials, such as residual soils should also be investigated. Through investigation on the effect of load mode on the instability behaviour of sand, Chu & Wanatowski (2009) suggested to study the instability behaviour under a load-controlled loading mode. This is consistent with previous studies carried out under axisymmetric conditions by Chu & Leong (2001) and Chu et al. (2003). As such, a series of constant-shear-drained (CSD) tests were conducted in this project to study the instability of residual soil in Singapore. Further research under both axisymmetric and plane-strain conditions was also carried out.

1.2 Objectives

The project aims to achieve the following objectives:

- 1) To characterize the shear strength and stress-strain characteristics of the intact residual soil of Bukit Timah granite under both axisymmetric and plane-strain conditions.

- 2) To study the effect of cementation on the shear strength behaviour of intact residual soil and an artificially cemented soil.
- 3) To investigate the deformation and instability behaviour of residual soils, especially under plane-strain conditions.

1.3 Scope of Work

The proposed study was performed on three types of soils: intact residual soil of Bukit Timah granite, compacted residual soil and an artificially cemented soil. To characterize the shear strength and stress-strain characteristics of the intact residual soil, a series of laboratory experiments were conducted under both axisymmetric and plane-strain conditions. The strength anisotropy of the intact residual soil was also investigated by carrying out laboratory tests on specimens in different cutting orientations, as well as by conducting tests at two different major principal stress directions namely compression and extension. To further study the cementation effect on the shear strength of bonded materials, an artificially cemented soil was tested under CK_0UC , CK_0UDSS and CK_0UE conditions to compare with the observations made on the intact residual soil. Finally, a series of constant-shear-drained (CSD) tests were also performed to study the deformation and instability behaviour of residual soils. The CSD data were also used as an element study to understand the mechanisms of hydraulic fracturing, which is the known or suspected cause of excessive leakage and failure of many dams. In addition, the K_0 consolidation results were interpreted to study the consolidation behaviour of the three tested soils. Based on the experimental data, a cementation index was proposed as a direct measurement of the degree of cementation for bonded materials.

1.4 Outline of Thesis

This report is divided into eight chapters. Chapter 2 presents a literature review on previous theoretical and experimental studies, especially on the engineering properties of soil, instability of granular materials and yield of bonded soils. Sample preparation and the experimental set-up for different type of tests are discussed in Chapter 3. The experimental methodology is also presented in this chapter. The

consolidation behaviour of three tested soils is studied in Chapter 4. The undrained shear strength and stress-strain behaviour of the intact residual soil of Bukit Timah granite under various testing conditions are discussed in Chapter 5. The strength anisotropy was also investigated in this Chapter. To study the effect of cementation on the shear strength of bonded materials, a comparative study was carried out between an artificially cemented soil and the intact residual soil in Chapter 6. The deformation and instability behaviour of three tested soils is discussed in Chapter 7. Finally, the conclusions and recommendations for future study are made in Chapter 8.

CHAPTER 2

LITERATURE REVIEW

2.1 Residual Soils

2.1.1 Introduction

Residual soils are widely distributed in tropical regions. They are formed in place by the following processes, e.g. incorporation of humus (decaying vegetation), physical and chemical weathering, leaching of insoluble materials, accumulation of insoluble residues, downward movement of fine particles (lessivage) and disturbance by root penetrations, animal burrowing, free fall and desiccation (Fookes, 1997). In tropical regions, weathering of the parent rocks of residual soils is more intense and occurs to greater depths than elsewhere. This is led by the tropical climatic conditions, such as high temperatures and precipitation. High temperatures are favourable to the development of chemical reactions that greatly responsible for rock weathering. The abundance of water, especially the generation of underground flows will cause leaching process and the inherent creation of new structural (micro and macro) matrices, and even new fabrics (Viana da Fonseca, 2003). The engineering behaviour of residual soils is significantly influenced by the progress of weathering process, the presence of bonded structures and fabric, and also by the depth of penetration of which will be influenced by the discontinuities in the parent rock (Hight & Leroueil, 2003; Viana da Fonseca, 2003).

Anon (1977) recommended a generalized description for a scale of weathering grades of the rock mass (as shown in Table 2.1). The terms given are general and may be modified to suit particular situations; subdivision may be necessary.

Chandler (1968) and Ward et al. (1968) also suggested different schemes for Keuper Marl and chalk.

It should also be pointed out that not all weathering grades could be seen in a given rock mass. And in some cases, a particular grade may be present to a very small extent. The distribution of the various weathering grades may be related to the porosity and the presence of open discontinuities of all types in the rock mass (Anon, 1977).

Table 2.1 Scale of Weathering Grades of Rock Mass (Anon, 1977)

Term	Description	Grade
Fresh	No visible sign of rock material weathering.	IA
Faintly Weathered	Discoloration on major discontinuity surfaces.	IB
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering.	II
Moderately Weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as core stones.	III
Highly Weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discolored rock is presented either as a discontinuous framework or as core stones.	IV
Completely Weathered	All rock material is decomposed and or disintegrated to soil. The original mass structure is still largely intact.	V
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Duchaufour (1982) distinguished three phases of residual soil development in tropical area (as shown in Table 2.2). They are characterized by increasing weathering of parent rocks, increasing loss of silica and increasing dominance of new clay minerals formed from dissolved materials. The general distribution of the

tropical residual soils is presented in Figure 2.1 (Fookes, 1997), which was determined based on the world soil map provided by Food and Agriculture Organization (FAO). In Figure 2.1, the broad classes of soils extend beyond the tropics, which also include high rainfall sub-tropical continental east coasts (ferrallitic soils), and the west coast/Mediterranean and continental interiors in mid-latitudes (fersiallitic soils). Area in the tropics but shown in blank include those where tropical residual soils are overlain by recent alluvial and aeolian deposits (Fookes, 1997).

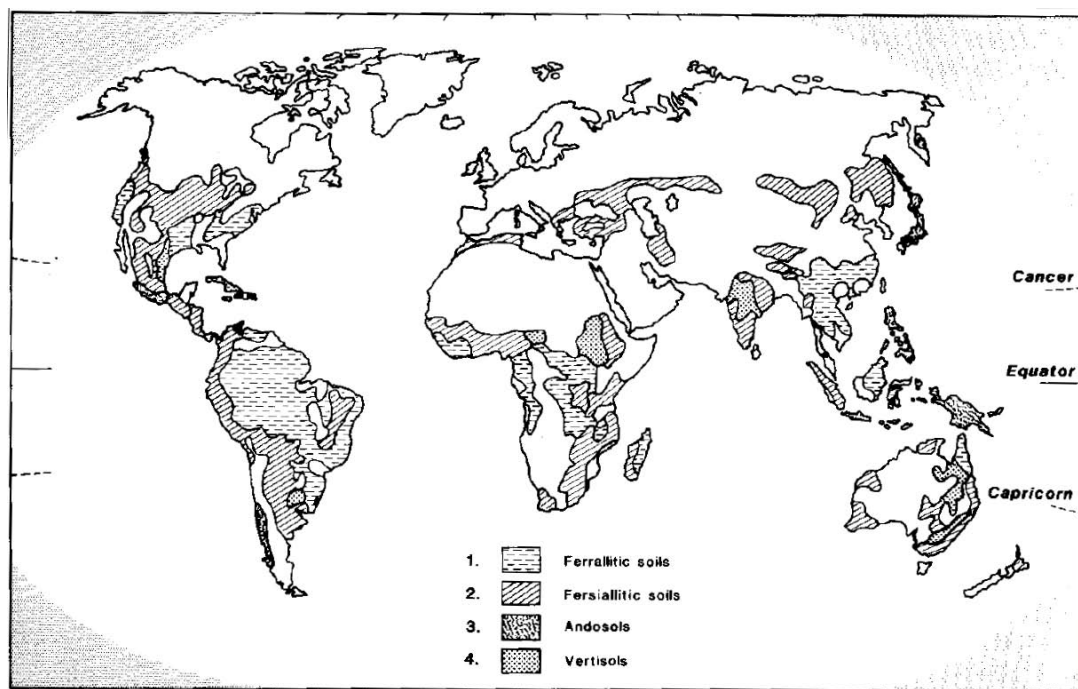


Figure 2.1 Simplified World Distribution of the Principal Types of Tropical Residual Soils Based on FAO World Soil Map (after Fookes, 1997)

The phases listed in Figure 2.1 represented by the soil profile at any particular location are determined by numerous factors (Fookes, 1997), which include: the age of the land surface, the climatic conditions during this period of soil development, the composition of the soil parent rocks, and topography (e.g. controlling lateral transportation of bases, silica and iron in solution, and rejuvenation of profiles by erosion on steeper slopes).

*Table 2.2 Summary of Residual Soils Phases in Relation to Climate Factors
(Duchaufour, 1982)*

Phase	Soil Type	Zone	Mean annual temperature (C)	Annual rainfall (m)	Dry Season
1	Fersiallitic	Mediterranean , subtropical	13-20	0.5 - 1.0	Yes
2	Ferruginous ferrisols (transitional)	subtropical	20-25	1.0 – 1.5	Sometimes
3	ferrallitic	tropical	> 25	> 1.5	No

2.1.2 Residual Soil in Singapore

Singapore, is an island city-state located at the southern tip of the Malay Peninsula, lying 137 kilometers north of the equator, south of the Malaysian state of Johor and north of Indonesia's Riau Islands. The geology of Singapore consists essentially of four formations: (i) igneous rocks of granitic composition (Bukit Timah Granite) in the centre and northwest, (ii) sedimentary rocks (Jurong Formation) in the west and southwest, (iii) a semi-hardened alluvium (Old Alluvium) which masks older rocks beneath in the east, and (iv) the recent deposits of Kallang Formation of the alluvium member, the transitional member and marine clay distributed throughout the island. Figure 2.2 shows a simplified geology map outlining the distribution of the four major geological formations of Singapore (PWD, 1976).

The formation process of residual soils is complex and their characteristics are very different from those of transported soils. Residual soils are highly structured in nature and may also be cemented depending on the degree of weathering. The mechanical behaviour of residual soils depends not only on the initial porosity and stress history, but mainly on the bonds between particles (Maccarini, 1993). These bonds present in the parent rocks still exist partially in the residual soil structure, depending on the degree of weathering.

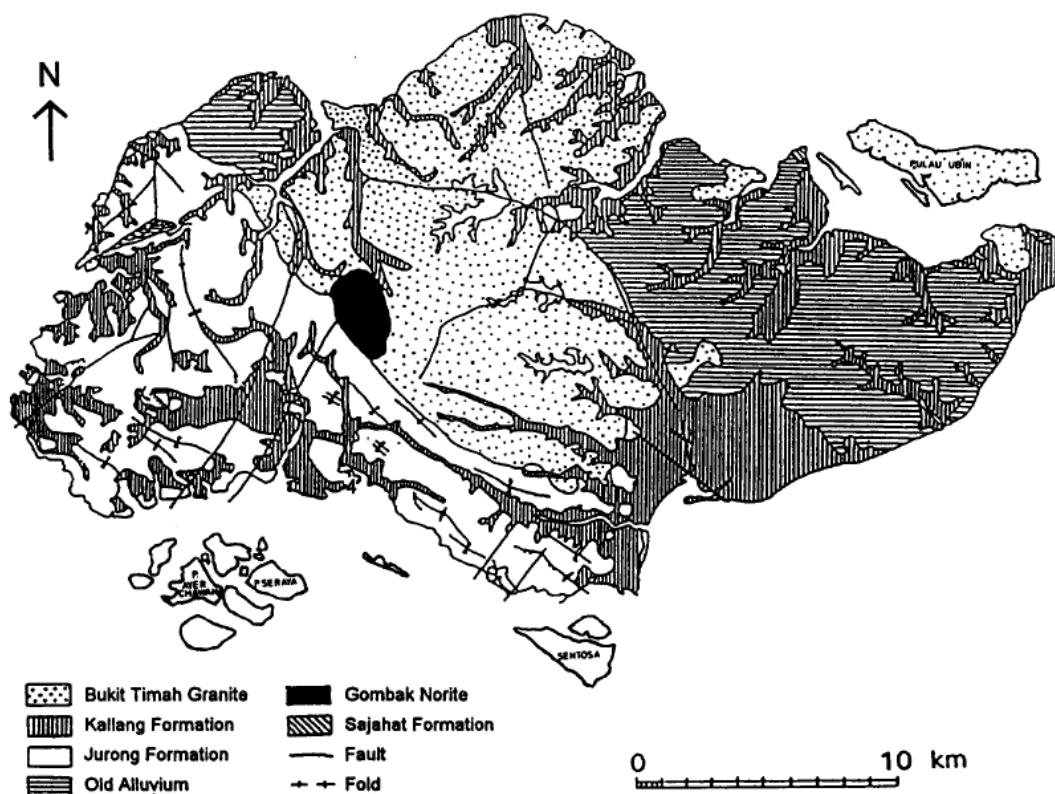


Figure 2.2 Simplified Geological Map of Singapore (after PWD, 1976)

The Bukit Timah Granite is one of the oldest formations in Singapore. It is widely distributed in the central and northern parts of Singapore Island. Large blocks of the intact residual soil of Bukit Timah Granite were studied in this project. The intrusion of the Bukit Timah Granite is believed to have taken place during the lower to middle Triassic period (200 to 250 million years ago). The thickness of the residual soils varies from a few meters to several tens of meters. The residual soils are highly heterogeneous and their properties appear to be highly variable. According to Winn et al. (2001), the residual soils of Bukit Timah Granite consists mainly of reddish brown sandy silty clay which is underlain by a thick layer of yellowish brown sandy clayey silt and greenish to whitish grey silty clayey sand and sandy silt. The total thickness ranges from a few meters to over 40 meters. The topmost part of reddish brown sandy silty clay, which is formed by the laterisation effect (secondary weathering), has a thickness of up to 5 meters. In some places, this layer is missing probably due to erosion and previous earthworks in the areas. The bottommost subsoil of greenish to whitish grey silty clayey sand and sandy silt,

encountered immediately above the parent rock, is coarser in grain size than the others. The greenish coloration is observed due to the chloritisation of biotitic (mica minerals) which still remains fresh in places. The original texture of granite is partially preserved in places. If the degree of weathering is intensive, the soil type is practically not present.

The degree of weathering of the granite decreases with depth. The top portion of the granite has been completely weathered into residual soils, while the fresh granite bedrock remains at great depths. The granite rocks, directly underneath the residual soils, are usually highly weathered and fractured. The residual soils derived from the heavily weathered granite have undergone further weathering resulting in changes in color. Field observations indicate that the weathering of the Bukit Timah Granite has been rapid and is primarily due to the chemical decomposition under the humid tropical climate of Singapore. The residual soil of Bukit Timah Granite can be divided into two general groups as shown in Table 2.3 (Winn et al., 2001). The corresponding weathering grades are defined by Dames and Moore (1983) as presented in Table 2.4.

The typical properties of the weathered Bukit Timah Granite are summarised in Table 2.5 (Sharma et al., 1999). It should be noted that the mechanical and engineering properties change significantly at different weathering grades.

Table 2.3 Two Main Groups of Bukit Timah Granite (Winn et al., 2001)

Group	Soil Description	Total Thickness (m)	Weathering Grade
I (fine grained soil)	Reddish brown sandy silty CLAY Yellowish brown sandy clayey SILT	Total up to over 70 m (commonly 40 to 50 m thick)	G4 G4
II (coarse grained soil)	Greenish to whitish grey silty clayey sand and sandy SILT		G2-G4

Table 2.4 Modified Weathering Grade Classification for Granitic and Sedimentary Residual Soils (Dames and Moore, 1983)

Symbol	Rock Type	Equivalent Weathering Grade	General Description
G1 / S1	Granite (G)/	I and II	Fresh to slightly weathered rock
G2 / S2	Sedimentary (S)	III and IV	Moderately to highly weathered rock
G3 / S3			Mouldery soil: Boulders of rock of variable weathering within completely weathered rock or residual soil
G4 / S4		V and VI	Completely weathered rock or residual soil

Table 2.5 Properties of the Weathered Bukit Timah Granite (after Sharma et al., 1999)

Weathering Guide	ρ_{bulk} (g/cm ³)	SPT N-value	RQD (%)	$I_{s(50)}$ (MPa)	σ_c (MPa)	V_p (Km/s)	E_d (GPa)	K (x 10 ⁻⁹) (m/s)
Residual soil (VI)	2.09	13	-	-	-	1.2	4.5	1.0*
Completely weathered (V)	2.14	33	-	-	-	0.9	5.4	-
Highly weathered (IV)	2.32	>100	45	1.8	32	0.7	5.4	5.12
Moderately weathered (III)	2.43	-	83	5.6	88	4.8	33.2	1.80
Slightly weathered	2.54	-	96	9.9	165	5.6	56.2	1.59
Fresh granite	2.66	-	99	11.1	192	5.8	60.3	0.58
Note: ρ_{bulk} = Bulk Density; SPT = Standard Penetration Test; RQD = Rock Quality Designation; $I_{s(50)}$ = Point Load Index; σ_c = Uniaxial Compressive Strength; V_p = Velocity of Compression Wave; E_d = Dynamic Modulus of Elasticity; k = Rock Mass Permeability								
* In the normally consolidated range								

2.1.3 Previous Studies on Residual Soils

Residual soil, as a common but important material in tropical regions, has been studied intensively for decades. Researches were carried out to characterize the engineering properties of residual soils (Winn et al., 2001; Viana da Fonseca, 2003; Hight & Leroueil, 2003; Rahardjo et al., 2004; Zhang et al., 2007), to study the weathering effects during soil formation (Vaughan & Kwan, 1984), to index the

engineering properties of residual soils (Vaughan et al., 1988), to investigate the effects of soil structure on the engineering properties (Leroueil & Vaughan, 1990; Nagaraj et al., 1998), to analyze the compressibility behaviour (Nagaraj et al., 1998) and also to evaluate the collapse behaviour of it (Rao & Revanasiddappa, 2006; Huat et al., 2008).

Vaughan & Kwan (1984) proposed a quasi-elastic theory to relate the formation of residual soil to the in-situ stresses developed in it. The weathering process during residual soil formation involved weakening and a reduction in both stiffness and strength. The corresponding loss in stiffness during weakening process was modelled by isotropic elastic theory with reducing values of compression modulus E' and Poisson's ratio, ν' ; and loss of strength by a Coulomb failure criteria with reducing c' and ϕ' . The proposed theory is useful to predict the in-situ horizontal stress and also to study the limiting elastic stress state in the infinite slope. It was observed that the weakening effect caused the horizontal stress tend to approach a limiting value calculated from the Poisson's ratio as follow:

$$\sigma'_h = \sigma'_v \frac{\nu'}{1 - \nu'} \quad (2-1)$$

It was also noted that the ground slope led to a reduction in horizontal in-situ stress. Tensile stresses may develop in steep slope. The combination of low horizontal stress with decreasing shear strength due to weathering is likely to cause shear failure, particularly in slopes.

By comparing the test results on artificial and natural residual soils, Vaughan et al. (1988) confirmed that the yield of the bonded structure can be mapped in q - p' - e space in a similar manner as sedimentary clays did. Vaughan et al. (1988) also suggested two methods to index the engineering properties of residual soil, in a way similar to the indexing of sands by relative density. The first method is to compare the in situ void ratio of residual soils with two standard ones for the de-structured soil. The fall-cone test and standard compaction test are suggested for determination of the higher and lower standard void ratios. The second indexing method involves the use of the oedometer test on the de-structured soil to establish the boundary

between the stable and meta-stable states. This method provides the extent of the stable zone for structured soil when subject to engineering loading or during shearing.

The effects of soil structure were also an important area studied by many researchers (e.g. Leroueil & Vaughan, 1990; Nagaraj et al., 1998; Newson et al., 2006). The classical models of soil mechanics involve the concepts of initial void ratio and its subsequent modification by stress history (Vaughan, 1985 and Vaughan et al., 1988). As mentioned in Leroueil & Vaughan (1990), the strength and stiffness which come from the cementation or soil bonding are only considered in rock mechanics. Mesri (1975) and Tavenas & Leroueil (1990) concluded that even most natural soft clays are structured.

Leroueil & Vaughan (1990) investigated the effects of soil structure on the engineering properties of natural sedimentary and residual deposited. They concluded that the soil structure increases the strength and enlarges the stress domain in which the soil shows stiff behaviour. It is also pointed out by Leroueil & Vaughan (1990) that the magnitude of the yield stress and the shape of the yield curve due to structure reflect density, as well as bond strength. As suggested by them, the effects of soil structure should be considered as important as the initial void ratio and stress history, in determining engineering behaviour of natural soils. This study has been supported by Mitchell (1993) and Nagaraj et al. (1998). By comparing the undrained strength data of undisturbed and remolded samples during oedometer tests, it suggested that the cementation effects should also be taken into consideration in the analysis of consolidation behaviour of residual soils.

Numerous studies were carried out on unsaturated residual soils, too, such as: Aung et al. (2001), Peixoto et al. (2000), Day, 2001; Bell & Culshaw, 2001; Rao & Revanasiddappa (2002), Huat & Ali (2005), Huat et al. (2005a, 2005b, 2006, 2007, 2008), Aziz et al. (2006) and Singh et al. (2009). Most of these researches were conducted to investigate the volume change and suction behaviour of unsaturated residual soils. Partially saturated residual soils often exhibit collapse and are

characterized by a bonded structure which greatly affects the compression and strength behaviour (Vaughan, 1988; Fookes et al., 1990; Alonso & Gens, 1994; Barksdale & Blight, 1997; Wheeler, 1994; Rao & Revanasiddappa, 2002, 2006).

In order to study the collapse behaviour of compacted residual soil fills, Rao & Revanasiddappa (2006) examined the influence of alternate wetting and drying on the collapse behaviour of compacted residual soil specimens from Bangalore District. Experimental results indicated that the cyclic wetting and drying process increased the degree of expansiveness of the residual soils and reduced their collapse tendency. Changes in the swell/collapse behaviour of compacted residual soil specimens from wetting drying effects are attributed to reduction in water content, void ratio and possible growth of cementation bonds.

Huat et al. (2008) also performed a study to examine the effect of wetting on the collapsibility and shear strength of two commonly found residual soils in Peninsular Malaysia. From the results of oedometer tests, collapse upon wetting (inundation) was observed for both types of soils. Huat et al. (2008) concluded that for a given type of soil, samples with higher initial dry densities and degree of saturation appeared to have lower collapse potential. Similar observation of collapse potential increase with dry density decrease is reported by Day (2001). The SEM analysis indicated that the soil structure became more compacted and voids between the structures became smaller upon collapse (Huat et al., 2008). The collapse process represents a rearrangement of soil particles into a denser state packing (Bell & Culshaw, 2001; Huat et al., 2008).

Huat et al. (2008) also conducted shear box tests to study the shear strength of the two types of residual soils. The results showed that the shear strength of both soils reduced significantly upon collapse, in particular the apparent cohesion.

It should point out that the majority of laboratory studies on residual soils were carried out on small scale of intact residual soils or compacted residual soils, and tested under triaxial or oedometer conditions. The studies on the strength and

deformation behaviour of large blocks intact residual soils, especially under plane-strain conditions were seldom. Relevant investigation should be carried out.

2.2 Engineering Properties of Soil

2.2.1 Shear Strength

Shear strength is one of the most important engineering properties of soil. It is also one of the most complex engineering properties. Unlike other civil engineering materials, such as steel, the shear strength of a soil is not a function of the material only, but also a function of the stress applied to it as well as of the manner in which these stresses are applied. The shear strength of soil, therefore, cannot be tabulated in codes of practice since the same soil can develop different shear strengths under different fields and engineering conditions.

There are a number of failure equations proposed in the past depending on whether effective stress analysis or total stress analysis is adopted, Mohr-Coulomb equation can be expressed either as:

$$\tau = c' + \sigma' \tan \phi' \quad (2-2)$$

where τ = shear strength, σ' = effective normal stress, c' = cohesion and ϕ' = angle of internal friction, or

$$\tau = c_u + \sigma \tan \phi_u \quad (2-3)$$

where $\phi_u = 0^\circ$ and c_u is the undrained shear strength.

The determination of the shear strength parameters, c' and ϕ' , or c_u , for soils is primarily based on laboratory tests on undisturbed samples or in-situ tests. Undrained triaxial tests are normally used to determine shear strength parameters of clay. In conducting these tests, K_0 consolidation may be adopted to restore the in-situ effective stresses before undrained shearing was carried out.

As pointed out by many researchers (Ladd, 1986; Kulhawy & Mayne, 1990; Chu et al., 1999), the undrained shear strength, c_u , is not a constant, but varies with many

factors, including effective overburden stress σ_{v0}' , OCR, consolidation path (isotropic or K_0 consolidation), stress states and stress anisotropy.

The compressibility and shear strength of soil deposit are strongly affected by its stress history. If the present effective overburden pressure of the soil is the maximum pressure which it was subjected to in the past, then the soil is said to be normally consolidated (NC), or the over-consolidation ratio $OCR = 1$. On the other hand, if the soil has been subjected to a larger vertical effective stress in the past than at present, the soil is said to be overconsolidated (OC), with $OCR > 1$.

The undrained behaviour of soil is greatly affected by the OCR of the soil. The typical undrained behaviour of NC and OC clay is schematically shown in Figure 2.3. For NC clay, it is seen that the deviator stress reaches a peak at small strain, and then reduces. The pore water pressure increases gradually as shearing proceeds. For OC clay, it is observed that the deviator stress increases and only reaches a peak at large axial strain. The excess pore water pressure for OC clay in undrained test increases first then decreases.

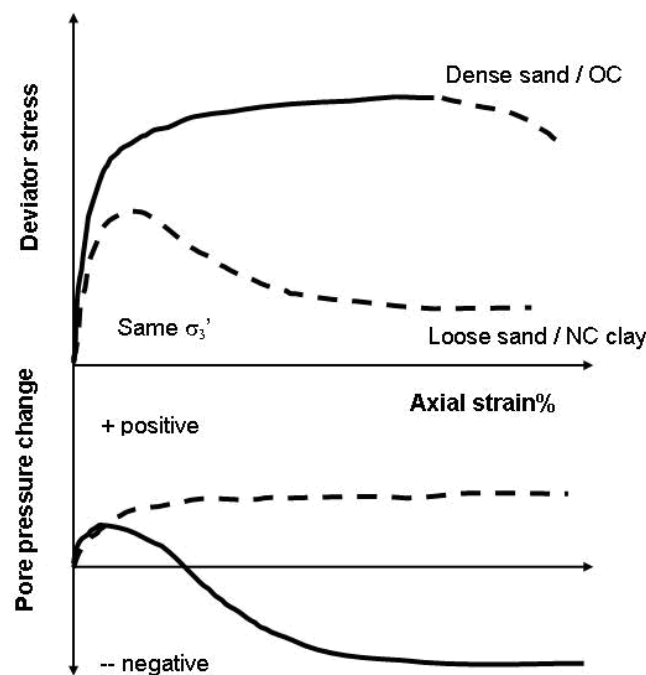


Figure 2.3 Illustration of Typical Undrained Behaviour of Sand / Clay

The effect of OCR on the undrained shear strength of clays is usually described by the following SHANSEP equation (Ladd & Foott, 1974):

$$\left(\frac{c_u}{\sigma_{v0}'}\right)_{OC} = \left(\frac{c_u}{\sigma_{v0}'}\right)_{NC} OCR^A \quad (2-4)$$

where c_u = undrained shear strength; σ_v' = effective overburden stress; and A = constant.

The above relationship can be derived from a well-established constitutive model, such as the Cam-Clay model (Wood, 1990) and has been verified by numerous experimental data (Ladd, 1986; Kulhawy & Mayne, 1990; Chu et al., 1999; Chung et al., 2002; Watabe et al., 2002; Chu et al., 2003; Bo et al., 2003; Moses et al., 2003). Once the equation is established experimentally, the undrained shear strength c_u of the soil can be determined using the equation.

However, as c_u is affected by many other factors, such as the rotation of the major principal stress and the testing methods adopted for c_u measurement. Figure 2.4 shows the c_u/σ_{v0}' versus OCR curves obtained from different tests conducted on Singapore marine clay. It is seen that the c_u/σ_{v0}' ratios obtained from isotropically consolidated triaxial compression (I.C.) tests is the highest, followed by that obtained from K_0 consolidated triaxial compression (K_0C) tests and direct simple shear (K_0DSS) tests. The c_u/σ_{v0}' ratios obtained from K_0 consolidated triaxial extension (K_0E) tests are the lowest.

Similar observation has been made on Boston Blue Clay, as presented in Table 2.6 (Mayne et al, 2009). The tests were conducted and reported by MIT (Ladd & Lambe, 1963 and Ladd et al., 1980). By conducting different laboratory tests on normally consolidated Boston Blue Clay ($OCR = 1$), it noted that the undrained shear strength ratio $S = (c_u/\sigma_{v0}')_{NC}$ varied from a low value of 0.14 for the unconfined compression test (UC) to a high value of 0.34 for plane strain compression (PSC). It is clearly evident that for Boston Blue Clay, the c_u (or s_u) value is dependent on the testing mode adopted. Ladd (1991) concluded that the

direct simple shear (DSS) test could provide an overall reasonable and representative c_u (or s_u) value.

In addition, Mayne et al. (2009) also pointed out that the range of c_u could be even more widespread from the in-situ tests. The various sets of undrained shear strength measurements for the well-documented Bothkennar soft clay site in the UK are presented in Figure 2.5. The tests conducted by Hight et al. (1992; 2003) included in-situ vane shear test (VST), pressuremeter tests (PMT), as well as laboratory a series of K_0 consolidated triaxial compression, triaxial extension and direct simple shear tests on undisturbed samples. At a certain depth, the c_u (or s_u) values might range up to a factor of six, depending on the testing method adopted. As summarized by Mayne et al. (2009), the factors attributed to the differences in the various c_u (or s_u) values may include: initial stress state (isotropic or anisotropic), direction of loading (compression or extension), boundary conditions (triaxial or plane-strain), rate of loading, inherent fabric anisotropy, sensitivity, strain compatibility and other variables.

It should also be mentioned that Eq. (2-4) is normally obtained for sedimentary soils where the undrained shear strength may be normalized by σ'_{v0} , i.e. c_u/σ'_{v0} may be unique for soil with the same OCR. Several studies have also shown the applicability of SHANSEP equation (Eq. 2-4) for clays with some degree of cementation (e.g., Chung et al., 2002; Watabe et al., 2002; Bo et al., 2003; Moses et al., 2003), even for clay shales (Steiger & Leung, 1991; Gutierrez et al., 2008). However, whether this is the case for residual soils needs to be investigated.

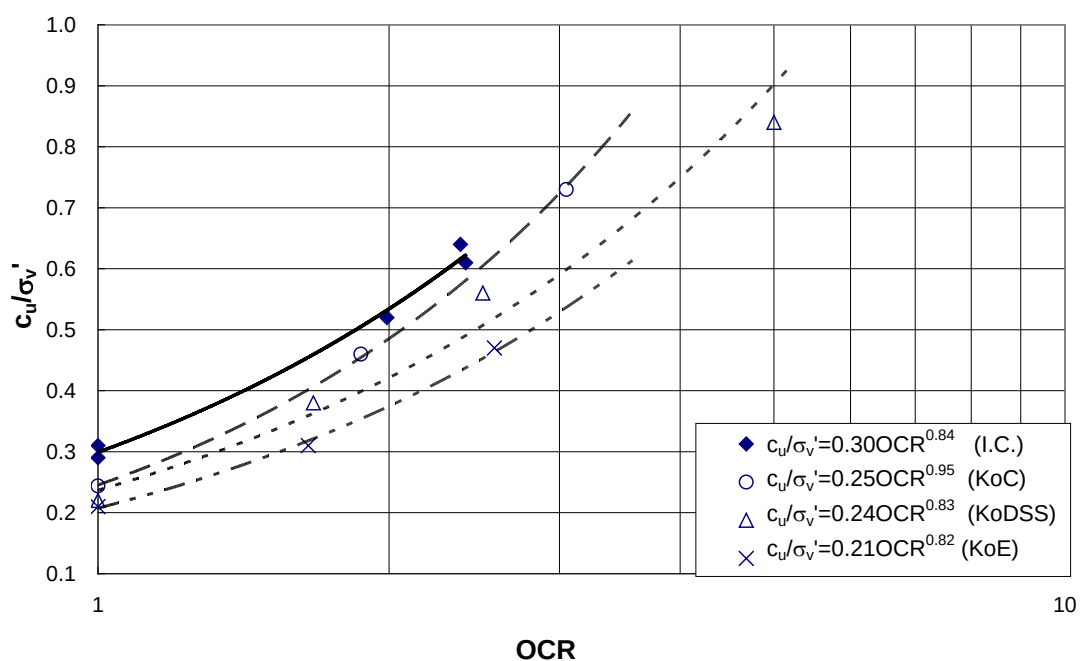


Figure 2.4 c_u/σ_{v0}' Ratio Obtained from Tests on Marine Clay (Lecture Notes by Chu)

Table 2.6 Undrained Strength Ratios (S) for Normally-Consolidated Boston Blue Clay (after Mayne et al., 2009, Based on the Data Reported by Ladd & Lambe, 1963 and Ladd et al., 1980)

Test Method	$S = \left(\frac{c_u}{\sigma_{v0}'} \right)_{NC}$
Plane-Strain Compression (PSC)	0.34
Triaxial Compression (CK ₀ UC)	0.33
Iso-consolidated Triaxial Compression (CIUC)	0.32
Iso-consolidated Triaxial Extension (CIUE)	0.24
Direct Simple Shear (DSS)	0.20
Plane-Strain Extension (PSE)	0.19
Unconsolidated Undrained (UU)	0.185
Triaxial Extension (CK ₀ UE)	0.16
Unconfined compression (UC)	0.14

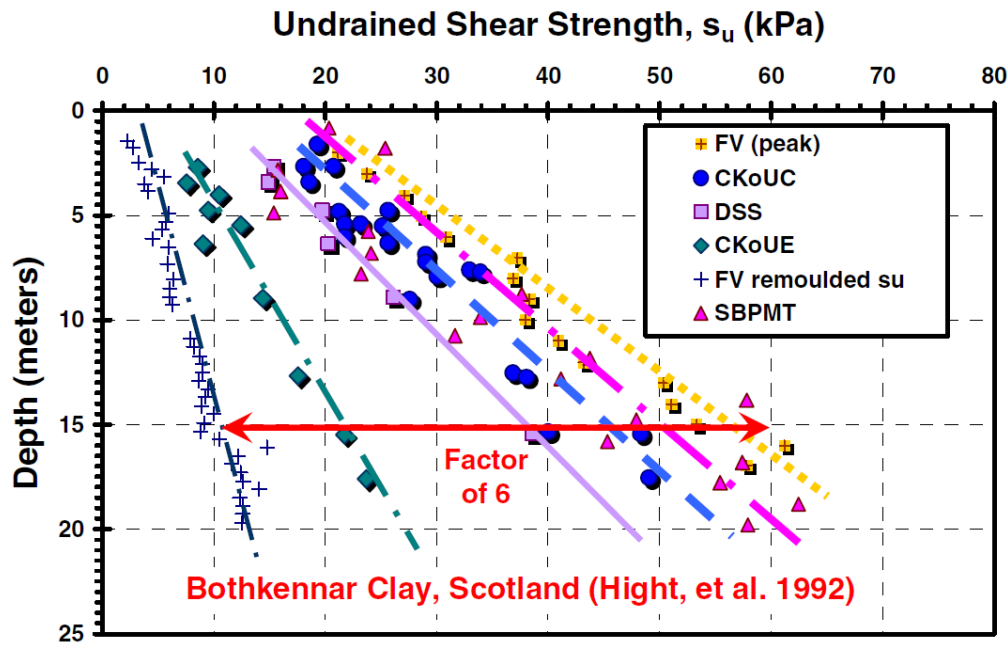


Figure 2.5 Six-Fold Variance of s_u (or c_u) from Different Laboratory and Field Tests at Well-Documented test site in Soft Bothkennar Clay (after Mayne et al., 2009, Based on the Data Reported by Hight et al., 1992, 2003)

Different relationships were also proposed by some other researchers to calculate the undrained stress ratio S , e.g. Wroth (1984), Ladd & DeGroot (2003). By studying the results from different type of tests (triaxial, plane-strain, direct simple shear, pressure-meter and vane shear tests), Wroth (1984) suggested the following equation to evaluate the undrained shear strength for over-consolidated soils:

$$S = (c_u / \sigma_{v0}')_{OC} = (1/2) \sin \phi' OCR^\Lambda \quad (2-5)$$

where $\Lambda = 1 - C_s / C_v$ (C_s is the compression index and C_v is the swelling index).

It is observed by Kulhawy & Mayne (1990), the parameter Λ lies within a relative narrow range of 0.8 ± 0.1 for low-medium sensitive clays up to around 0.9 ± 0.1 for sensitive and structured soils.

As can be seen from Figure 2.6 (after Mayne et al., 2009), the above equation provides a reasonable estimation on the normalized undrained shear strength. It is also noticed that for fissured geomaterials (such as London clay at Brent Cross

indicated by the dashed line in Figure 2.6), the undrained strength can be reduced by one-half or more, depending upon the extent, closeness, and frequency of the discontinuities.

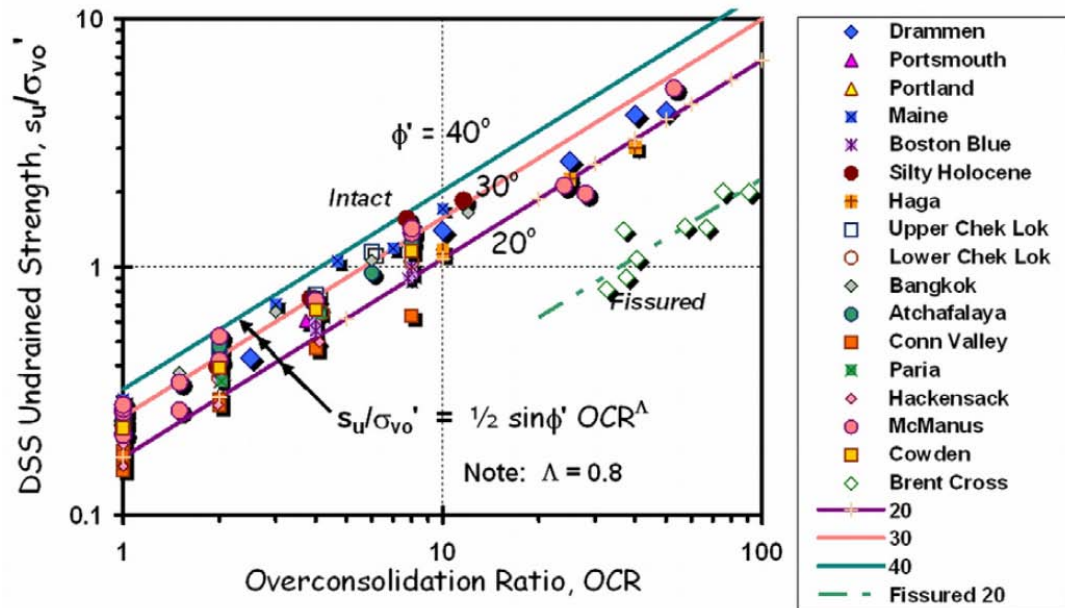


Figure 2.6 Undrained Strength Ratios from DSS Tests on OC Clays (after Mayne et al., 2009)

It is also pointed out by Mayne et al. (2009), while the importance of ϕ' is clear, the much larger effect of OCR must be realized as impacting the above relationship. In that case, the following relationship was concluded to estimate the DSS strength for homogeneous lean to plastic sedimentary clays of low to moderated sensitivity (Ladd & DeGroot, 2003). The coefficients in the relationship should be adjusted for sensitive marine clays, varved clays, silts and organic soils (e.g. Ladd, 1991).

$$S = (c_u/\sigma_{v0}')_{OC} = 0.22OCR^{0.80} \quad (2-6)$$

Another equation was proposed by Mesri (1975) to calibrate the vane shear test results with failure case studies of embankments, footings and excavations.

$$(c_u)_{Mobilized} = 0.22\sigma_p \quad (2-7)$$

For NC cases ($\text{OCR}=1$), the pre-consolidation stress equals to the effective overburden stress. However, vane shear tests may not be applicable to firm to hard residual soils.

2.2.2 Stiffness

The stiffness of soil is a very important parameter for the prediction of soil movement. Commonly the stiffness of soil is defined as the elastic Young's modulus, E measured as either tangent or secant to the stress-strain curve (Chu & Lo, 1993) (as shown in Figure 2.7). Various definitions of soil stiffness are explained in Tatsuoka & Shibuya (1991).

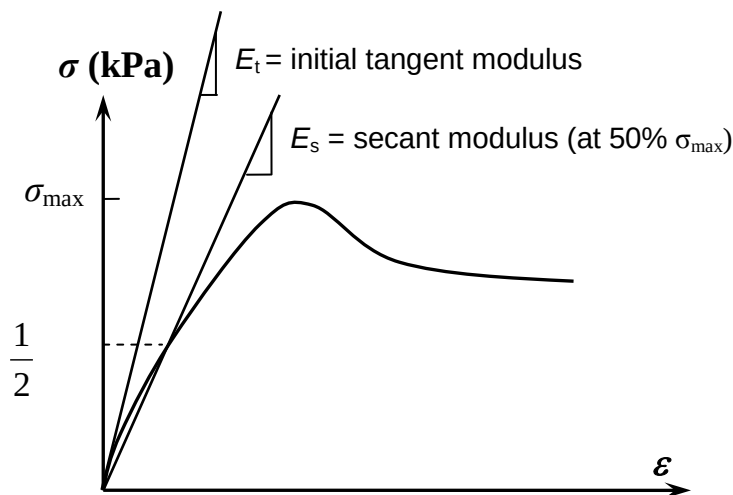


Figure 2.7 Definitions of the Initial Tangent Modulus and the Secant Modulus

The stiffness of soil is a strain level dependent parameter. Its magnitude decreases significantly with the increase in the strain level. As shown schematically in Figure 2.8, in which, E is the elastic modulus, and ϵ_c is a threshold strain value in the order of 10^{-3} %. According to the magnitude of the strain, the stiffness can be characterized into three regions. The stiffness of soil in micro strain region (strain value is less than ϵ_c) may be regarded as constant. Within small strain region (strain value between ϵ_c and 1%), the stiffness shows a considerable variation with strain. When the strain value is beyond 1% (within medium strain region), the reduction of stiffness with strain tends to be smaller.

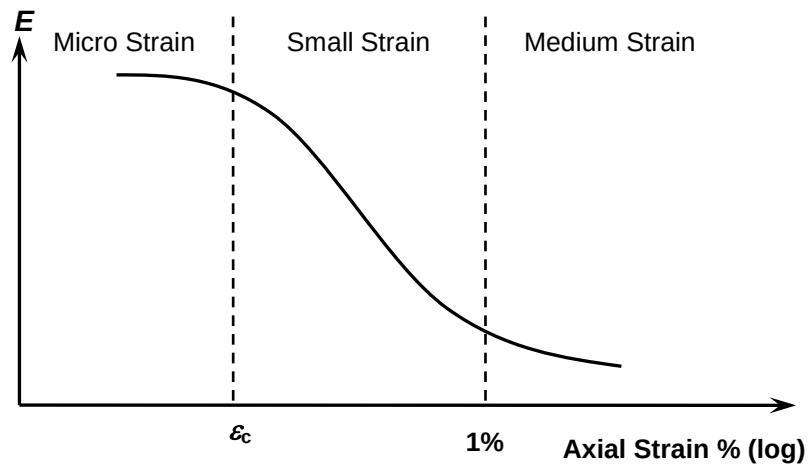


Figure 2.8 Characteristic Stiffness-Strain Curve for Soil (after Mair, 1993)

The level of strain encountered in practice is presented in Figure 2.9 (Mair, 1993). However, due to the limit of the conventional triaxial apparatus, axial strain within a level of less than 0.1% cannot be measured accurately. To overcome this difficulty, different types of local strain measuring devices have been devised and used in triaxial testing (Jardine et al., 1984; Tatsuoka & Shibuya, 1991). It has been shown that by using a local strain gauge, it is possible to measure the stiffness of stiff clays reliably by laboratory tests (Jardine et al., 1984; Burland, 1989; Chu & Lo, 1993; Mukabi et al., 1994; Ng et al., 1998; Atkinson, 2000). Since all these devices were tested under axisymmetric stress conditions, studies under plane-strain conditions are required to investigate the performance and workability of those local strain measuring devices.

The stiffness of soil is also stress path dependent. As shown in Figure 2.10 as example, the stress-strain curves for different stress paths A, B, C and D are different. As a result, the stiffness obtained will also be different.

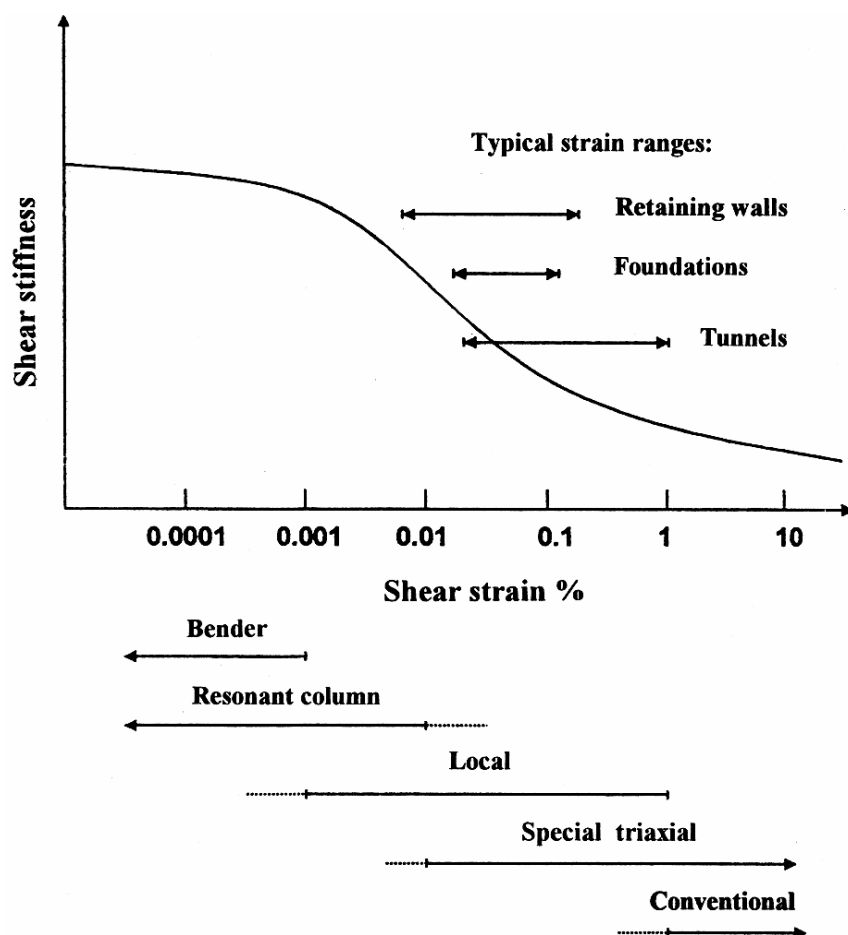


Figure 2.9 Approximate Strain Limits for Soil Structures (After Atkinson & Sallfors, 1991 and Mair, 1993)

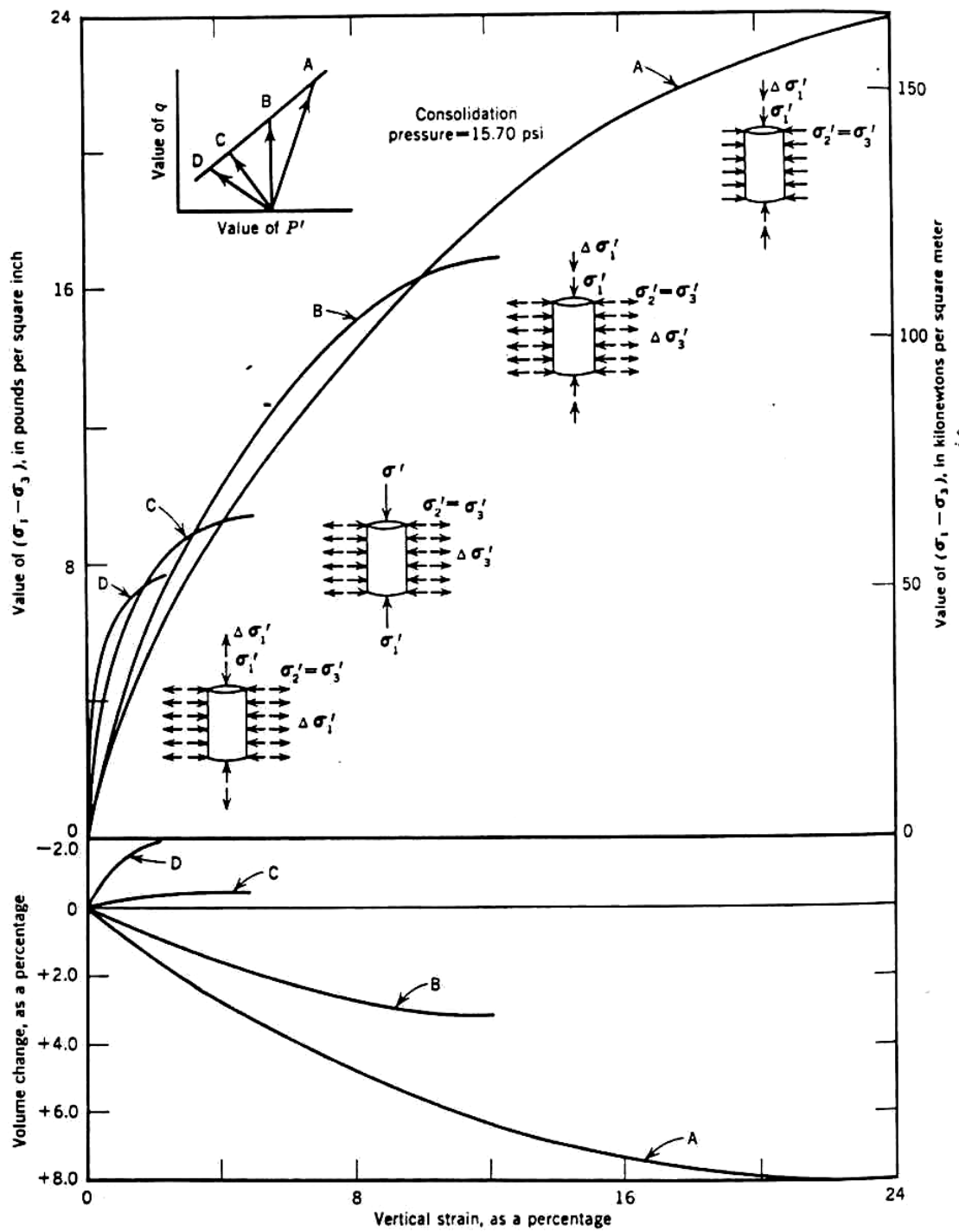


Figure 2.10 Stress-Strain Behaviour for Different Tests (Lecture Notes by Chu)

2.2.3 Anisotropy Behaviour

Anisotropy refers to the variation in material properties with direction. When soil deposits, its particles have strong tendency to align themselves in a direction normal to the direction of deposition. Therefore, natural clays exhibit significantly different structure and strengths in the vertical and horizontal directions.

Civil engineering construction almost invariably involves changes to the stress state in the ground, resulting in movements and requiring stability under the new stress regime to be considered (Leroueil & Hight, 2003). The construction process of earth structures will also lead to principal stress rotation. Figure 2.11 shows an example of stress changes beneath a vertically loaded circular footing (Leroueil & Hight, 2003). The stress state changes in terms of deviator stress are presented in Figure 2.11(a), as contours of $(\Delta\sigma_1 - \Delta\sigma_3)/2p$ associated with a pressure, p applied on the circular footing. $\Delta\sigma_1$ and $\Delta\sigma_3$ are the changes in major and minor principal stresses respectively. The changes of stresses beneath a circular footing could also be presented in terms of major principal stress direction or changes in the relative magnitude of the intermediate principal stress (Δb) (as shown in Figure 2.11(b) and (c)). The direction of major principal stress was defined as an angle, α to the vertical, and the Δb value is calculated as:

$$\Delta b = \frac{\Delta\sigma_2 - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} \quad (2-8)$$

It is clearly observed that both magnitude and direction of principal stresses were affected due to the loading imposed on circular footing.

Another example is given in Figure 2.12 for the stress conditions along failure surface beneath embankment. If the soil is anisotropic, the available undrained strength will vary along the potential failure surface. The c_u changes with σ_1 along the slip surface as shown in Figure 2.12 (Bjerrum & Aitchison, 1973; Graham, 1979; Zdravkovic et al, 2002). The rotation of principal stresses was also obtained in excavation project (Figure 2.13) (after Clough & Hansen, 1981).

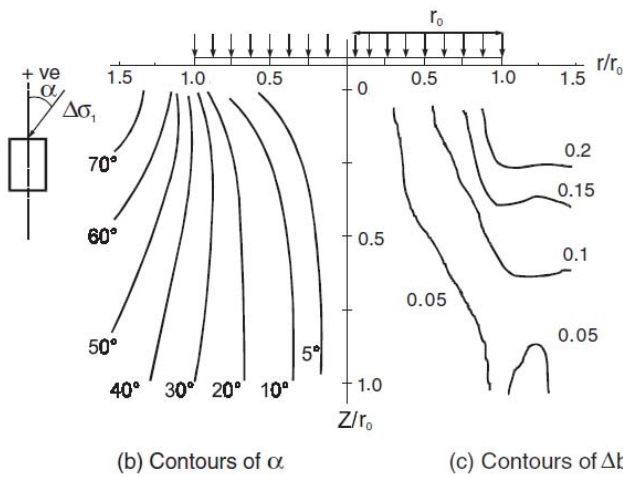
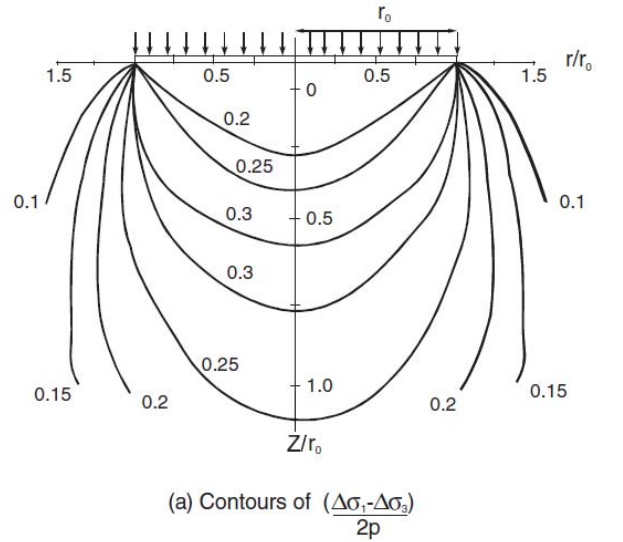


Figure 2.11 Stress Changes Beneath Vertically Loaded Circular Footing on Isotropic Linear Elastic Foundation ($\nu = 0.45$): (a) Contours of $(\Delta\sigma_1 - \Delta\sigma_3)/2p$; (b) Contours of α ; (c) Contours of Δb (after Leroueil & Hight, 2003)

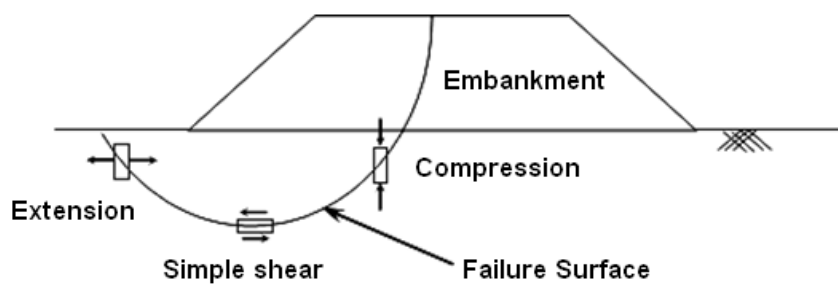


Figure 2.12 Laboratory Experiments that Resemble Stress Conditions along the Failure Surface beneath an Embankment (after Bjerrum & Aitchison, 1973)

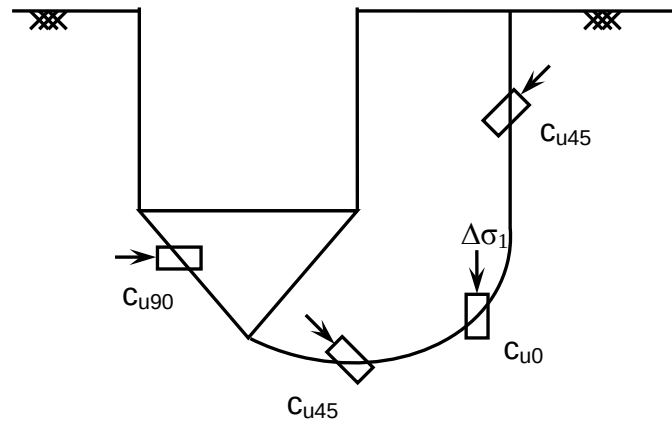


Figure 2.13 Rotations of Principal Stresses in Excavation Project

(After Clough & Hansen, 1981)

The effects of the principal stress rotation can be attributed to the anisotropy of soils, including inherent anisotropy and induced anisotropy. The inherent anisotropy refers a physical characteristic inherent in the material and entirely independent of the applied stress, whereas the induced anisotropy refers a physical characteristic due exclusively to the strain associated with applied stress (Casagrande & Carrillo, 1944).

Since sedimentary soils are inherently anisotropic, their response to loading will depend on the directions of principal stresses in relation to the deposition direction. Rotation of the principal axes of stress occurs in practice under most types of loading. Even the loading imposed by the simplest foundation induces a rotation of axes at all points, except under the center of the foundation (or under the centerline if it is a strip footing) (Joer et al, 1998). The importance of rotation of axes is probably greatest where cyclic loading is involved, such as an offshore platform (Ishihara & Towatha, 1983). As a consequence, deformations occur not only because of changes in the magnitudes of principal stresses but also due to the changes in their directions. For a given magnitude of principal stress increment, deformations due to the accompanying change in directions depend on the inclinations of principal stresses to the material axes prior to the stress change (Vaid et al., 1990).

As for induced anisotropy, Broms and Casbarian (1965) proposed that its effects on clay may be attributed to the reorientation of the individual clay particles. It has been observed (Lambe, 1958; Hvorslev, 1960) that flat clay particles have a tendency to orient themselves perpendicular to the direction of the major principal stress. The tendency of the individual clay particles to align themselves parallel with the final failure plane will increase with increasing rotation of the principal stress directions.

Figure 2.14 shows the variation of undrained shear strength ratio for four soft Norwegian clays with the angle of the major principal stress reorientation (α) during loading (after O'Rourke et al., 1976). The condition of $\alpha = 0^\circ$ represents the normal compression testing condition, with the major principal stress at failure, σ_{1f}' acting vertically. If $\alpha = 90^\circ$, the major principal stress, which begins in a vertical plane, is rotated so that at failure it is horizontal. The results suggest that the undrained shear strength is a maximum at $\alpha = 0^\circ$, and decreases steadily as α increases.

Besides α (the rotation of the major principal stress to the vertical), the relative magnitude of the intermediate stress, b is another stress parameter to describe the stress conditions, where $b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3}$. The areas of generalized stress space (q - b - α)

for some standard shear tests are illustrated in Figure 2.15 (after Hight et al., 1987). It should be noted that $c_{u(TC)}/c_{u(TE)}$ is often referred to as anisotropy, it actually reflects the combined effects of b and α , as well as any inherent soil property. Similar conclusion has been reported by Ladd (1991) as shown in Figure 2.16.

Parry & Nadarajah (1974) conducted a series of triaxial compression and extension tests on silty clay under multi-stage conditions. The clay sample was cut in vertical, horizontal and inclined (45° to the vertical) directions. The test results showed that the anisotropy effect is reduced as isotropic consolidation pressure increases. They

also observed that the q_E/q_C at failure and friction angle ϕ' is independent of the specimen orientation.

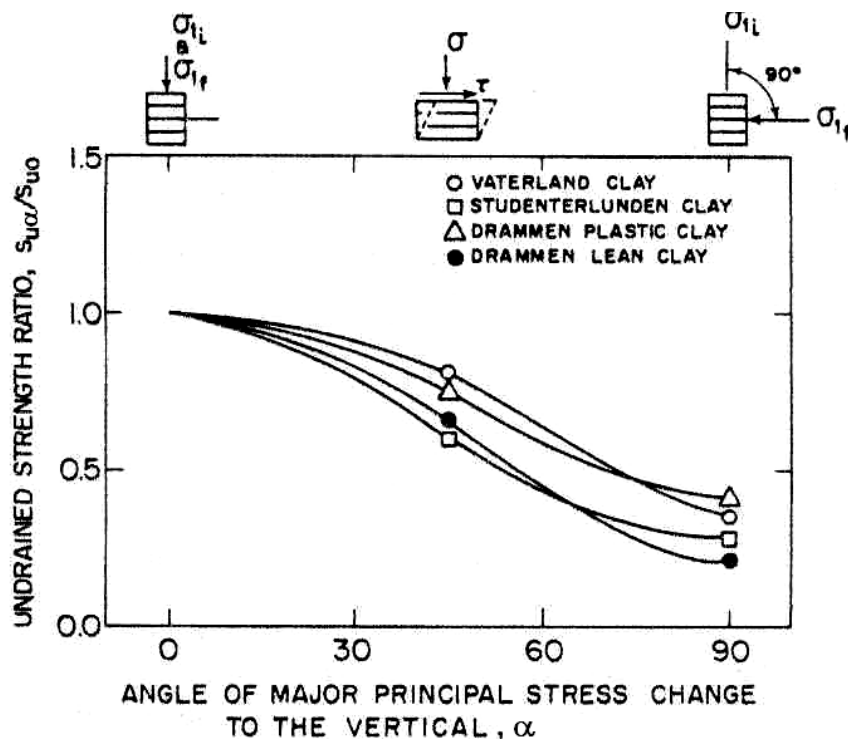
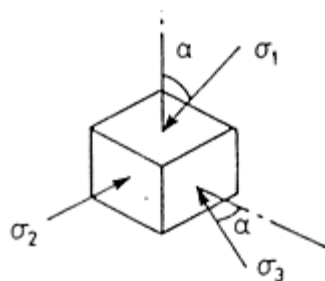
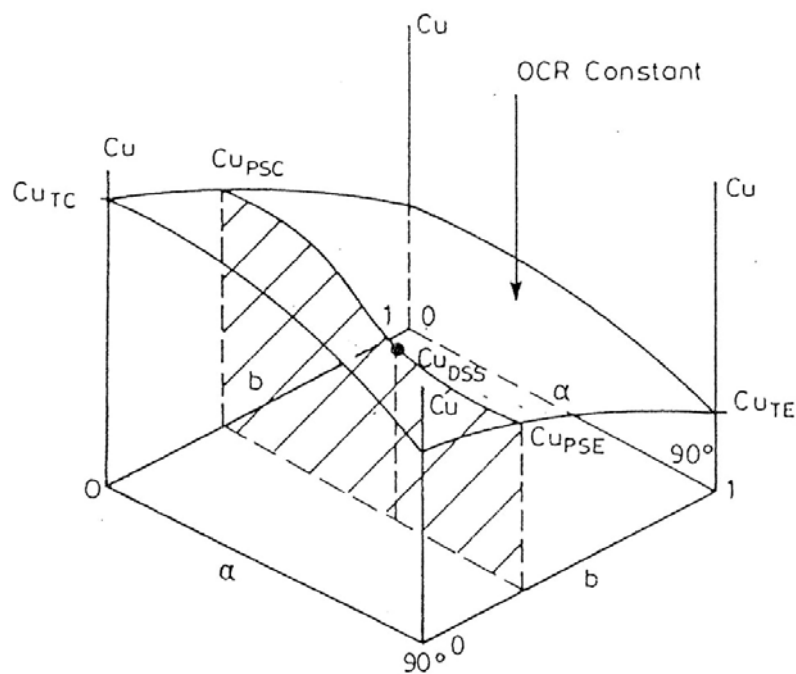


Figure 2.14 Variation of Undrained Shear Strength Ratio with Angle of Stress Reorientation for Four Norwegian Clays (after O'Rourke et al., 1976)



$$b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3)$$

TC = Triaxial Compression

TE = Triaxial Extension

PSC = Plane-Strain Compression

PSE = Plane-Strain Extension

DSS = Simple Shear

*Figure 2.15 Effects of Shearing Direction and Stress State on Undrained Strength
(after Hight et al., 1987)*

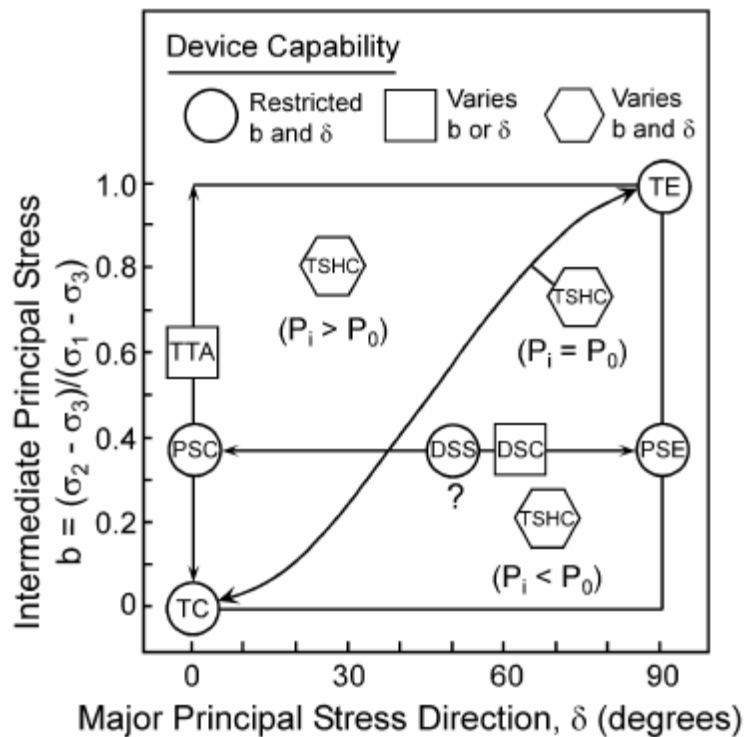


Figure 2.16 Stress Systems Achievable by Shear Devices for CK_0U Testing
(Modified from Germaine, 1982) (after Ladd, 1991)

2.3 Critical State Soil Mechanics (CSSM)

Critical state soil mechanics (CSSM) is an effective stress framework to describe the mechanical soil response. It was firstly introduced in the 1930's in an attempt to relate volume change behaviour, specimen density and confining pressuring during shearing (Casagrande, 1936). Based on the results of drained direct shear tests, he found there is a critical void ratio for a given type of sand, dependent only on the confining stress imposed during shearing. Casagrande's concept became a major component of the CSSM, developed in the 1960's to characterize soil behaviour in an elasto-plastic framework. Critical state is reached when the soil specimen continues to deform at constant shear stress, constant normal stress and constant volume (Schofield & Wroth, 1968). For the soils which exhibit strain softening, critical state occurs at maximum q/p' ratio; for other soils, critical state occurs at

peak deviator stress. Nevertheless, in common geotechnical design the failure is assumed at the peak strength.

In the CSSM, the following stress and strain parameters (or invariants) were used under an axisymmetric condition:

$$p' = (\sigma'_1 + 2\sigma'_3)/3 \quad (2-9)$$

$$q = \sigma_1 - \sigma_3 = \sigma'_1 - \sigma'_3 \quad (2-10)$$

$$\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 \quad (2-11)$$

$$\varepsilon_s = 2(\varepsilon_1 - \varepsilon_3)/3 \quad (2-12)$$

where p' and q are the mean effective stress and deviator stress, ε_v and ε_s are the volumetric strain and shear strain.

By using these parameters, the critical state can also be defined as:

$$\frac{dp'}{d\varepsilon_s} = \frac{dq}{d\varepsilon_s} = \frac{d\varepsilon_v}{d\varepsilon_s} = 0 \quad (2-13)$$

In the basic form, CSSM is merely a linkage between compression and shearing by their stress state spaces. A single and unique line of failure points of both drained and undrained tests is defined as the critical state line (CSL) (Atkinson & Bransby, 1982). It is premised that any shearing will tend towards and eventually reach the critical state line (CSL) regardless of the initial state of the soil. The unique CSL can be presented in the ν - p' - q space.

The projection of the critical state line (CSL) onto the q - p' space can be modelled by a liner relationship, which represents the shearing behaviour of the soil:

$$q = Mp' \quad (2-14)$$

where M is its gradient. It represents the stress ratio at critical state and can be calculated as:

$$M = \frac{6\sin\phi_{cs}}{3 - \sin\phi_{cs}} \text{ for compression} \quad (2-15)$$

and

$$M = \frac{6\sin\phi_{cs}}{3 + \sin\phi_{cs}} \text{ for extension} \quad (2-16)$$

in which ϕ_{cs} is the angle of friction at critical state.

The consolidation response of the soil is shown in v - p' space. The projection of critical state line on the v - p' space is curved. However, if we plot the same data onto the v - $\ln(p')$ space, the CSL could be described by a linear relationship as following:

$$\Gamma = v + \lambda \ln(p') \quad (2-17)$$

where v is the specific volume ($v=1+e$); Γ is defined as the value of v corresponding to $p' = 1.0 \text{ kN/m}^2$ on the CSL.

The normal consolidation line (NCL) and swelling line (κ -line) could also be modelled on the v - $\ln(p')$ space:

$$v = N - \lambda \ln(p') \text{ for NCL} \quad (2-18)$$

$$v = v_{\kappa} - \kappa \ln(p') \text{ for } \kappa - \text{Line} \quad (2-19)$$

The meanings of Γ , λ , N , v_{κ} and κ are presented in Figure 2.17. Some typical values of these parameters for various clay soils are listed in Table 2.7 (after Schofield & Wroth, 1968).

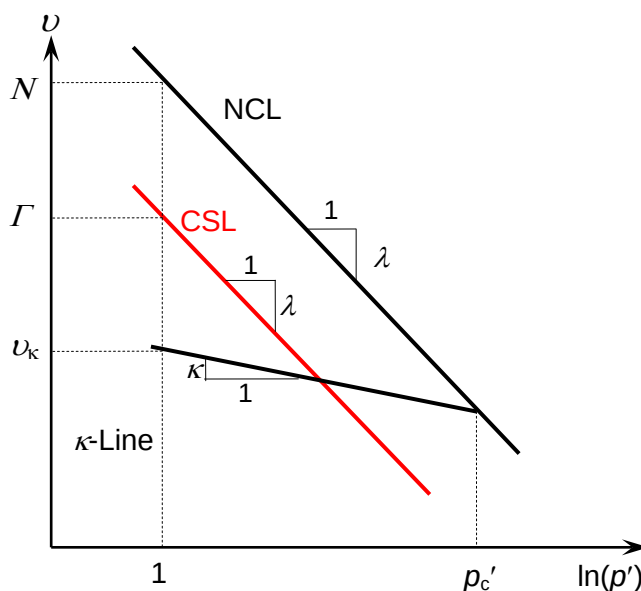
Figure 2.17 The CSL, NCL and κ -Line in v - $\ln(p')$ Spaces

Table 2.7 Values of Soil Constants for Various Clays (after Schofield and Wroth, 1968)

Properties		Klein Belt Ton	Wiener Tegel V	London Clay	Weald Clay	Kaolin
Critical state values	λ	0.356	0.122	0.161	0.093	0.26
	Γ	3.990	2.130	2.448	1.880	3.265
	v for $p = 100 \text{ lb/in}^2$	2.350	1.558	1.700	1.480	2.065
	M	0.845	1.01	0.888	0.95	1.02
Equivalent friction angle		$21\frac{3}{4}^\circ$	$25\frac{3}{4}^\circ$	$22\frac{1}{2}^\circ$	$24\frac{1}{4}^\circ$	26°
Average κ		0.184	0.026	0.062	0.035	0.05
$A = (\lambda - \kappa)/\lambda$		0.483	0.788	0.614	0.628	0.807

Note: the value of Γ is that for $p' = 1.0 \text{ lb/in}^2$

Traditionally, the critical state is typically related to drained behaviour of contractive specimens. Under undrained conditions, the pore water pressure varies while volume change of the specimen maintains constant. Therefore, a state in

which the soil mass is continuously deforming at constant volume, constant stress, and constant velocity has been defined as the steady state (Castro, 1969). The steady state is normally used in liquefaction analysis of very loose specimens under undrained conditions. The steady state is essentially the same as critical state and can be regarded as a special case of the critical state.

The uniqueness of critical state (or steady state) is a very important concept in strain softening and instability, which was intensively investigated by many researchers since 1980', such as Kuerbis et al. (1988), Vaid et al. (1990), Been et al. (1991), Castro et al. (1992), Negussey & Islam (1994), Verdugo & Ishihara (1996), Yamamuro & Lade (1998), Khalili et al. (2004), Ferreira & Bica (2006), Geiser et al. (2006), Yin & Chang (2009), Ng (2009). It should point out that majority of experimental studies on the uniqueness of CSL were conducted under axisymmetric stress conditions, and based on the response of NC/loose material. As conclude by several researchers (Vaid et al., 1990; Riemer & Seed, 1997; Yamamuro & Lade, 1998; Yin & Chang, 2009), the uniqueness of steady state appears to be dependent on shearing mode. By conducting drained compression tests under plane-strain conditions, Finno & Rechenmacher (2003) reported that the relationship between void ratio and the effective stress is not unique, but depends on consolidation history and initial state of sand. Wanatowski & Chu (2007) also suggested that the CSL established under triaxial conditions would not be in general applicable to the plane-strain conditions.

As summarised in Mayne et al. (2009), in consideration of work-energy models, the mathematical forms of CSSM could be given by the Original Cam-Clay (Schofield and Wroth, 1968), Modified Cam Clay (Roscoe & Burland, 1968; Wood, 1990), NorSand (Jefferies, 1993; Jefferies & Shuttle, 2005), and high-order constitutive relationships (e.g., Whittle & Kavvas, 1994; Pestana & Whittle, 1999).

2.4 Instability Behaviour of Soil

Instability in granular materials is defined as a condition in which large plastic strain are generated rapidly due to the inability of a soil element to carry or sustain a

given load (Lade et al., 1987, 1988; Lade & Pradel, 1990; Chu et al., 1993; Yamamuro & Lade, 1997; Lade, 1999). As emphasized by Wanatowski (2005), the instability of granular materials is not synonymous with failure, because it can occur inside the established failure surface (Lade, 1993, 1989 & 1999).

2.4.1 Instability Line and Instability Zones

Lade (1992) proposed an instability line by connecting the peak of effective stress path (i.e. peak q values) to separate the potentially unstable stress state from the stable stress state. It also marked the lower limit of the potential instability region, and defined the condition for the occurrence of pre-failure instability (as shown in Figure 2.18). The upper limit of the potential instability region is defined by the failure envelope.

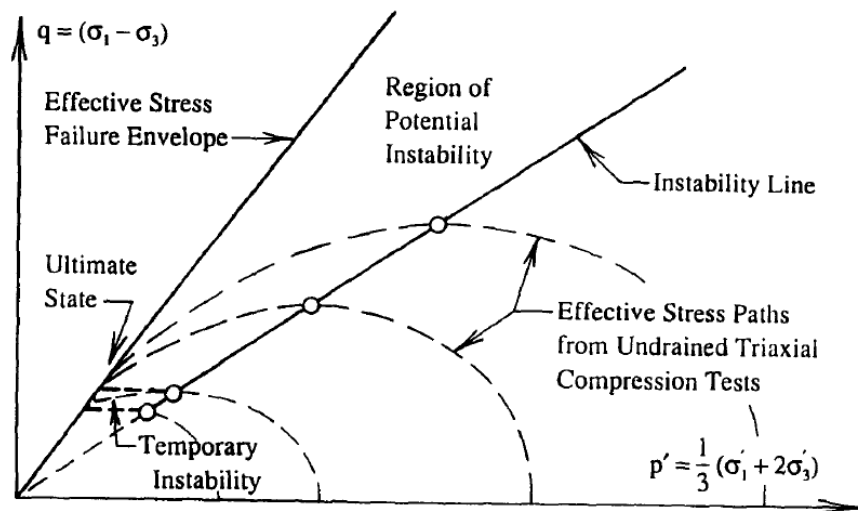


Figure 2.18 Schematic Diagram of Location of Instability Line in q - p' Space (after Lade, 1993)

As reported by Lade (1992, 1994, 1999), the granular materials are unable to sustain the current stress state when instability occurs. Such stress corresponds to the top of the current yield surface (as shown in Figure 2.19). It is also observed that top of effective stress path occurs slightly after, but very close to the top of yield surface. The difference is due to the additional stability provided by the elastic strains, as defined in Hill's postulate of instability (Lade, 1992, 1993, 1999). It is observed

that the granular materials become unstable when the state of stress located on or above the instability line. In the region above the instability line, the soil will deform plastically under decreasing stresses. Numerous experimental studies prove that the instability line is straight and passing through the origin of q - p' space (Lade, 1992, 1994, 1999; Leong et al., 2000; Chu et al., 2003).

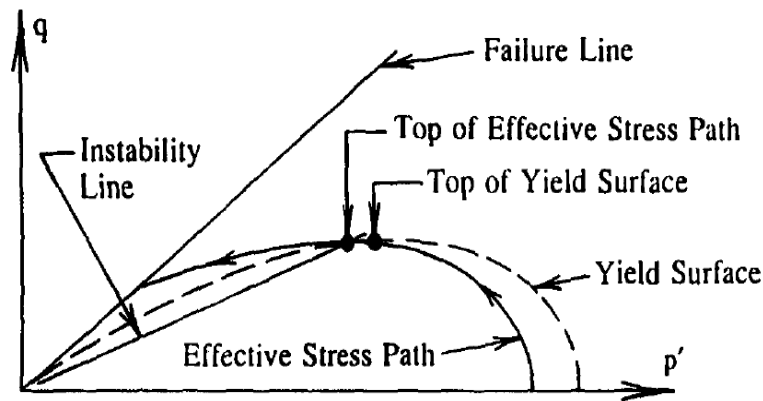


Figure 2.19 Location of Instability Line for Loose Sand (after Lade, 1992)

2.4.2 Previous Studies

It is important to point out that the instability of granular materials is different from the strain softening behaviour. Strain softening refers to the reduction in the shear resistance of a soil element with further shearing after the deviator stress reaches its peak value (Wanatowski, 2005). In contrast, the instability is defined as the inability of a soil material to carry or sustain a given load, resulting in large strains, which eventually lead to failure (Lade et al., 1987, 1988; Lade & Pradel, 1990; Chu et al., 1993; Yamamuro & Lade, 1997).

By conducting a comparative study on the strain softening behaviour and instability of sand, Chu & Leong (2001) reported that the onset of instability associated with a sudden collapse of soil specimen. However, during the strain softening, the deviator stress reduced gradually without observation of specimen collapse. It is also pointed out by Chu (1998) and Chu & Leong (2001), the strain softening occurs under

deformation-controlled loading conditions, but the instability occurs under load-controlled loading conditions. Through investigation on the effect of load mode on the instability behaviour of sand, Chu & Wanatowski (2009) suggested to study the instability behaviour under a load-controlled loading mode. This is consistent with previous studies carried out under axisymmetric conditions by Chu & Leong (2001) and Chu et al. (2003). As such, the constant-shear-drained (CSD) tests were proposed in this project to study the instability of residual soil in Singapore.

During the CSD test, the effective confining pressure is gradually reduced while the shear stress is held constant (Clough & Schmidt, 1981; Anderson & Riemer 1995; Anderson & Sitar 1995; Zhu & Anderson, 1998; Dai et al., 1999; Farooq et al., 2002; Futai et al., 2004), which leads to a horizontal movement of effective stress path towards the effective failure envelope (as shown in Figure 2.20, after Anderson & Reimer, 1995). A CSD test can be conducted by increasing the back pressure (or pore water pressure, PWP) while maintaining the confining pressure (σ_3) constant or by decreasing the confining pressure while maintain the back pressure (or PWP) constant. It should be pointed out that although the back pressure or PWP may change during CSD test, it is still considered as a “drained” test, since the volume of the specimen is changing. Several studies (Anderson & Riemer, 1995; Leong, 2001; Wanatowski, 2005) have proved that there is no difference between the two types of testing methods.

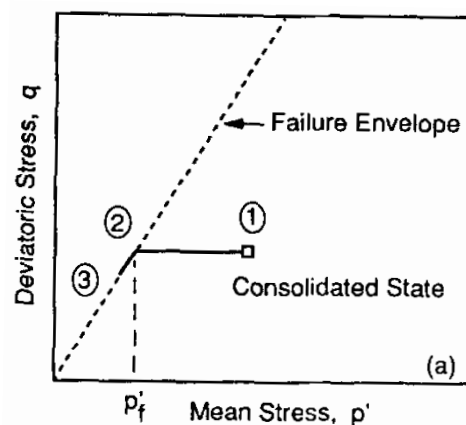


Figure 2.20 Effective Stress Path Determined from CSD Test (after Anderson & Reimer, 1995)

Majority of experimental studies on CSD tests were conducted on sandy soils (Zhu & Anderson, 1998; Dai et al., 1999; Farooq et al., 1999, Chu et al., 2003). The CSD test results were normally compared with other conventional tests, such as anisotropically consolidated undrained (CAU) tests, isotropically consolidated drained (CID) or undrained (CIU) tests. It is reported by Zhu & Anderson (1998), the shear strength and soil behaviour of Hawaiian residual soil were substantially different between CAU and CSD tests. Abrupt failure was observed in CSD tests after reaching the failure envelope, which may be caused by the instability behaviour of tested material.

Based on a study conducted on a loosely compacted volcanic-derived soil, Dai et al. (1999) concluded that the critical states determined from CID, CIU and CSD tests are consistent. It indicated that the testing methods used have little influence on location of the critical state line. Dai et al. (1999) also reported that, for compressive soils, the increase pore water pressure generated due to the infiltration of water led to a compressive failure in drained manner. However, 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.

Chu et al. (2003) carried out a study on the instability of sand by conducting CSD tests. The instability behaviour observed in the tests was interpreted using a concept similar to the state parameter concept proposed by Been & Jefferies (1985). A new framework formulated using the modified state parameter $\bar{\psi}$ and the critical state line (CSL) has been proposed to describe the instability conditions of both contractive and dilative sand. The framework is also favorable to investigate the failure mechanism of loose and dense sand slope. The modified state parameter is defined as:

$$\bar{\psi} = e_{IL} - e_{cr} \quad (2-20)$$

where e_{IL} is the void ratio at the instability point and e_{cr} is the corresponding void ratio at the critical state at the same mean effective stress as e_{IL} .

Based on the experimental results, Chu et al. (2003) reported that instability occurs under both drained and undrained conditions for contractive sand. Under drained conditions, the onset of instability is a conditional type, which only occurs along a stress path with reduction in mean effective stress. In contrast, the instability under undrained conditions is a run-away type (referred to as pre-failure instability). For dilative sand, conditional instability can occur under drained conditions along a stress path with a reduction in mean effective stress, when the stress state is above the instability line. It is also reported by Chu et al. (2003) the instability line is not unique as critical state line, but varies with void ratio and the stress level of the tested marine sand.

To investigate the behaviour of loosely compacted decomposed granite (silty sand), Ng et al. (2004) conducted four series of triaxial compression tests included isotropically consolidated drained and undrained (CID and CIU) tests, anisotropically consolidated undrained (CK₀UC) tests and anisotropically consolidated constant (deviator) stress (CSD) tests. During the CSD tests, a structural collapse behaviour characterized by an abrupt development in both axial and contractive volumetric strain was observed by Ng et al. (2004). This is consistent with the instability behaviour of granular materials.

Although a number of studies have been carried out on the occurrence of instability, most of them were conducted on sand, except Zhu & Anderson (1998). The instability behaviour of other granular materials, such as residual soils should also be investigated. Further research under both axisymmetric and plane-strain conditions was carried out in this study.

2.5 Yield of Bonded Soils

Yielding was defined by Wood (1990) as “the departure from stiff elastic response that occurs as reloading proceeds beyond the past maximum load”. According to Smith et al. (1992), it is always associated with a sharp change from elastic to plastic behaviour. However, in the historical development of soil mechanics, the traditional “yield” is used to describe a clear change in stress-strain behaviour

(Malandraki & Toll, 1996). Vaughan (1985, 1990) defined a yield stress as “a stress state at which a material shows a discontinuity in stress-strain behaviour under monotonic stress changes”.

The term “structure” or “bond” refers to both fabric, including particle association and arrangement, and interparticle forces, which are not purely frictional in nature (e.g. cementation, electrical attraction) (Lambe & Whitman, 1969; Zhang et al., 2004). As pointed out by Mitchell (1993), soil properties depend directly upon the mineralogical and chemical compositions, the current state of structure, and the historical effects of the environment. Yield of structure is demonstrated by an irreversible post-yield change in the stiffness and strength of the material. Vaughan (1985, 1990) also pointed out that the yield stress is strain-rate dependent, and yield stress increases with strain rate. Numerous experimental studies have shown that yield in soft clay is easily defined, even in isotropic compression (Mitchell, 1970; Lo & Morin, 1972; Sangrey, 1972; Crooks & Graham, 1976; Tavenas & Leroueil, 1977; Larsson, 1981; Graham et al., 1983; Leroueil et al., 1985; Tavenas et al., 1978; Tavenas & Leroueil, 1985). Evidence of yielding of soft clays under embankment is also shown by Pelletier et al. (1979) and Kabbaj et al. (1988). The yielding of weak rocks was also reported by Adachi & Takase (1981).

As pointed out by Vaughan et al. (1988), yield of residual soil under increasing stress occurs in a manner similar to the yield of over-consolidated sedimentary soil, however, it is due to bonding, not to stress history. Stress history has a negligible influence on current properties.

According to the definition of yield given by Vaughan (1985, 1990), there is a possibility for more than one “yield”. Multi-yielding has been observed for residual soil (Vaughan, 1988), artificially bonded soil (Maccarini, 1987; Bressani, 1990), artificially cemented carbonate sand (Coop & Atkinson, 1993) and soft rocks (Leddra et al., 1990; Anagnostopoulos et al., 1991; Kavvas et al., 1993, 1994).

Vaughan (1985) suggested that yield is best identified from the stress-strain curve plotted to a natural or log-log scale. Two yields were observed in his study of behaviour of residual soil (Vaughan, 1988). The first yield was defined at the stress level when bonds start to breakdown. A more significant yield (second yield) occurred when the stress applied achieves its bond strength. Vaughan et al. (1988) stated that second yield does not coincide with the complete breakdown of the bonds, which occurs at much larger strains.

Stress-strain curves and variations of stiffness determined from stress-strain curves are most common methods used to define the yielding points. It should point out that among the studies mentioned above, Leddra et al. (1990) identified two yield points based on the changes of K_0 value in a one-dimensional compression test. In this case the first point reflects a first loss in stiffness and the second point reflects a major loss in stiffness.

Based on the experimental results on compacted soil, Jardine et al. (1991) and Jardine (1992) divided the stress space into three zones, with the following boundary conditions:

- Y1 represents the limit of linear elastic zone. Theoretically, no such region exists for granular materials (Bowden & Tabor, 1964). If the zone exists, it is very small for soils. As observed by Lee (1985), Jardine (1985), the soil behaviour at very small strains is likely to be anisotropic.
- Y2 represents the limit of recoverable zone. Based on the tests results on several soils, Jardine et al. (1991) and Jardine (1992) reported that the stress-strain behaviour is non-linear in this zone, but it is fully recoverable.
- Y3 represents the limit of plastic zone, in which, irrecoverable strains developed. Any structure within the soil was destructed completely in zone 3. No strong tendency of dilation or contraction was observed for soils in this zone. The stiffness dropped to comparatively low values by now.

It is important to highlight that the zones represented by Y1 and Y2 are kinematic, and move according to the current stress path direction. In contrast, Y3 is static and independent of stress history (Jardine et al., 1991).

By studying the behaviour of an artificially bonded soil, Toll & Malandraki (1993) proposed a method to identify yielding point from the changes in the tangential stiffness (E_{tan}) versus axial strain curve on log-log scale. After comparing with the studies conducted by Maccarini (1987) and Bressani (1990), it was observed that the yield may be overestimated from the stress-strain curve plotted to a natural or log-log scale. Substantial amount of stiffness has already been lost at the first yield point defined by Maccarini (1987) and Bressani (1990). As stated by Malandraki & Toll (1996), the second yield point determined from the E_{tan} versus axial strain curve (log-log scale) coincided with the definition of “first yield” from Anagnostopoulos et al. (1991), Kavvadas et al. (1993, 1994) and “yield” from Coop & Atkinson (1993). The variation of the tangential stiffness provided a more accurate and simpler method to define yielding point, since the initiation of major loss in stiffness is distinctive.

The failure modes of bonded soils under triaxial compression conditions have been studied since 1970's by many researchers, such as Sangrey, 1972; Saxena & Lastrico, 1978; Sitar, 1979; Clough et al., 1981; Maccarini, 1978; Bressani, 1990; Anagnostopoulos et al., 1991. At a low confining pressure, the bonded soil reaches higher strength than unstructured soil, and yield (final yield) coincides with failure. In contrast, at high confining pressure, yield occurs before failure, and the soil's strength is governed by that of the unstructured soil. After conducting a series of triaxial tests on carbonate sands, Coop & Atkinson (1993) observed a transitional failure mode when shearing at intermediate stress level. Malandraki & Toll (1996) verified their observation by testing an artificially bonded soil and a de-structured soil at different confining stress levels. Based on the test results on the artificially bonded soil (Malandraki, 1994), Malandraki & Toll (1996) identified four zones of behaviour from the relative positions of the failure surface and the second yield surface (as shown in Figure 2.21).

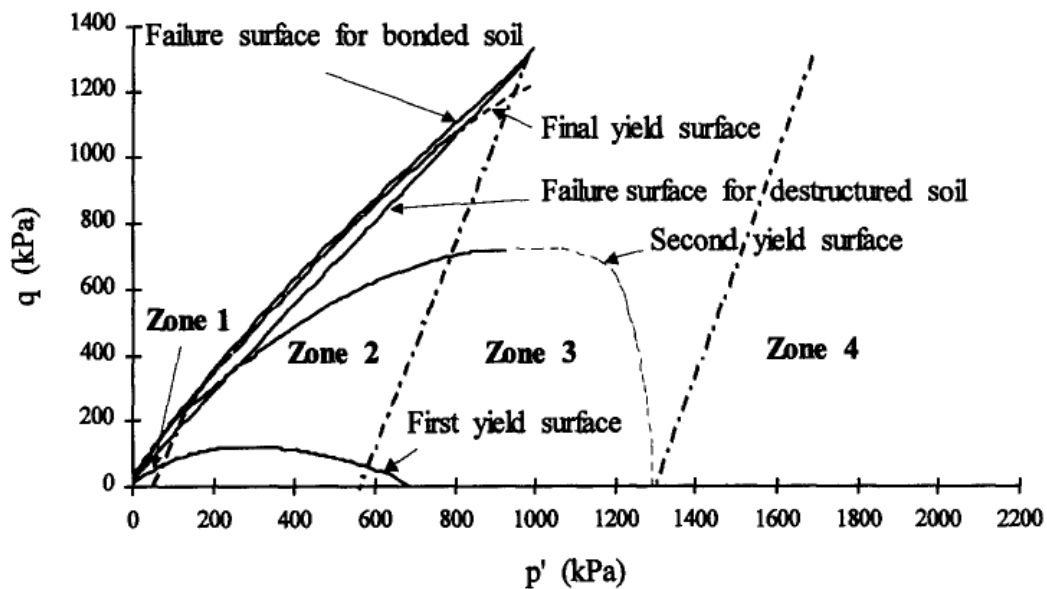


Figure 2.21 A Conceptual Picture of Yield for Bonded Structured Materials (after Malandraki & Toll, 1996)

In Zone 1, the soil strength is completely controlled by bonds. The failure surface coincides with the second yield surface. In Zone 2 (transitional zone), the bonding partially controls the soil strength. The bonded soil reaches a higher failure surface than de-structured soil, due to the post-yield influence of the bonded structure. Since the post-yield influence reduces as effective confining stress increases, the slope of the failure surface for boned soil reduces gradually. In Zone 3, the soil bonds breakdown completely before failure. The soil strength is independent of bonding. In Zone 4, the second yield occurs in isotropic compression, the soil strength is independent of bonding.

To date, extensive studies have been carried out to investigate the yield behaviour of bonded soils. Various methods were proposed for yield point determination. Majority of these investigations were conducted on artificially bonded soils, cemented sand and weak rocks. Limited amount of publications were found on residual soils (Vaughan, 1985, 1988). Almost all the experimental data reported in the literatures were obtained from shearing stage, except Jardine et al. (1991). Further investigation on the yield behaviour of a residual soil during K_0

consolidation was carried out in this study. Yield points were determined for a residual soil under both axisymmetric and plane-strain conditions.

2.6 Hydraulic Fracture

Hydraulic fracturing is a common technique used in petroleum engineering. It was firstly used to create deep-penetrating fractures in hydrocarbon reservoirs to simulate the production of oil and gas wells in the United States in 1947 (Howard & Fast, 1970). The importance of hydraulic fracture was identified in 1970's, since the in-situ outflow permeability was overestimated due to the hydraulic fractures, which was generated under the high operation pressure (Bjerrum et al., 1972). The term "Hydraulic Fracturing" was defined as "the condition leading to the creation and propagation of a thin physical separation in a soil or rock mass due to high fluid pressure" by the Independent Panel to Review the Cause of the Failure of Teton Dam (Jaworski et al. 1981).

The investigation of hydraulic fracture was focused on preventing it from occurring in engineering applications, such as pressure grouting, dam design, in-situ permeability testing and pile driving. Hydraulic fracturing is the known or suspected cause of the excessive leakage and failure of many dams, such as Balderhead Dam in Britain (Vaughan et al. 1970), Wister Dam and Yard's Creek Dam in the United States (Sherard et al. 1972), Hyttejuvet Dam in Norway (Wood et al. 1976), Viddalsuatu Dam in Norway (Vestad, 1976), Manicouagan Dam in Canada (Sherard, 1987) and Teton Dam in the United States (Jaworski, 1981). These cases showed that the potential risk of hydraulic fracturing is an important issue in dam design. To date, most of the existing laboratory experiments and field tests focused on studying the fracture initiation pressure (e.g. Jaworski, 1981; Mori et al., 1987; Panah & Yanagisawa, 1989; Andersen et al., 1993 and Yanagisawa & Panah, 1994). Different theories of the mechanics of failure have been proposed for soils subjected to internal borehole fluid pressure. These theories were summarized by Alfaro & Wong (2001) as shown in Table 2.8.

Table 2.8 Approaches Used in Evaluating Fracture-Initiation Pressure (Alfaro & Wong, 2001)

Theory or approach	Equation	Definition of terms	Reference
Cavity-expansion theory	[1] $P_f = \sigma_h + c \left[1 + \ln \left(\frac{G}{c} \right) \right]$	c = cohesion of soil G = shear modulus σ_h = horizontal confining pressure	Carter et al. 1986
Elastic theory	[2] $P_f = \frac{(1 + \sin \phi) \sigma_h + c \cos \phi \left(1 - \frac{r_i^2}{r_h^2} \right)}{1 + \frac{r_i^2}{r_h^2} \sin \phi}$	ϕ = angle of internal friction of soil r_i = radius of injection well r_h = radius of cylinder being considered	Panah and Yanagisawa 1989
	[3] $P_f = \sigma_h + c$	for negligible value of ϕ	
Empirical formula	[4] $P_f = m \sigma_h + \sigma_t$	m = empirical factor σ_t = tensile strength of soil	Mori and Tamura 1987
Tensile failure mechanism	[5] $ \sigma_3 - u \geq \sigma_t$	u = pore pressure σ'_3 = effective minor principal stress	Andersen et al. 1994
	[6] $\Delta u = B \Delta \sigma_m + D(\Delta \sigma_r - \Delta \sigma_\theta)$	$\Delta \sigma_m$ = change in total mean normal stress $\Delta \sigma_r$ = change in radial stress $\Delta \sigma_\theta$ = change in tangential stress B and D = pore-pressure parameters	
Linear elastic fracture mechanics*	[7] $P_f = \frac{K_{Ic}}{\sqrt{\pi a}} + \sigma_n$	K_{Ic} = fracture toughness a = half-width of initial fracture slot σ_n = stress normal to the plane of fracture	Murdoch 1992a

* For wells with an initial fracture slot.

Since Bjerrum et al. (1972), laboratory experiments on hydraulic fracturing in granular materials by pressurizing a borehole/injection well have been conducted by Nobari et al. (1973); Jaworski et al. (1981), Mori & Tamura (1987), Panah & Yanagisawa (1989), Lo & Kaniaru (1990), Andersen et al. (1993), Murdoch (1993), Vallejo (1993), Atkinson et al. (1994), Yanagisawa & Panah (1994), Reed (1997) and Alfaro & Wong (2001). Researches on hydraulic fracture were carried out for both unsaturated (e.g. Fang, 1989; Harison et al., 1994; Murdoch, 1993 and 2000) and fully-saturated soils (e.g. Lo & Kaniaru, 1990; Murdoch, 1993; de Pater et al., 2003; Soga et al., 2004 and Ohta et al., 2007). These studies showed that hydraulic fractures can be created in unsaturated as well as in fully-saturated soils. In addition to laboratory experiments, field tests on hydraulic fracturing in soil have also been performed by many researchers, such as Bjerrum et al. (1972), Lefebvre (1991), Reed (1993), Murdoch (1985, 1995 and 2002), and Hocking (1996).

Tensile and shear failure hypotheses of the hydraulic fracture criteria are most commonly proposed in many publications. Numerous researchers (Bjerrum et al., 1972; Massarsch, 1978; Jaworski, 1981; Lo & Kaniaru, 1990; Andersen et al., 1994;

Vallejo, 1993; Yanagisawa & Panah, 1994; Ohta et al., 2007) believed that hydraulic fracturing is associated with tensile failure. It is commonly accepted in the literature that hydraulic fracturing occurs when the minor principal effective stress σ_3' becomes negative (tensile) with a magnitude exceeding the tensile strength (σ_t) of the soil (Andersen et al., 1994), i.e. when

$$\sigma_3' + \sigma_t \leq 0 \quad (2-21)$$

Based on the experimental results, Ohta et al. (2007) verified that the hydraulic fracturing is triggered by the pore water pressure higher than minor principal stress σ_3 . It indicated that the hydraulic fracturing take place in the laboratory test when the effective minor principal stress becomes negative. This is consistent with Kennard et al. (1967), Seed and Duncan (1981) and Murdoch (1992) who gave $\Delta u \geq \sigma_3$ as the condition for the occurrence of hydraulic fracturing.

Some important equations used to determine fracture-initiation pressure under tensile failure are summarized in Table 2.9. Among the studies listed in Table 2.9, it is necessary to highlight the one recently carried out by Zhang et al. (2008) with three dimension stress state. Two fracture-initiation pressure calculating formulas were proposed by the authors, based on tensile failure mechanism and shear failure mechanism, respectively. With further investigation on the suitability of the proposed two calculating formulas, Zhang et al. (2008) conclude that soil fracturing based on shear failure occurs easily than the tensile failure in the case of high minor principal stress and (or) small soil shear strength, whereas fracturing is governed by the tensile failure mechanism in the case of lower minor principal stress and (or) lager soil shear strength. The corresponding equation for fracture-initiation pressure under shear failure mechanism is presented in Table 2.10.

Table 2.9 Evaluation Methods for Fracture-Initiation Pressure under Tensile Failure

Equations	Definition of Terms	References
$P_f = m\sigma_h + \sigma_t$	m = empirical factor σ_t = tensile strength of soil	Vaughan (1971)
$P_f = \sigma_h + \frac{\sigma_t}{2}$	σ_t = tensile strength of soil	Jaworski (1979)
$P_f = 1.3\sigma_h + 1.7\sigma_t$	σ_t = tensile strength of soil	Seed & Duncan (1981)
$P_f = (1 + \sin\phi)\sigma_h + c\cos\phi$	c = cohesion of soil ϕ = angle of internal friction	Camberfort (1981)
$P_f = c\cos\phi + \frac{(1 + \sin\phi)\sigma_t}{2} + (1 - \sin\phi)\sigma_h$	c = cohesion of soil ϕ = angle of internal friction	Lo et al. (1990)
$P_f = 3\sigma_3 - \sigma_2 + \sigma_t$	σ_t = tensile strength of soil	Zhang et al. (2008)

The shear failure mechanism is the other most commonly suggested hydraulic fracture criteria. It was firstly proposed by Morgenstern & Vaughan (1963) based on a study on allowable grouting pressure. Mori & Tamura (1987) observed horizontal and inclined fractures in laboratory experiments. They believed that the observation indicated the hydraulic fracture of cohesive soils is not necessarily initiated by tensile failure. They also suggested that the initiation pressure of hydraulic fracturing P_f should be calculated as follow, irrespective of the direction of fracture surface,

$$P_f = \sigma_{min} + q_u \quad (2-22)$$

where σ_{min} is the minimum principal stress, q_u is the unconfined compression strength.

By analyzing the empirical formula of the fracture-initiation pressure determined from laboratory test results, Panah (1989) concluded that hydraulic fracture was

resulted from shear failure rather than tensile failure. Cater et al. (1986), Mori et al. (1987) and Panah (1989) all assumed that soil yields according to the Mohr-Coulomb criterion.

Some important equations proposed to determine fracture-initiation pressure under shear failure are summarized in Table 2.10.

Table 2.10 Evaluation Methods for Fracture-Initiation Pressure under Shear Failure

Equations	Definition of Terms	References
$P_f = \sigma_h + c \left[1 + \ln \left(\frac{G}{c} \right) \right]$ for cylindrical cavity	c = cohesion of soil G = elastic shear modulus	Cater et al. (1986)
$P_f = \sigma_h + \frac{4c}{3} \left[1 + \ln \left(\frac{G}{c} \right) \right]$ for spherical cavity	c = cohesion of soil G = elastic shear modulus	Cater et al. (1986)
$P_f = \sigma_{min} + q_u$	σ_{min} = minimum principal stress q_u = unconfined undrained strength	Mori & Tamura (1987)
$P_f = \frac{(1 + \sin \phi) \sigma_h + c \cos \phi (1 - b^2/a^2)}{1 + (b^2/a^2) \sin \phi}$	c = cohesion of soil ϕ = angle of internal friction a = internal radius of sample b = external radius of sample	Panah & Yanagisawa (1989)
$P_f = \frac{1}{2} (3\sigma_3 - \sigma_2) (1 + \sin \phi) + c \cos \phi + \frac{\sigma_t}{2} (1 + \sin \phi)$	σ_t = tensile strength of soil ϕ = angle of internal friction c = cohesion of soil	Zhang et al. (2008)

Although a number of studies have been carried out on hydraulic fracture, the investigation was focused on preventing it from occurring in engineering applications. In laboratory borehole fracturing tests, hydraulic fracturing is

generally believed to initiate when volumetric strain rate increased abruptly with a constant increment in back pressure (Bjerrum et al., 1972; Bjerrum & Andersen, 1972; Jaworski et al., 1981; Mori & Tamura, 1987; Alfaro & Wong, 2001; Ohta et al., 2007). Numerous equations were proposed for the calculation of fracture-initiation pressure based on tensile failure mechanism and shear failure mechanism. In the research project presented in this thesis, the CSD test performed to investigate the instability behaviour of residual soils could also be used as an element study, to understand the mechanism behind hydraulic fracture.

2.7 Summary

A literature review was presented in this chapter, especially on the engineering properties of soil, instability of granular materials and yield of bonded soils. Although significant progress in experimental and analytical research has been made, the residual soil was not commonly used in these studies. As pointed out by many researchers, the engineering behaviour of residual soils is significantly influenced by the progress of weathering process, the presence of bonded structures and fabric, and also by the depth of penetration of which will be influenced by the discontinuities in the parent rock (Leroueil & Vaughan, 1990; Maccarini, 1993; Malandraki & Toll, 1994; Hight & Leroueil, 2003; Viana da Fonseca, 2003). Therefore, further study should be carried out on intact residual soils, especially large block of intact residual soils, which may provide specimens with much better quality than “intact” borehole samples. The study presented in this thesis will be concentrated on the saturated residual soil behaviour.

Shear strength is one of the most important and most complex engineering properties of soil. As pointed out by many researchers (Ladd, 1986; Kulhawy & Mayne, 1990; Chu et al., 1999), the undrained shear strength, c_u , is not a constant, but varies with many factors, including effective overburden stress σ_{v0}' , OCR, consolidation path (isotropic or K_0 consolidation), stress states and stress anisotropy. Based on the experimental data from Hight et al. (1992, 2003), Mayne et al. (2009) concluded that at the c_u values might range up to a factor of six at a given depth, depending on the testing method adopted. This is particularly the case for residual

soil as it is highly anisotropic in strength and deformation behaviour. A well-established relationship between (c_u/σ'_{10}) and OCR is normally used to determine undrained shear strength ratio. The relationship has also been verified for sedimentary soils by numerous experimental data (Ladd, 1986; Kulhawy & Mayne, 1990; Chu et al., 1999). However, whether this is the case for residual soil needs to be investigated. The effect of anisotropy on the shear strength should also be further studied.

Instability of loose granular materials under undrained conditions has been well understood and established (Lade et al., 1988; Lade & Pradel, 1990; Lade, 1992, 1993, 1994; Leong, 2001; Chu et al., 2003). It has also been demonstrated by many researchers that the granular materials can become unstable under drained or strain path controlled conditions (Lade et al., 1987, 1988; Chu, 1991; Chu et al., 1993, 2003; Chu & Leong, 2001 and Loke, 2004). Although a number of studies have been focused on the occurrence of instability, most of them were conducted on sand, except Zhu & Anderson (1998). The instability behaviour of other granular materials, such as residual soils should also be investigated. Through investigation on the effect of load mode on the instability behaviour of sand, Chu & Wanatowski (2009) suggested to study the instability behaviour under a load-controlled loading mode. This is consistent with previous studies carried out under axisymmetric conditions by Chu & Leong (2001) and Chu et al. (2003). As such, the constant-shear-drained (CSD) tests were proposed in this project to study the instability of residual soil in Singapore. Further research under both axisymmetric and plane-strain conditions was also carried out.

Yield of structure is demonstrated by an irreversible post-yield change in the stiffness and strength of the material. Vaughan (1985, 1990) defined a yield stress as “a stress state at which a material shows a discontinuity in stress-strain behaviour under monotonic stress changes”. Numerous experimental studies have shown that yield in soft clay is easily defined, even in isotropic compression (Mitchell, 1970; Lo & Morin, 1972; Sangrey, 1972; Crooks & Graham, 1976; Tavenas & Leroueil, 1977; Larsson, 1981; Graham et al., 1983; Leroueil et al., 1985; Tavenas et al.,

1978; Tavenas & Leroueil, 1985). According to the definition of yield given by Vaughan (1985, 1990), there is a possibility for more than one “yield”. Different analysing methods were proposed to determine the location of yield points. Multi-yielding has been observed for residual soil (Vaughan, 1988), artificially bonded soil (Maccarini, 1987; Bressani, 1990; Toll & Malandraki, 1993; Malandraki & Toll, 1996), artificially cemented carbonate sand (Coop & Atkinson, 1993) and soft rocks (Leddra et al., 1990; Anagnostopoulos et al., 1991; Kavvadas et al., 1993, 1994). To date, extensive studies have been carried out to investigate the yield behaviour of bonded soils. Majority of these investigations were conducted on artificially bonded soils, cemented sand and weak rocks. Limited amount of publications were found on residual soils (Vaughan, 1985, 1988). Almost all the experimental data reported in the literatures were obtained from shearing stage, except Jardine et al. (1991). The yield behaviour during anisotropic consolidation could also be investigated.

Hydraulic fracturing is a common technique used in petroleum engineering. Its importance in geotechnical engineering was identified in 1970's, since the in-situ outflow permeability was overestimated due to the hydraulic fractures, which was generated under the high operation pressure (Bjerrum et al., 1972). Currently, the investigation of hydraulic fracture was to prevent it from occurring in engineering applications. Most of the existing laboratory experiments and field tests focused on studying the fracture initiation pressure (e.g. Jaworski, 1981; Mori et al., 1987; Panah & Yanagisawa, 1989; Andersen et al., 1993 and Yanagisawa & Panah, 1994). Different theories of the mechanics of failure have been proposed. Among those, tensile failure (Bjerrum et al., 1972; Massarsch, 1978; Jaworski, 1981; Lo & Kaniaru, 1990; Andersen et al., 1994; Vallejo, 1993; Yanagisawa & Panah, 1994; Ohta et al., 2007; Zhang et al., 2008) and shear failure (Morgenstern & Vaughan, 1963; Mori & Tamura, 1987; Panah, 1989; Mori et al., 1987; Panah, 1989; Zhang et al., 2008) hypotheses of the hydraulic fracture criteria are most commonly suggested in many publications. Although a number of studies have been carried out on hydraulic fracture, the investigation was focused on the determination of fracture-initiation pressure based on tensile failure mechanism and shear failure

mechanism, which are studies on the boundary condition. By conducting laboratory borehole fracturing tests, hydraulic fracturing is generally believed to initiate when volumetric strain rate increased abruptly with a constant increment in back pressure (Bjerrum et al., 1972; Bjerrum & Andersen, 1972; Jaworski et al., 1981; Mori & Tamura, 1987; Alfaro & Wong, 2001; Ohta et al., 2007). In the research project presented in this thesis, the CSD test performed to investigate the instability behaviour of residual soils could also be used as an element study, to understand the mechanism behind hydraulic fracture. The onset of instability and initiation of hydraulic fracture could be related.

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 Introduction

This chapter describes the properties of the soils used in the study. The methodologies adopted to prepare the specimens and conduct various tests are also discussed.

3.2 Soils for Testing

3.2.1 Intact Residual Soil

In order to determine the undrained shear strength of a residual soil in Singapore, a study was carried out as a joint research project between NTU and the Singapore Branch of Ed. Züblin AG, a German based construction company. Large blocks of intact residual soil samples were taken from a construction site of T-06 (Queensway Tunnel) of the Deep Tunnel Sewerage System project in Singapore. The samples were retrieved from the depth of 19.4m of the residual soil layer that belongs to the formation of weathered Bukit Timah granite by carefully cutting the soil using a cutter. These blocks samples were immediately wrapped by two layers of plastic sheet and one layer of aluminum foil and stored into plastic boxes. The position and orientation of the block samples were recorded. The boxes were then sealed, transported to the laboratory and stored into a humidity control room.

The ground water table was 2.1 m below the ground level. The in-situ effective stress was 195 kPa. The photos of sample blocks were presented in Figure 3.1(a)

and (b). The basic soil properties of the intact residual soil samples are listed in Table 3.1. It was sandy clay consisting of about 50% fines and 50% sand.

Table 3.1 Basic Properties of Intact Residual Soil

Tests	Results
Bulk Density	1.8-1.93 Mg/m ³
Liquid Limit	42.86 %
Plastic Limit	24.51 %
Plastic Index	18.35 %
Fines Content	50%
Specific Gravity	2.693
Water Content	31%
In-situ OCR	1.54
Permeability	10 ⁻⁹ m/s
N Value	31



(a)

(b)

Figure 3.1 Block Intact Residual Soil Samples

The intact residual soil sample of weathered Bukit Timah granite was soaked into distilled water to study whether there was any cementation reflected on the shear

strength. The sample disintegrated only a few minutes after distilled water was poured into the container, and dissolved completely after 15 minutes. Picture taken at different time intervals are presented in Figure 3.2

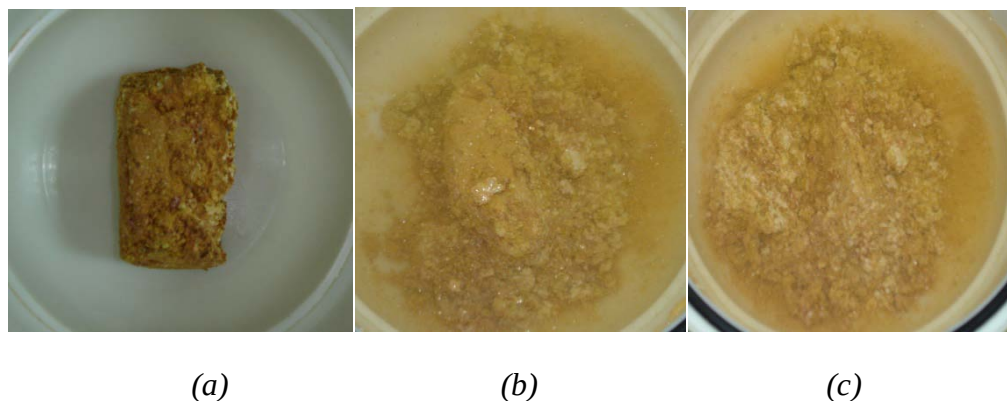


Figure 3.2 Cementation of Intact Residual Soil of Weathered Bukit Timah Granite:

(a) Without Water; (b) 5 Minutes; (c) 15 Minutes

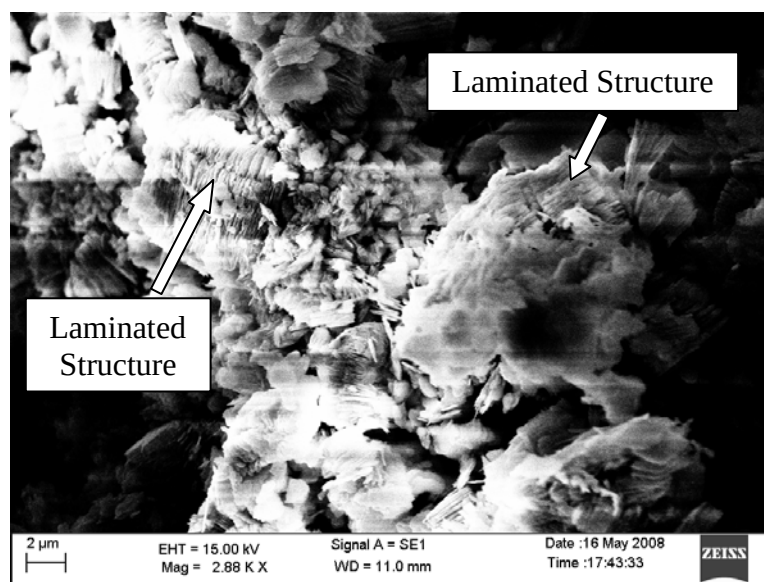


Figure 3.3 SEM Image of Intact Residual Soil of Bukit Timah Granite

The microstructure of intact residual soil of Bukit Timah granite was studied with the aid of a scanning electron microscope (SEM). An example of SEM images was given in Figure 3.3. With the scale of $2.0\mu\text{m}$ provided by SEM, laminated microstructure was observed clearly, which indicated anisotropy may occur for intact residual soil of Bukit Timah granite. To investigate the anisotropic behaviour of the soil, block samples were cut in both horizontal and vertical direction before tests, as

shown in Figure 3.4. The samples then were properly wrapped with plastic sheets and aluminum foils. The wrapped samples were sealed by waxing and kept in a moist room to maintain the water content constant. The water content of sample was 31%.

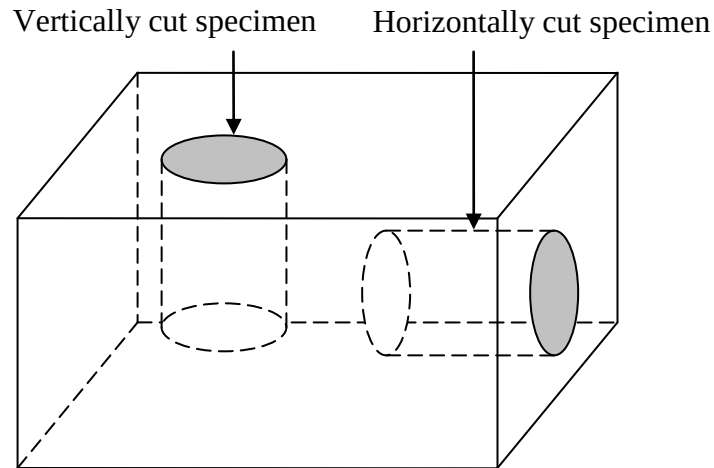


Figure 3.4 Cutting Orientation of Undisturbed Soil Specimen

3.2.2 Compacted Residual Soil Specimens

The compacted residual soil specimens were made using trimmings of the intact residual soil of Bukit Timah granite described in Section 3.2.1. Both standard proctor tests and static compaction tests were conducted. The compaction curves determined are compared in Figure 3.5. Since a static compression machine was used to prepare compacted residual soil specimen (as shown in Figure 3.6), the maximum dry density of 1.54 Mg/m^3 and water content of 24% (at maximum dry density) was adopted for sample compaction.

Distilled water was added into oven-dried residual soil to achieve a predetermined water content of 24%. The wet mixture was kept in a plastic zip bag to cure overnight before compaction. A two-part split mould (as shown in Figure 3.7) was used to compact the cured soil mixture into desired dimensions. The compacted residual soil specimen was compacted in layers. The weight of wet mixture for each layer was calculated based on the dry density (1.54 Mg/m^3) and water content (24%). The compaction of each layer was controlled by the thickness of specimen. For example, the plane-strain specimen ($60 \times 60 \times 120 \text{ mm}$) was compacted in three

layers. Compaction stopped when each layer reached a thickness of 40mm. To enhance the contact between each layer, the top surface of the first two layers were scratched before pouring in the soil mixture for the next layer. Once the compaction process was done, the specimen was extruded from the mould and set up for triaxial / plane-strain test immediately. The experimental set-up will be discussed in detail in Section 3.3.

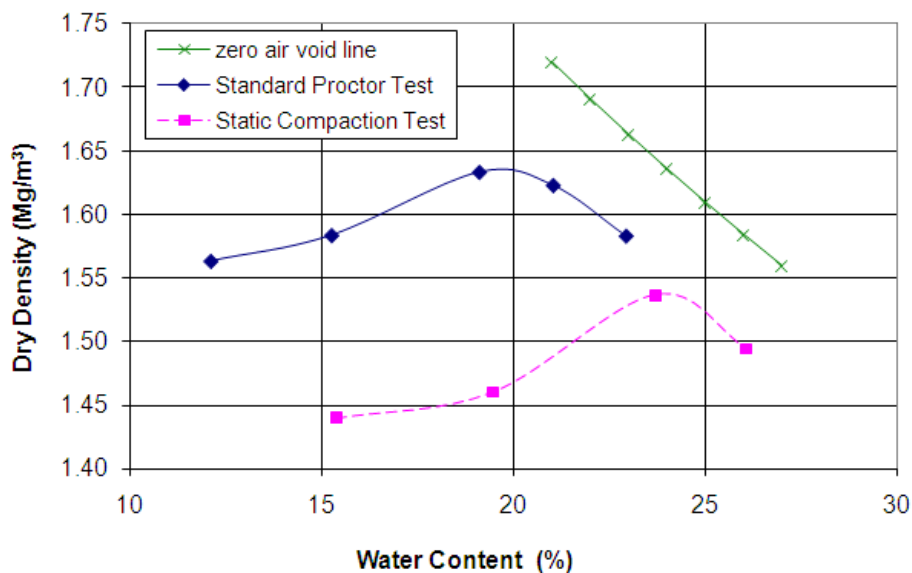


Figure 3.5 Compaction Curves



Figure 3.6 Static Compression Machine



(a)

(b)

Figure 3.7 Two-Part Split Aluminium Mould: (a) Cylindrical Triaxial Mould; (b) Prismatic Plane-Strain Mould



(a)

(b)

Figure 3.8 Compacted Residual Soil Specimens: (a) Triaxial Specimen; (b) Plane-Strain Specimen

3.2.3 Artificially Cemented Soil Specimens

An artificially cemented soil was tested to compare with the study on intact residual soil of Bukit Timah granite. It was prepared by using a method similar to that described by Maccarini (1987), Malandraki (1994) and Rahman (2008).

To achieve a fines content of 50% as the intact residual soil, 50% kaolin powder was mixed with 50% oven-dried Changi sand. The sand-kaolin mixture was mixed

initially without water. Distilled water was added later (about 15% of water content), to allow the wet kaolin to attach around the sand particles uniformly. The wet mixture was stirred thoroughly with a spatula, and then sealed for an overnight curing. The artificially cemented soil specimens were compacted to achieve a dry density of 1.37-1.47 Mg/m³ (same as the intact residual soil). The approximate wet weight of sand-kaolin mixture for each specimen was pre-calculated based on the volume of the specimen (50mm in diameter & 100mm in height). In order to get a more uniformly distributed specimen, the specimen was compacted layer by layer. The wet sand-kaolin mixture was divided into ten equal portions; each portion is for one layer. One portion (about 32g) of mixture was put into a two-part split mould then compacted to a thickness of 10 mm by using a static compression machine. The split mould and compression machine used are the same as the one used to prepare the compacted residual soil specimens (as shown in Figure 3.6 & Figure 3.7(a)). After the compaction of the first layer, the top surface of the layer was scratched; then the next portion of mixture was placed into the mold. The process was repeated until all ten layers were done. The specimen was extruded from the mould and transferred into the furnace and burnt at 500°C for 5 hours. As studied by Grim (1953), temperature around 500°C or higher is likely to result in bonding and probably bond strength could be increased at higher temperature. The artificially bonded sample would be setup in a triaxial cell when it had completely cooled down the next day. The dimension of artificially cemented soil specimen before and after burn was measured carefully with vernier caliper. An example of artificially cemented soil specimen was presented in Figure 3.9. The basic soil properties of the artificially cemented soil were listed in Table 3.2.

The microstructure of the artificially cemented residual soil was observed through a scanning electron microscope (SEM) as shown in Figure 3.10.

A piece of artificially cemented specimen was soaked into water to study the cementation behaviour. A series of pictures was taken at different time intervals (as shown in Figure 3.11). The specimen stood still after two months. Green-color algae started to grow inside the specimen after 30 days.



Figure 3.9 Artificially Cemented Soil Specimen for Triaxial Test

Table 3.2 Basic Properties of Artificially Cemented Residual Soil

Tests	Results
Bulk Density (before burn)	1.58-1.69 Mg/m ³
Dry Density	1.37-1.47 Mg/m ³
Kaolin Content	50%
Changi Sand Content	50%
Specific Gravity	2.644
Water Content (wet mixture)	about 15%

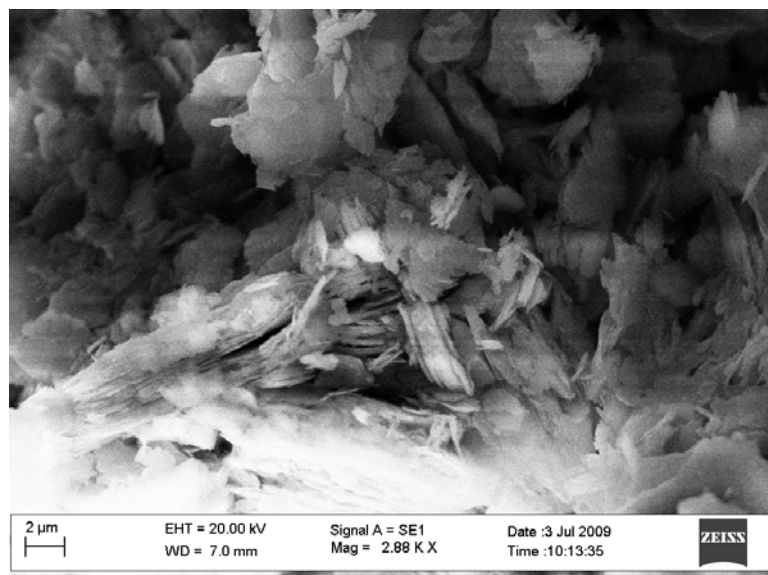


Figure 3.10 SEM Image of Artificially Cemented Soil Specimen



(a)



(b)



(c)



(d)

Figure 3.11 Compacted Residual Soil Specimens: (a) Day 1; (b) Day 30; (c) Day 45; (d) Day 60

3.3 Experimental Set-Up

3.3.1 Triaxial Testing System

3.3.1.1 Testing System

A schematic diagram of triaxial experimental set-up is shown in Figure 3.12.

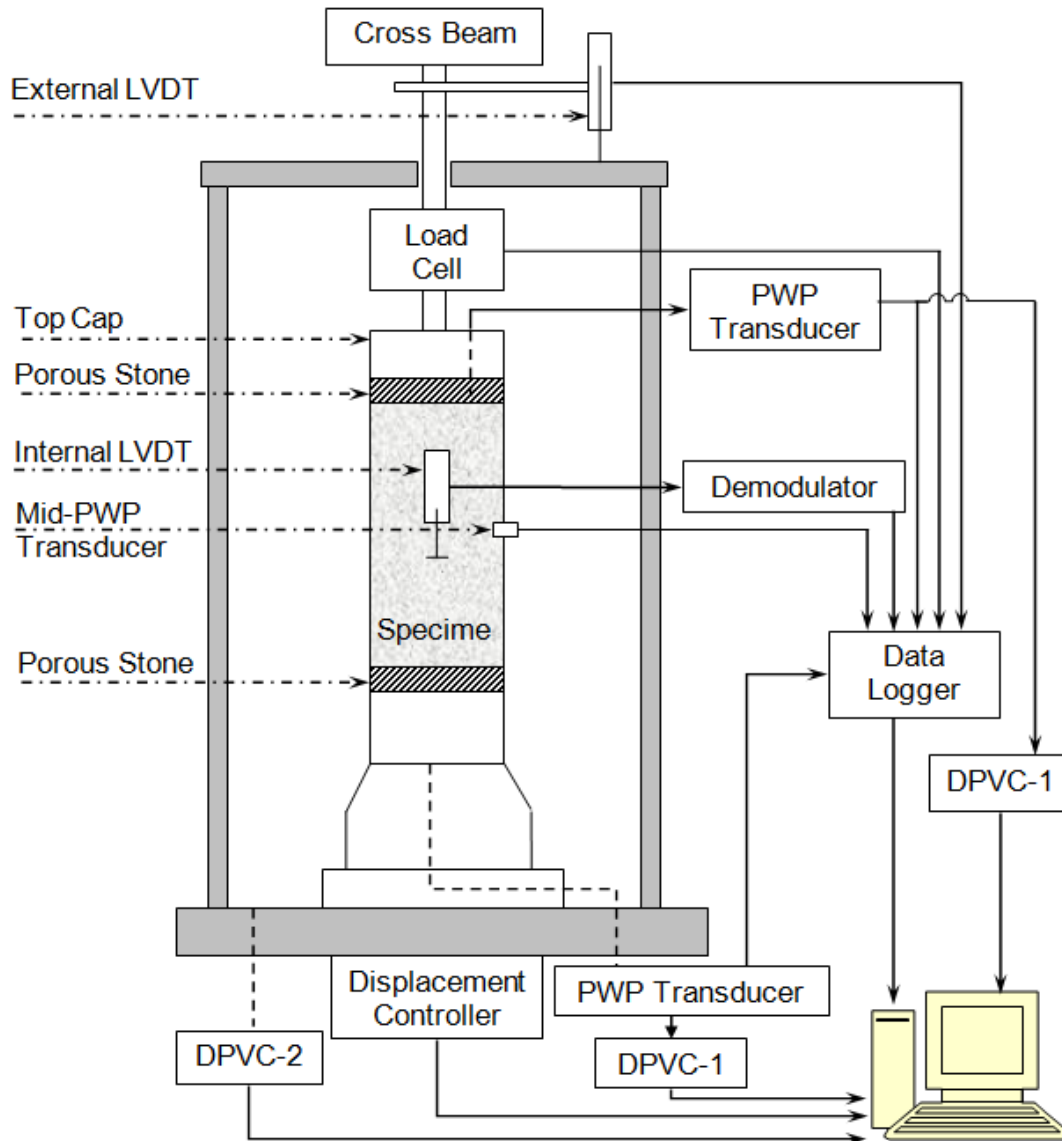


Figure 3.12 A Schematic Diagram of Triaxial Experimental Set-up

An internal load cell was used to measure the axial load and an external LVDT (Linear Variable Differential Transformer) was installed at the top of the cell to

measure the axial deformation of the specimen. The capacity of internal load cell is 5 kN. The vertical load was applied by moving up/down the base pedestal of the compression machine at a desired rate of displacement. It can be measured and controlled to an accuracy of 1 Newton. One external Linear Variable Differential Transformer (LVDT) and a pair of miniature submersible LVDTs (internal LVDTs) were used for vertical strain measurement. The external LVDT had a maximum range of 50 mm and it measured directly from the movement of the triaxial cell. A pair of internal LVDTs was installed to measure local strain. They were used for initial displacement measure during the crucial beginning stage of the tests. To install internal LVDTs in the cell, a pair of small spindle holders and a pair of LVDT holders were fabricated. Both spindle holders and LVDT holders were attached to the specimen using super glue. The LVDT and spindle must be proper aligned, to make sure the spindle could move in and out smoothly. A picture of installed internal LVDTs is presented in Figure 3.17.

The cell pressure (σ_3) in the triaxial cell was controlled by a digital pressure/volume controller (DPVC). The DPVC could apply a pressure up to 2000 kPa. The cell pressure was measured to an accuracy of 1 kPa. Another DPVC with an accuracy of 0.5 kPa was used to control the back pressure (PWP, u), which was applied into the specimen (from both top and bottom) during the testing. The DPVCs were controlled by IEEE-PC488 card via personal computer (PC). The cell/back pressure change and change in volume of specimen could be read directly from DPVCs and recorded into the PC.

In order to check the pore water pressure equalization within the soil specimen, two external pore water pressure transducers were used to measure PWP at the top and bottom of the specimen. A mid-pore pressure transducer was also used to measure the PWP at middle of the specimen. The purpose of mid-pore pressure transducer is not only to monitor the non-homogeneous pore pressure distribution during undrained shearing, but also to check the strain rate for shearing.

To conduct triaxial extension (CK₀UE) tests, a loading frame was used to hold load cell in position. The fiberglass repair resin was used to connect top cap and load cell

together (as shown in Figure 3.13). Triaxial machine moved downwards to extend specimen until failure. The photos of CK₀UE testing system were shown in Figure 3.14.



(a)



(b)

Figure 3.13 Connection between Load Cell and Top Cap by Fiberglass Repair Resin



(a)



(b)

Figure 3.14 CK₀UE Testing Set-Up: (a) Overview; (b) Loading Frame for CK₀UE Testing System

3.3.1.2 Specimen Set-up for Triaxial Test

The specimen set-up for the intact residual soil is described as example in this section. All the triaxial tests were conducted on specimens of 70 mm diameter and 140 mm height. The specimen was trimmed into dimensions by using a wire saw and a specimen trimming frame. As shown in Figure 3.15, the surface of the intact residual soil specimen was porous. To reduce the membrane penetration, the Dow Corning 3110 RTV silicone rubber was used on rubber membrane. The silicone rubber was mixed with catalyst at a ratio of 10:1. After curing for half an hour, the liquid rubber was applied inside rubber membrane, and then cured for another half an hour before placed over the soil specimen (Figure 3.16).

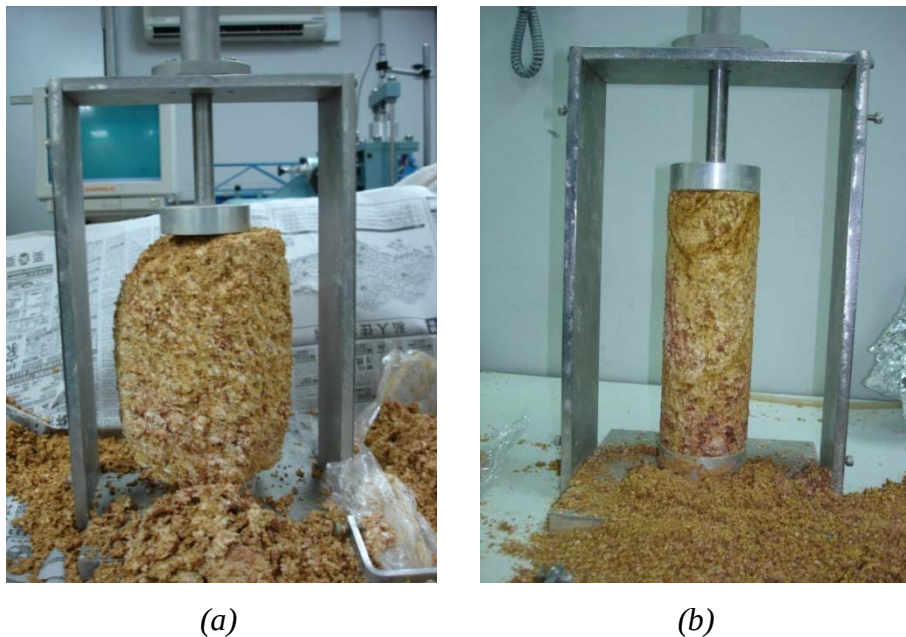


Figure 3.15 Intact Residual Soil Sample in Trimming Frame: (a) Before Trimming;
(b) After Trimming

In order to prevent leakage between top cap (and base pedestal) and the membrane, vacuum grease was applied on the side surfaces of both top cap and base pedestal. Before placing the specimen in the triaxial cell, its weight was recorded, and the pre-test water content of the sample was measured also. During setting-up the specimen for triaxial test, a porous stone and filter paper were placed over the base pedestal of motorized triaxial cell. The specimen was kept over the filter paper

underlain by porous stone. After that, the rubber membrane with liquid rubber inside was placed over the specimen using a membrane stretcher. The membrane stretcher was removed and top cap was placed carefully on top of the specimen after placing o-rings on the membrane over the base pedestal, then release the o-rings over the top cap.



Figure 3.16 Liquid Rubber Applied inside Rubber Membrane



(a)



(b)

Figure 3.17 The Set-up of the Residual Soil Specimen in the Triaxial Cell: (a) Location of Internal LVDTs; (b) Location of Mid-Pressure Transducer

A pair of internal LVDTs was installed for local strain measurement afterwards. The spindle holders and LVDT holders were attached to the specimen using super glue. The LVDT and spindle were aligned properly, to allow the spindle to move in and out smoothly. Finally, mid-pore pressure transducer was attached to specimen and tied into position by o-rings. A picture of set-up of clay specimen in the triaxial cell was shown in Figure 3.17.

3.3.1.3 Testing Procedures

The triaxial compression tests were conducted in 3 stages: (1) saturation; (2) consolidation and (3) shearing.

1). Saturation refers to a process in which the pore water pressure in the specimen is increased so that air as a separate phase in the void spaces is eliminated. This enables pore water pressure to be measured reliably. Saturation was effected by raising the pore water pressure to a level high enough for the water to absorb all the air, which is originally in the void spaces, into solution. Meanwhile, the confining pressure was raised in order to maintain a small positive effective stress in the specimen. The method used was to apply a “back pressure” to the pore fluid by step of 50 kPa, alternating with increment of cell pressure. The back pressure is always 10 kPa less than the cell pressure to ensure that the effective stress remains positive.

B-value check was performed after the saturation stage to check the degree of saturation.

$$B - Value = \frac{\Delta u}{\Delta \sigma_3} \quad (3-1)$$

The cell pressure was increased by 50 kPa while keeping the back pressure valves closed and the increment in pore water pressure was recorded (Δu). When *B*-value was measured to be greater than or equal to 0.95, the saturation was considered complete.

It should be mentioned that another saturation method was adopted for artificially cemented soil. Due to the strong bonding of soil particles, the burnt artificially

cemented soil specimen was saturated in a vacuum desiccator. It was much faster than the ordinary saturation process conducted in triaxial cell. The burnt artificially cemented soil specimen was placed inside the desiccator, and submerged with distilled water. A vacuum was applied through the stopcock provided on the cover of desiccators to remove the air bubbles out of the specimen. Once all the air bubbles disappeared, the specimen was taken out from the desiccator for degree of saturation measurement. The calculation was based on the weight and volume change of the specimen before and after saturation. It was considered to be fully saturated if the degree of saturation was 95% or above. The specimen was then set up in the triaxial cell for consolidation and shearing.

2). Two types of different consolidation methods were adopted in this research, which were K_0 consolidation and constant principal stress ratio or constant R consolidation.

During K_0 consolidation, a vertical load was applied on the specimen until a certain σ_1' value was reached. The specimen was consolidated under the condition of $d\varepsilon_v/d\varepsilon_1 = 1$ by increasing the vertical load gradually. This technique was proposed by Chu 1991 and Lo & Chu 1991. The vertical load was applied by raising the base pedestal of the compression machine at a pre-determined rate of displacement. Drainage of water and vertical displacement were controlled by DPVC and LVDTs respectively. For over-consolidated specimen, it was consolidated to twice or four times of in-situ stress (195 kPa) and then unloaded to in-situ stress, to get an OCR value of two or four. For normally consolidated specimen, it was consolidated to a certain value of effective vertical stress, σ_1' , which was far away from the estimated pre-consolidation pressure.

Constant R consolidation was carried out on artificially cemented soil, in order to consolidate the specimen to a fixed stress state. A reasonable $R = \sigma_1'/\sigma_3'$ value was pre-determined based on the effective friction angle. In this case, an R value of 2.68 was adopted. At the beginning stage of constant R consolidation, the specimen was allowed to consolidate along the drained stress path by raising the base pedestal of

the compression machine at a strain rate of 0.005 mm/min, until an R value of 2.68 was reached. Then the R value was kept constant by adjusting the cell pressure accordingly, with an increasing load applied. Consolidation stopped when a desired stress state was reached. The control loop to maintain a constant R value was as follows:

- (a) take LVDT reading and volume change of specimen (from back pressure DPVC) to update the cross-sectional area of specimen
- (b) read load cell and back pressure (from DPVC), and compute σ_1'
- (c) compute σ_3 from constant R value of 2.68, σ_1' and back pressure
- (d) send digit signal to cell pressure DPVC to target the respective pressure
- (e) read all transducers, plot the effective stress path and strain path, record the stress-strain data and return to step (a)

3). In the shearing stage, the axial load was gradually increased (or decreased for CK_0UE tests) while the total cell pressure (σ_3) remained constant, until failure occurred due to the maximum available shear strength of the sample being overcome. Shearing can be extended to reach an “ultimate” condition. The failure was marked by the peak effective stress ratio (q/p') and the test was stopped when shear band was fully developed and q was going down with further shearing. Figure 3.18(a) & (b) presented the failure mode of residual soil specimens after compression and extension shearing.

A pre-determined constant rate of strain was applied from the control box of a compression machine. A computer program was used to record and plot the data. For all the triaxial tests, the strain rates used were 0.01mm/min, 0.007mm/min and 0.005mm/min. In the CU tests, no drainage was permitted, but measurements were made of pore water pressure changes.

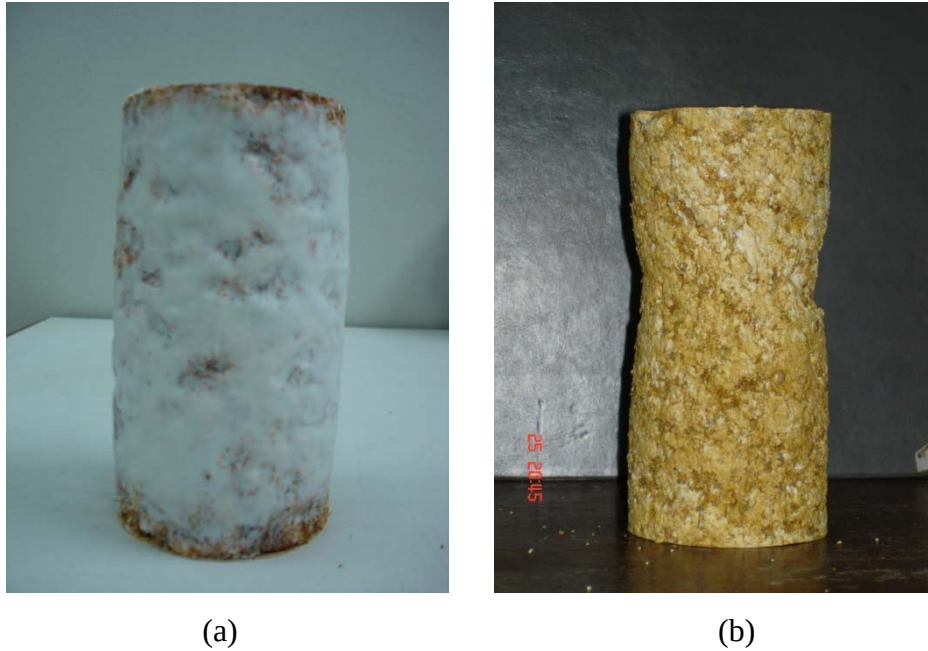


Figure 3.18 Pictures of Failure Mode: (a) from CK_0UC Test; (b) from CK_0UE Test

3.3.2 Direct Simple Shear (DSS) Testing System

3.3.2.1 Testing System

A modified NGI type of direct simple shear (DSS) apparatus was used in this study (as shown in Figure 3.19). The reinforced membrane used by Bjerrum & Landva (1966) was replaced by a stack of slip metal rings. The 73mm diameter and 20 mm high circular disk specimen was enclosed in a rubber membrane and confined in metal rings. The use of metal rings allowed the specimens to tilt rather than to slide along a predetermined failure surface.

Linear Variable Differential Transformers (LVDTs) with a maximum range of 10mm were installed to measure vertical and horizontal displacement. Load cells were used to measure the load applied on both horizontal and vertical directions. The analogue data from load cell and LVDTs were converted into digital format by data logger. The data was scanned continuously during the test and recorded in the PC.

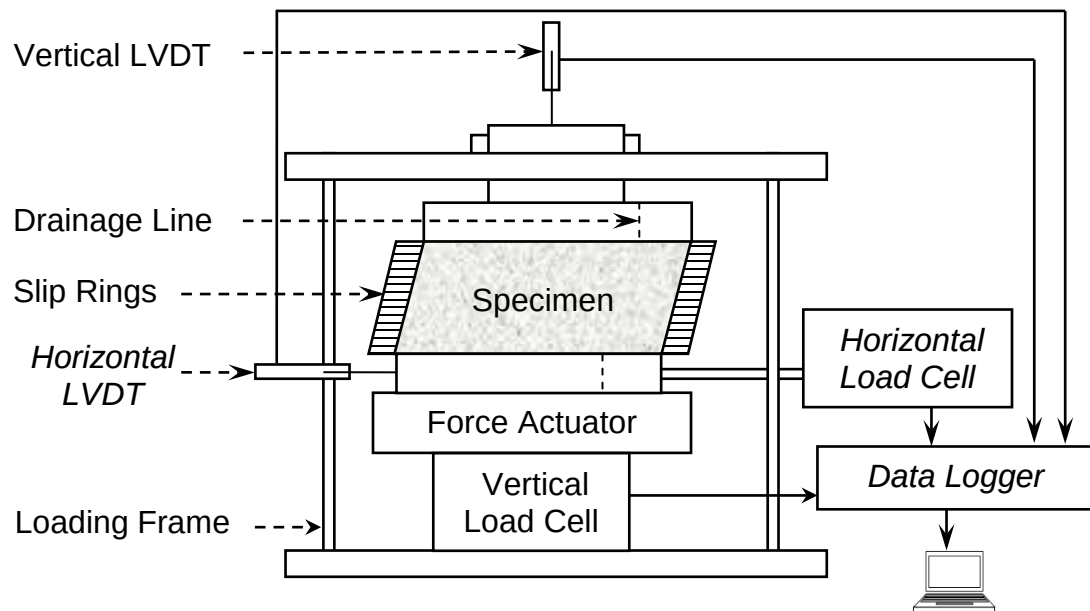


Figure 3.19 A Schematic Plot of Direct Simple Shear Testing System

3.3.2.2 Specimen Set-up

Figure 3.20 presents the set-up for DSS testing system. A cutting ring was used to cut the soil sample into a specimen of 70 mm in diameter and 20 mm in height. Porous stones and filter papers were used to improve drainage of water in base platen and top cap. Same as triaxial tests, rubber membrane and grease were used to prevent leakage. After placed the specimen in position, and covered by rubber membrane, metal rings were used over the specimen to prevent lateral displacement of specimen. Drainage lines, which were flushed by de-air water, connected to base pedestal and top cap respectively, to allow excess pore water pressure to dissipate. Vertical displacement and horizontal displacement of the specimen were measured by vertical and horizontal LVDTs respectively.



Figure 3.20 Set-up for DSS Testing System

3.3.2.3 Testing Procedures

Different from triaxial tests, only 2 stages were needed for direct simple shear tests, which were:

1) Consolidation

A predetermined vertical stress was applied to consolidate the specimen. Vertical displacement was measured by vertical LVDT. Excess pore water was allowed to flow out of the specimen freely through the drainage lines provide at both top cap and bottom pedestal. When no more vertical settlement was recorded, it indicated the completion of consolidation.

2) Shearing

The specimen was sheared horizontally by moving the bottom pedestal, where the specimen sat on, to the right, while keeping the top of specimen still. For undrained shearing, the excess pore water pressure was allowed to dissipate during shearing; meanwhile, the vertical displacement of the specimen should be maintained

constant by controlling the normal stress applied, so as to keep the volume of specimen unchanged. It is a technique commonly adopted by other researchers (Bjerrum and Landva 1966; Chu et al. 1999; Chu et al. 2003; Ladd 1991; Ladd and Edgers 1972). Vertical load, horizontally load, vertical displacement and horizontal displacement were measured by load cells and LVDTs respectively. The change in the vertical stress can be considered as the change in pore pressure (Chu et al., 1999; Chu et al., 2003). The soil specimen considered to have failed when the horizontal load reached ultimate stage or started decrease.

Direct simple shear (DSS) test, as an effective method to measure the undrained shear strength and OCR of soil, is commonly used in geotechnical engineering design and analyses. In 2000, ASTM developed the first standard for testing clay soils in the direct simple shear test (ASTM D 6528) making it readily useable in engineering practice. The DSS test has following advantages over the triaxial test: 1) DSS test is cheap, and fast, especially for granular soil; (2) the DSS specimen is significantly smaller than triaxial specimen and therefore, the specimen disturbance involved in DSS test should be smaller (Baxter et al., 2002); (3) the DSS test is simple, which does not require saturation and measurement of pore water pressure; (4) the mode of failure in DSS represents an average shear strength on a failure plane beneath a footing or in an embankment (Ladd 1991); (5) in DSS test, it could observe the failure plane and thin failure zone in nature.

It should be point out that the DSS test also has some disadvantages, which the practicing engineer may want to consider, such as stress concentration, forced horizontal failure plane and rotation of principal stresses. However, as pointed out by Chu et al. (2003), these disadvantages should not affect the determination of undrained shear strength and OCR.

3.3.3 Plane-Strain Testing System

3.3.3.1 Testing System

Figure 3.22 presents a schematic diagram of plane-strain testing system (Wanatowski and Chu 2006). A prismatic soil specimen with a height of 120mm

and a cross-section of 60 x 60mm was used for all the plane-strain tests in this study. The lateral loading platens were fixed in position by two pairs of horizontal tie rods (as shown in Figure 3.23). The rods were located two on each side of specimen, one above and one below. Therefore, no lateral movement was allowed along the rods during a test, and thus the plane-strain condition was imposed. The lateral stress σ_2 was measured by four submersible pressure transducers. The transducers of a diameter of 35mm were embedded in the two lateral loading platens. The total lateral pressure was taken as an average value of the four transducers readings. Pictures of the lateral loading platen and the submersible pressure transducer are shown in Figure 3.21.

During plane-strain tests, the axial load was applied through a hydraulic force actuator mounted at the bottom of the loading frame (as shown in Figure 3.22). The actuator was controlled by a computer via a digital load/displacement control box. The control box adjusted the movement of the base pedestal to achieve desired rate of load or rate of displacement.

Similar to the triaxial testing system, two digital pressure/volume controllers (DPVCs) were used for cell pressure and back pressure control. Both internal and external linear variable differential transformers (LVDTs) were installed for axial strain measurements. Due to the limited numbers of data logger channels, the pore water pressure was measured at top and middle of the specimen through PWP transducers.

The DPVCs and the digital load/displacement control box were connected to the host PC via IEEE port. The data from other transducers (four individual pressure transducers, two LVDTs and two PWP transducers) were converted from analogue format into digital format by a data acquisition system. They were scanned continuously during the test and stored in the PC.

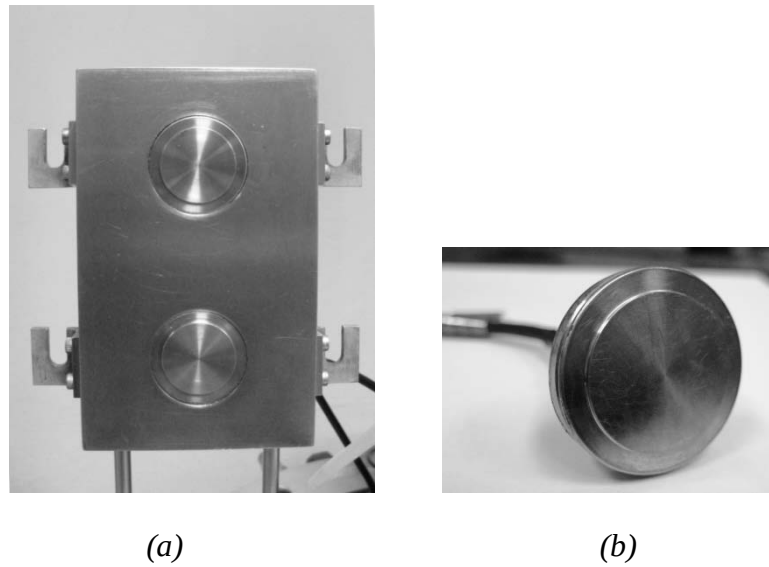


Figure 3.21 (a) Lateral Loading Platen; (b) Pressure Transducer

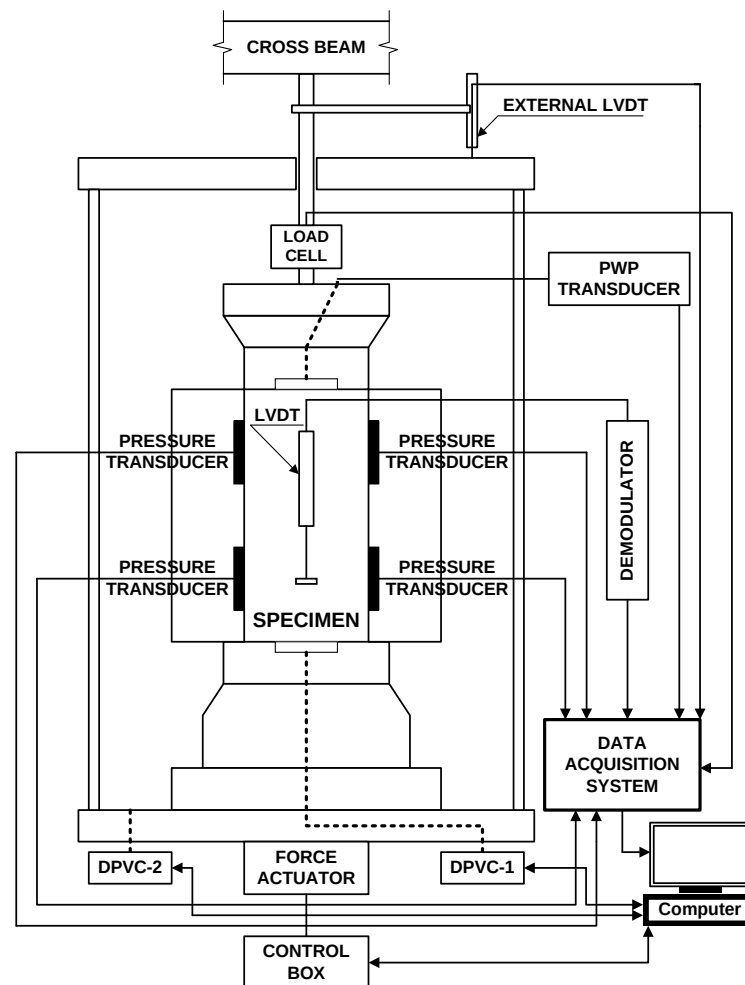


Figure 3.22 The Plane-Strain Testing Arrangement (Wanatowski and Chu 2006)

3.3.3.2 Specimen Set-up

The pictures of plane-strain apparatus taken from different angles of view are presented in Figure 3.23. Large block of intact residual soil needed to be trimmed into a prismatic specimen with the dimension of 60×60×120mm. For compacted residual soil, the specimen was compacted into the desired dimension by using a two-part split mould.

The setup sequence for plane-strain tests are the same as triaxial tests. Before placement of the specimen, vacuum grease was applied onto the side surface of top cap and base pedestal to prevent leakage. Porous stones and filter papers were used to facilitate the drainage during tests. The prismatic specimen was kept over the filter paper underlain by porous stone. The specimen was covered by a rubber membrane, which was sealed by o-rings at top and bottom. Liquid rubber technique was also applied for intact residual soil specimens.

The plane-strain testing condition was achieved by restraining the specimen movement in horizontal direction. It was imposed by the installation of a pair of vertical platens as shown in Figure 3.23. The vertical platens were fixed into position by horizontal tie rods, which located two on each side of the specimen, one above and one below. The internal LVDT used for local strain measurement was attached on one of the upper tie rod. The spindle holder was glued onto the specimen. The spindle was aligned properly to allow a free move in / move out. The mid PWP transducer was installed on the other side of the specimen. It was tied into position by o-ring(s).

Another notable feature of the plane-strain apparatus should be mentioned was the application of free-end technique on the vertical platens. It was helpful to reduce the boundary restraints and to delay the occurrence of non-homogeneous deformations (Rowe and Barden 1964; Wanatowski 2005). A layer of vacuum grease and a Teflon sheet was used to enlarge and lubricate the rigid platen.

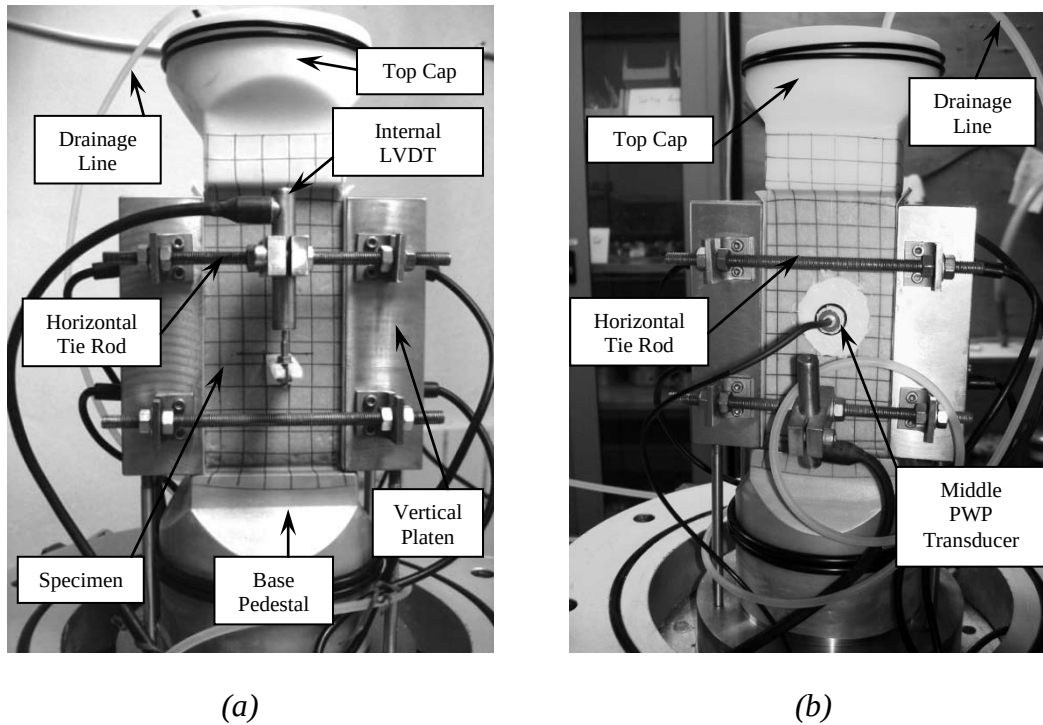


Figure 3.23 Specimen Set-up for Plane-Strain Test: (a) Front; (b) Back

3.3.3.3 Testing Procedures

Same as triaxial tests, the plane-strain tests were also conducted in three stages, which were saturation, consolidation and shearing. All the plane-strain specimens were saturated in the plane-strain cell, by raising the pore pressure to a level high enough for the water to absorb all the air into solution. An effective confining stress of 10 kPa was applied all the way along saturation. Once a B -value of 0.95 was achieved, anisotropic consolidation (K_0 consolidation) was conducted under the condition of $d\varepsilon_v/d\varepsilon_1 = 1$ by increasing the vertically load gradually (Chu 1991; Lo and Chu 1991). Consolidation stopped until the specimen reached a pre-determined stress state. A standard undrained axial compression test was carried out subsequently. The axial load was increased gradually while cell pressure maintained constant. The specimen was sheared until failure occurred.

3.4 Conclusions

This chapter presented the experimental set-up adopted in this research. The different types of soil used for study were introduced, which are intact residual soil,

compacted residual soil and artificially cemented soil. The testing systems and specimen set-up under triaxial, direct simple shear and plane-strain conditions were described. The methodologies adopted to conduct various types of tests were discussed.

CHAPTER 4

CONSOLIDATION BEHAVIOUR OF RESIDUAL SOILS AND ARTIFICIALLY CEMENTED SOIL

4.1 Introduction

Compressibility is an important soil property that is required for geotechnical designs. In this chapter, the consolidation behaviour of three different soils was studied. These were intact residual soil, artificially cemented soil and compacted residual soil. Anisotropic (K_0) consolidation was conducted on each specimen under the condition $od\varepsilon_v/d\varepsilon_1 = 1$ by increasing the vertical load gradually. The testing data and interpretation are presented and discussed in this chapter.

4.2 K_0 Consolidation of Intact Residual Soil

4.2.1 Vertically Cut Specimens

4.2.1.1 Using Triaxial Cell

The pre-consolidation stress (σ_p'), as an important parameter related to the consolidation, deformation and strength characteristics of soil, is normally determined from the laboratory one-dimensional consolidation (oedometer) tests. Casagrande (1936) defined the term σ_p' as the “largest overburden in which soil had been consolidated”. It separates small strain from large strain deformation, regardless of the mechanisms that cause a clay to have an $OCR > 1$ (DeGroot et al.,

2007). Therefore, σ_p' together with the undrained shear strength (c_u), are two of the most important soil parameters for geotechnical designs (Ladd & DeGroot, 2003). It should be pointed out that for a cemented material, such as residual soil, the overconsolidation and shear strength are affected not only by mechanical loading-unloading, but also by the other diagenetic process, particularly cementation at the clay particle contacts (Gutierrez et al., 2008).

Numerous methods have been proposed for estimating σ_p' from oedometer test data (Casagrande, 1936; Schmertmann, 1955; Janbu, 1969; Butterfield, 1979; Sridharan, 1991). These methods are usually based on the relationship of experimental void ratio (e) and effective consolidation stress (σ_1'). Among these methods, the most popular are the e -log(σ_1') method (Casagrande, 1936) and the log(1+ e)-log(σ_1') method proposed by Butterfield (1979) and supported by Sridharan (1991). Strain energy method, which firstly proposed by Becker et al. (1987) is also commonly used for σ_p' determination. Considering σ_p' as the maximum yielding stress to which a soil has ever been subjected, work by Tavenas et al. (1979) and Senol et al. (2006) is supportive of this method. Theoretically, the energy method is equivalent to the e -log(σ_1') method (Li, 1989), or the log(1+ e)-log(σ_1') method (Wang & Frost, 2004). Lately, a dissipated strain energy method (DSEM) was proposed by Wang & Frost (2004), which could minimize the sample disturbance effects and eliminate the effect of elastic deformation.

Two oedometer tests were conducted on the vertically cut intact residual soil specimens as summarized in Table 4.1. The cutting orientation of the intact residual soil was mentioned in Figure 3.4. Several series of K_0 consolidated undrained triaxial compression (CK₀UC) and extension (CK₀UE) tests were also conducted on the same type of soils to study their strength and deformation behaviour (as listed in Table 4.2). The K_0 consolidation data were compared with oedometer test results in Figure 4.1. It can be seen that the $\varepsilon_1\%$ versus log σ_1' and void ratio (e) versus log σ_1' curves obtained from the two types of tests are very different. The reasons for the differences include the bedding errors and wall friction encountered in the oedometer test and the inability to saturate the oedometer specimen and prevent it

from drying or wetting during the test (Chu et al, 2005). Due to the small dimension of testing specimen (19mm in height), a bedding error of 1 mm would lead to an error of 5% in vertical strain and 0.1 in void ratio, which was close to the difference between the oedometer and K_0 consolidation results. However, these difficulties could be overcome in K_0 consolidation test using triaxial cell. Therefore, K_0 consolidation results using triaxial cell are more accurate.

Table 4.1 Summary of Oedometer Tests conducted on Vertically Cut Intact Residual Soil Samples

Test No.	Oedo 1	Oedo 2
Diameter (mm)	63	63
Height (mm)	19	19
Weight of Specimen (g)	104.04	105.58
Water Content (%)	31.48	29.61
Initial Void Ratio	1.02	1.00
Testing Method	Incremental Load	Incremental Load
Minimum Load Applied (kg)	0.5	0.5
Maximum Load Applied (kg)	32	64

Table 4.2 Summary of K_0 Consolidation Tests Conducted on Vertically Cut Intact Residual Soil under Triaxial Conditions

Test No.		GN5	GN8	GN9	GN10	GN11	EGN4
Diameter (mm)		70	70	50	50	50	70
Height (mm)		140	140	100	100	100	140
Weight of Specimen (g)		1039.2	1037.63	351.86	352.95	357.00	1054.43
Water Content (%)		30.18	31.41	30.22	29.91	31.56	31.84
Initial Void Ratio, e_0		0.816	0.834	0.817	0.813	0.836	0.840
Type of Test		CK ₀ UC	CSD	CK ₀ UC	CK ₀ UC	CK ₀ UC	CK ₀ UE
Pressure for Saturation (kPa)	Cell	1010	860	860	860	860	1010
	Back	1000	850	850	850	850	1000
Consolidation Rate (mm/min)		0.005	0.005	0.005	0.005	0.005	0.007
Maximum Consolidation Pressure (kPa)	σ_1'	195.27	288.98	731.56	599.79	901.07	452.22
	q	0	74.48	364.56	321.79	448.07	265.72
	p'	200.76	239.33	488.52	385.26	602.36	275.57

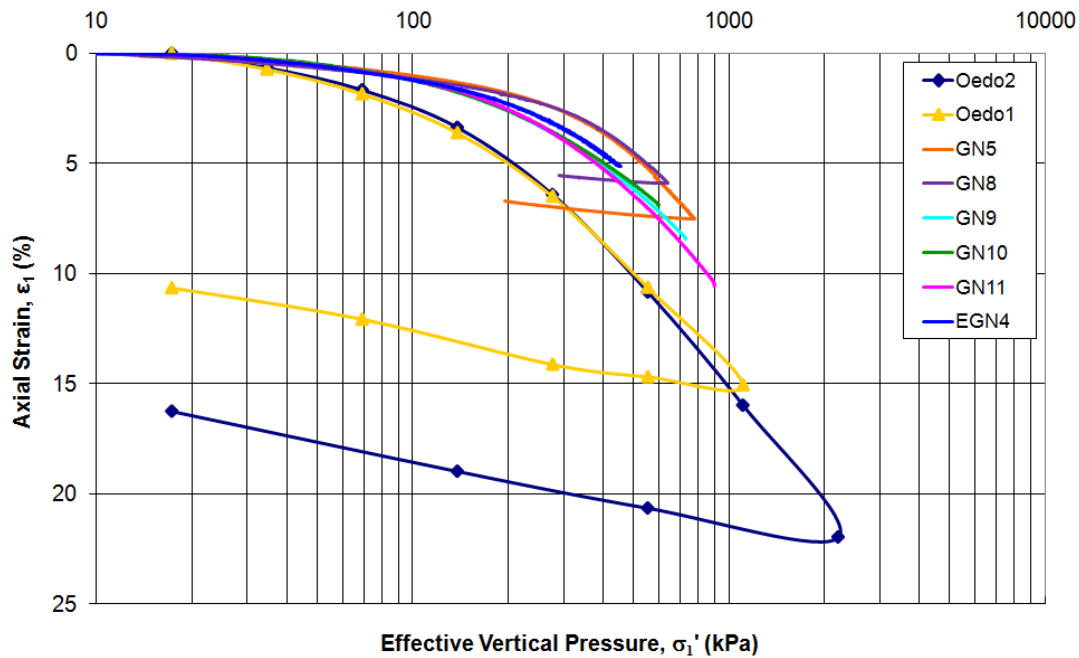


Figure 4.1(a)

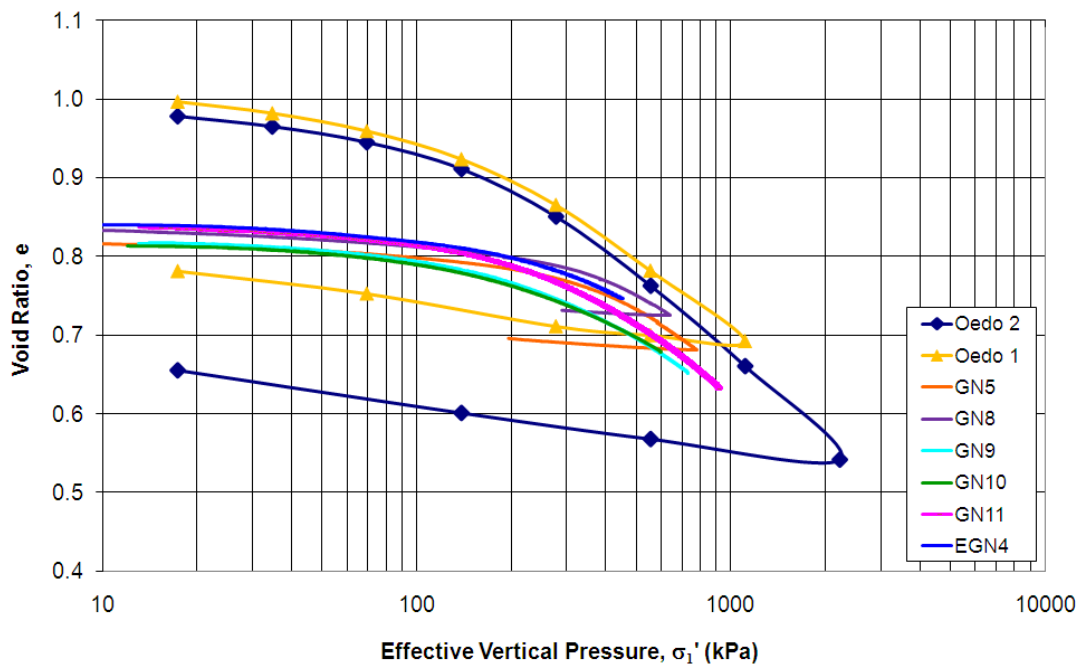


Figure 4.1 (b)

Figure 4.1 Comparison between Oedometer Tests and K_0 Consolidation Tests: (a) $\varepsilon_1\%$ vs. $\log \sigma_1'$ Curves; (b) e vs. $\log \sigma_1'$ Curves

In order to determine an accurate value of σ_p' , four commonly used methods were applied to analyze the K_0 consolidation test results, which were proposed by

Casagrande (1936), Butterfield (1979), Becker et al. (1987) and Senol et al. (2006). It can be seen from Figure 4.2, all these four methods are not suitable for the intact residual soil used in this study. The void ratio varied from specimen to specimen, as shown in Figure 4.2(a) and (b). The strain energy curves presented in Figure 4.2(c) and (d) diverged a lot at high stress level. It is impossible to identify the point of maximum curvature and the slope of curves at high stress levels, which are essential for σ_p' determination.

Recently, DeGroot et al. (2007) proposed another simple and direct procedure for estimating the σ_p' from the excess pore water pressure (Δu) data measured during a constant rate of strain consolidation (CRS). A similar method was applied on the K_0 consolidation data of the intact residual soil, as shown in Figure 4.3. The Δu induced from the deformation of soil specimen during consolidation is an indication of the void ratio changes. It can be seen from Figure 4.3, the Δu versus $\log(\sigma_1')$ curves were more consistent than the e - $\log(\sigma_1')$ curves, because the effect of initial void ratio variation was eliminated in this method. The σ_p' of 300kPa was determined from the intersection of the minimum and maximum slopes of the curves.

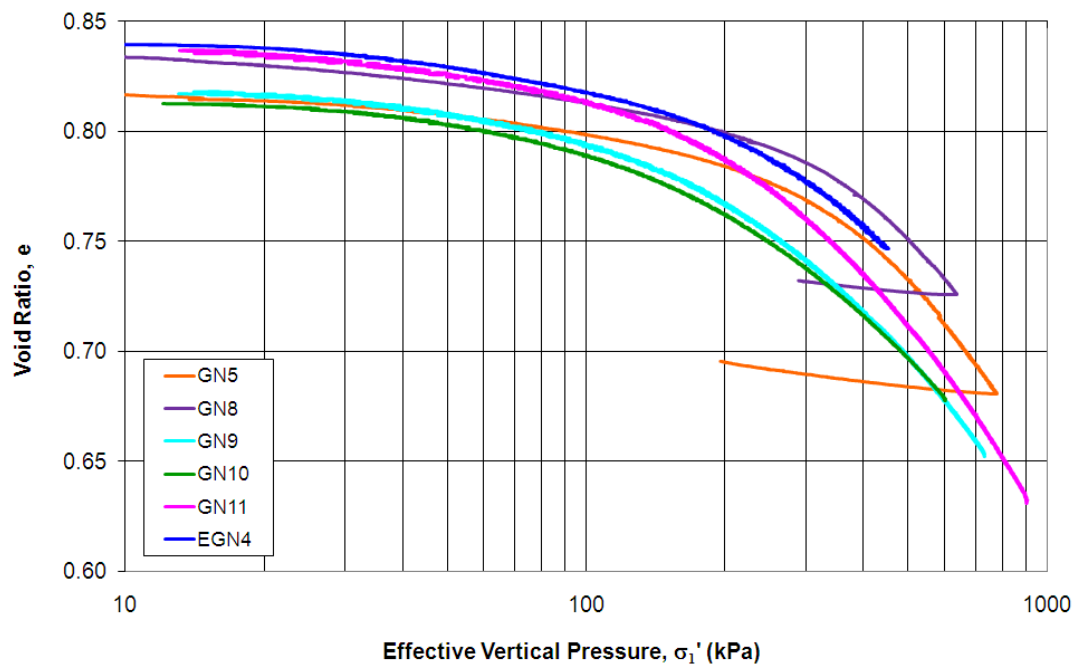


Figure 4.2 (a)

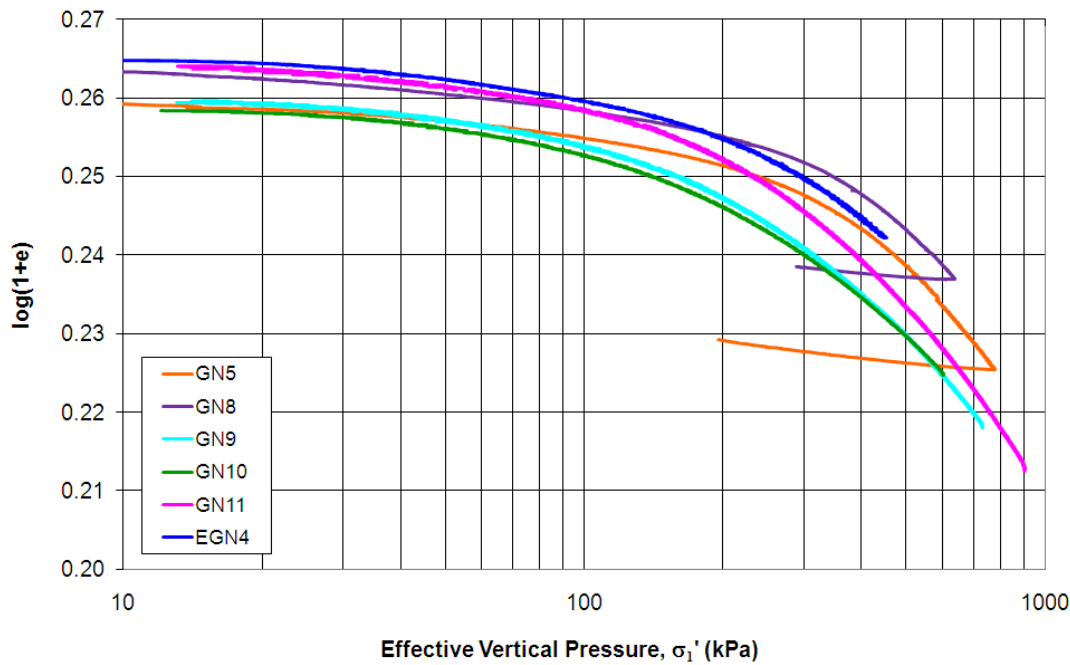


Figure 4.2 (b)

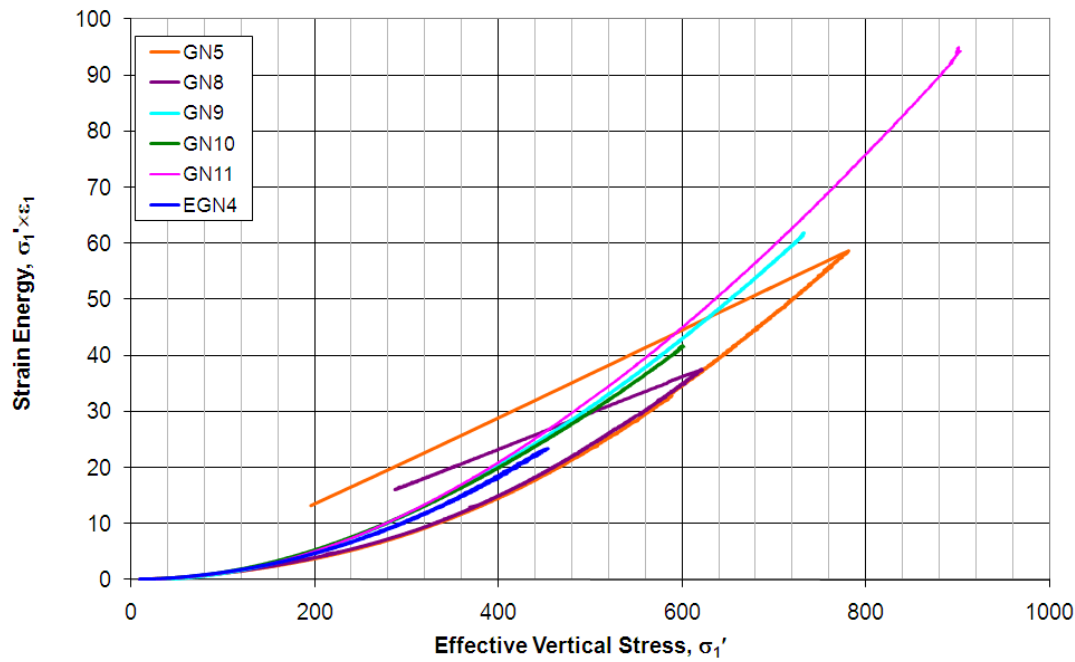


Figure 4.2 (c)

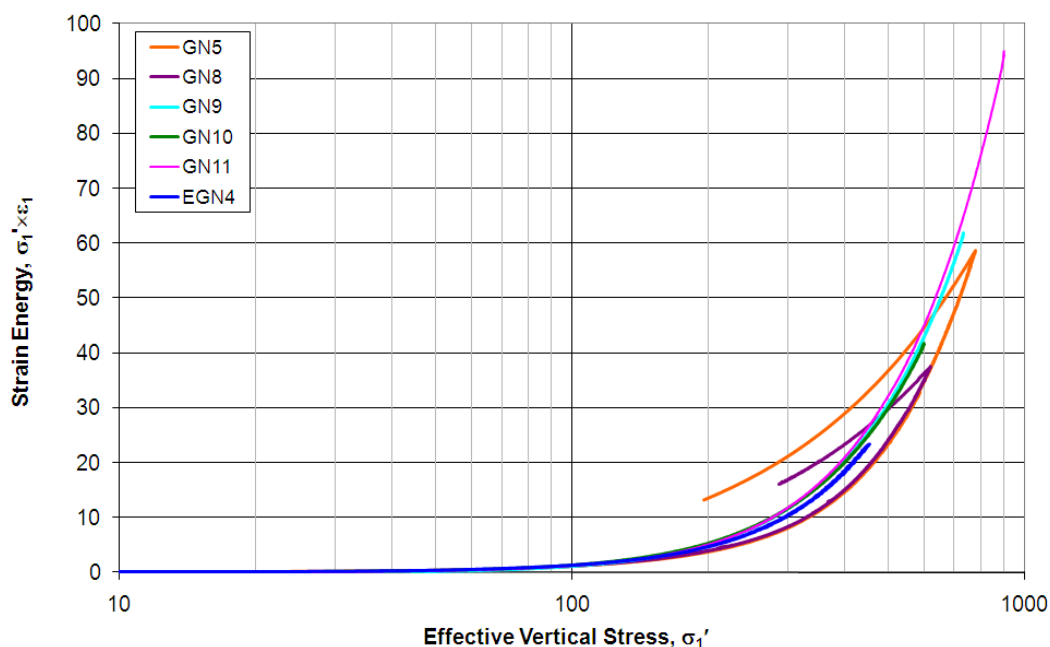
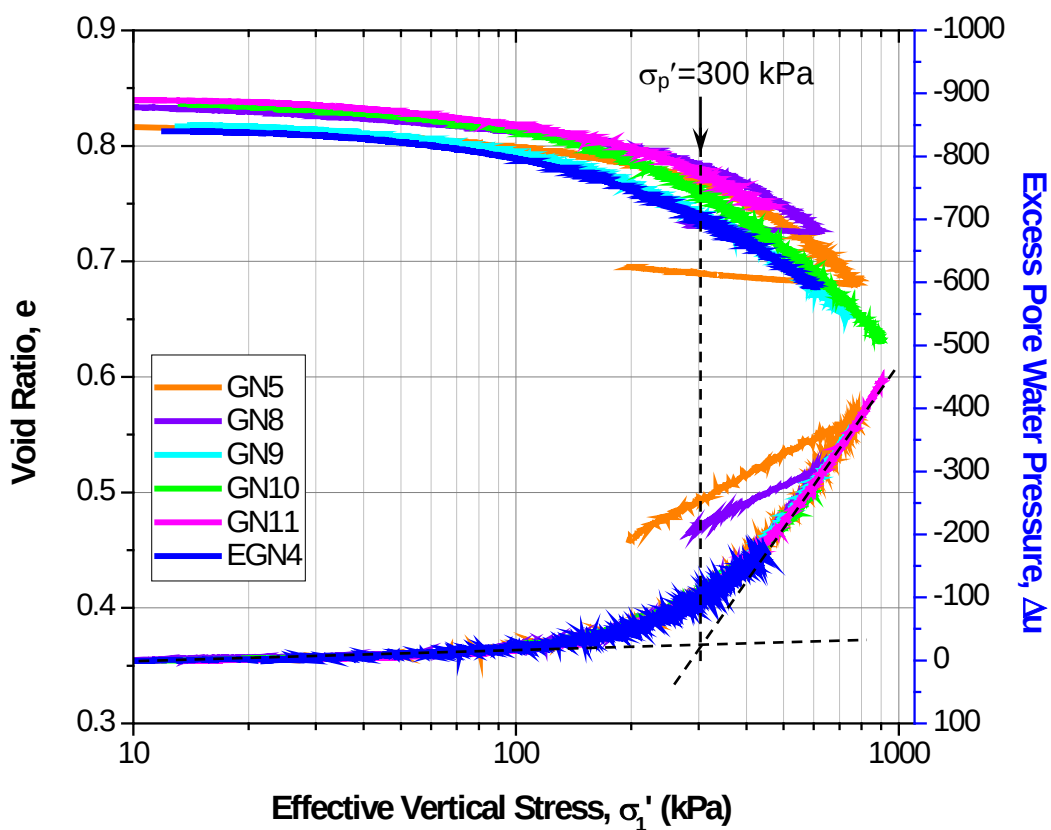


Figure 4.2 (d)

Figure 4.2 Comparison of σ_p' Determination Methods: (a) Casagrande's Method; (b) Butterfield's Method; (c) Becker's Method; (d) Senol's Method

Figure 4.3 Determination of σ_p' from e and Δu vs. $\log(\sigma_1')$ Curves

A series of CK_0UC and CK_0UE tests were conducted on the intact residual soil specimens to study its strength and stress-strain characteristics under axisymmetric conditions. The consolidation data of vertically cut specimens is presented as following to investigate the consolidation behaviour during anisotropic (K_0) consolidation. The details of these tests can be found in Table 4.2.

The stress-strain curves determined from K_0 consolidation tests on vertically cut intact residual soil are presented in Figure 4.4(a). An elastic response was observed at the beginning of the consolidation, as shown in the enlarged plot. The deviator stress increased as consolidation continued. The stress-strain curves diverged after the elastic range; this may be due to the variation in initial void ratios. However, the stress paths obtained from these tests are very consistent, as presented in Figure 4.4(c). It shows that the stress paths are not sensitive to the void ratio, as stress-strain curves did.

Two yielding points are identified from changes in the tangent stiffness (E_s) versus axial strain curves plotted in a double log scale, which located at 1.1% and 5% of axial strain. The corresponding yielding stresses were named σ_B' and σ_D' , as shown in Figure 4.4(b). The pre-consolidation stress (σ_p') is observed in between the two yielding points. The location of σ_B' , σ_p' and σ_D' were highlighted in each plot of Figure 4.4. The first notable drop in stiffness (σ_B') indicated the initiation of de-structuring process. The de-structuring process completed when the second yielding occurred (σ_D'), which led to another change in stiffness. As proposed by Malandraki & Toll (1996), three zones of behaviour were identified on the effective stress path according to the two yield points. Zone 1 was marked as section AB on the stress-strain curves and the effective stress paths. It is observed from the enlarged plot on Figure 4.4(a) that soil specimen behaved elastically in Zone 1. This is due to the presence of intact soil bonding. The de-structuring process carried out in Zone 2 (section BD in Figure 4.4(a) and (c)). Large plastic strain was developed when σ_p' is achieved, due to breaking down of sufficient soil bonding. The soil bonding was completely destroyed at σ_D' . Consequently, soil specimen behaved similarly as an unstructured material. The effective stress path migrated towards the K_0 line, and

remained on the K_0 line until the pre-determined effective vertical stress, σ_1' was reached. As shown in Figure 4.4(c) and (d), it is noted that σ_B' is consistent with the lowest value of K_0 . The second yielding occurs when K_0 approached to the constant value. It may be concluded that the variation of K_0 during consolidation test could be an indication of the de-structuring process of soil bonds.

It is important to point out the tangent stiffness used for yielding points determination is the 1-D stiffness from K_0 consolidation tests. It is different from the tangent stiffness measured during undrained shearing. By comparing them in Figure 4.5, it is noted the 1-D stiffness is much lower than the undrained ones. And the first yield occurred much earlier during undrained shearing. A small strain measurement is required for further investigation on the stiffness of residual soil.

It is reported by many researchers (Maccarini, 1987; Vaughan, 1988; Leroueil & Vaughan, 1990; Toll & Malandraki, 1993; Malandraki & Toll, 2000; Leroueil & Hight, 2003; Toll et al., 2006; Lo & Hinchberger, 2007), the presence of soil structure will lead the effective stress path moves along the yielding surface before yield occurs, which was above the K_0 line. The soil material shows stiff behaviour, with stress path consistent with “elastic” materials (Leroueil & Hight, 2003). The soil structure is destroyed once yield occurs. The stress path migrates towards that for the destructed soil, which is along the K_0 line. Based on this observation, it may conclude that the cementation of soil structure is related to the distance between the stress path and K_0 line. Therefore, a cementation index (I_c) is proposed in this research to measure the cementation of soil, which can be calculated using the mathematic equations for measuring the distance from a point to a straight line, as following:

$$I_c = \frac{|\overline{K_0}p_m' - q_m|}{\sqrt{\overline{K_0}^2 + 1}} / P_a \quad (4-1)$$

where p_m' and q_m are the coordinates of the point furthest away from K_0 line; $\overline{K_0}$ is the slope of the K_0 line, which can be expressed as $\overline{K_0}p' - q = 0$, and P_a is the Atmosphere pressure (101.325 kPa) used to eliminate the effect of units.

As shown in Figure 4.4(c), the vertically cut intact residual soil had an I_c value of 0.48. The pre-consolidation stress (σ'_p) coincided with the point furthest away from K_0 line. It was evident that σ'_p was dependent on the cementation of soil bonds and the rearrangement of soil particles after bonds breakdown.

Figure 4.4(d) and (e) present the variation of the coefficient of lateral pressure at rest ($K_0 = \sigma'_3/\sigma'_1$) value with ε_1 and σ'_1 respectively. As the K_0 -path initiated from an isotropic state ($\sigma'_3 = \sigma'_1 = 10$ kPa), the curve started from $K_0 = 1$ and dropped rapidly at the beginning. After the trough (0.25-0.33), the K_0 value picked up which could be related to the de-structure process of the intact residual soil. A constant K_0 of 0.5 was obtained at the end of the test, which may be an indication that the bonding has been completely broken down. It is also noted that the K_0 value was both stress and strain dependent.

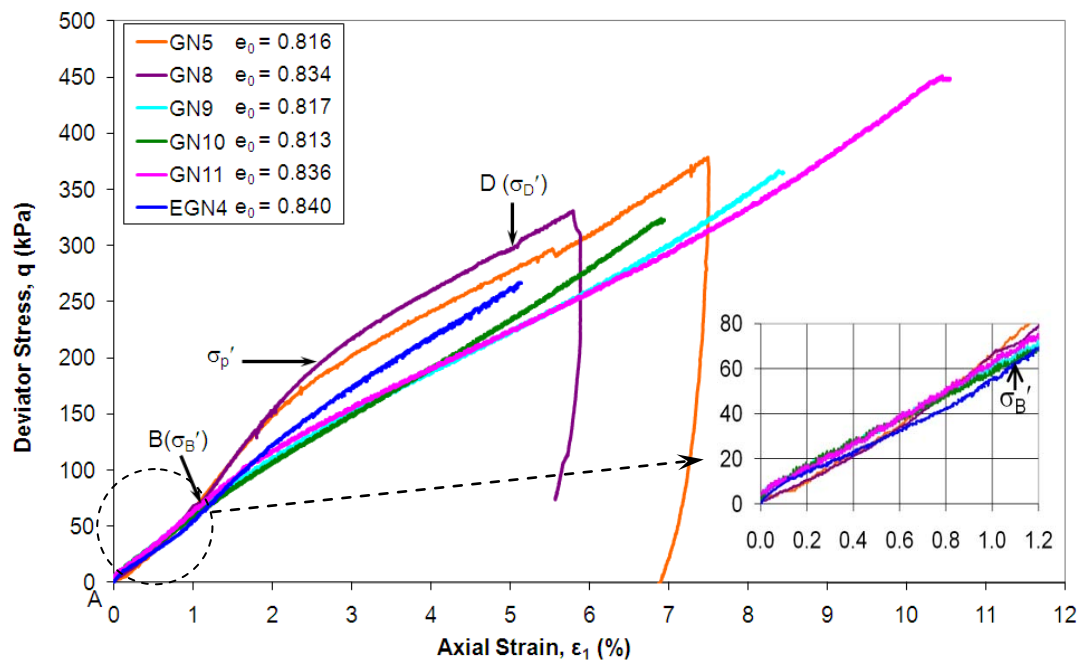


Figure 4.4 (a)

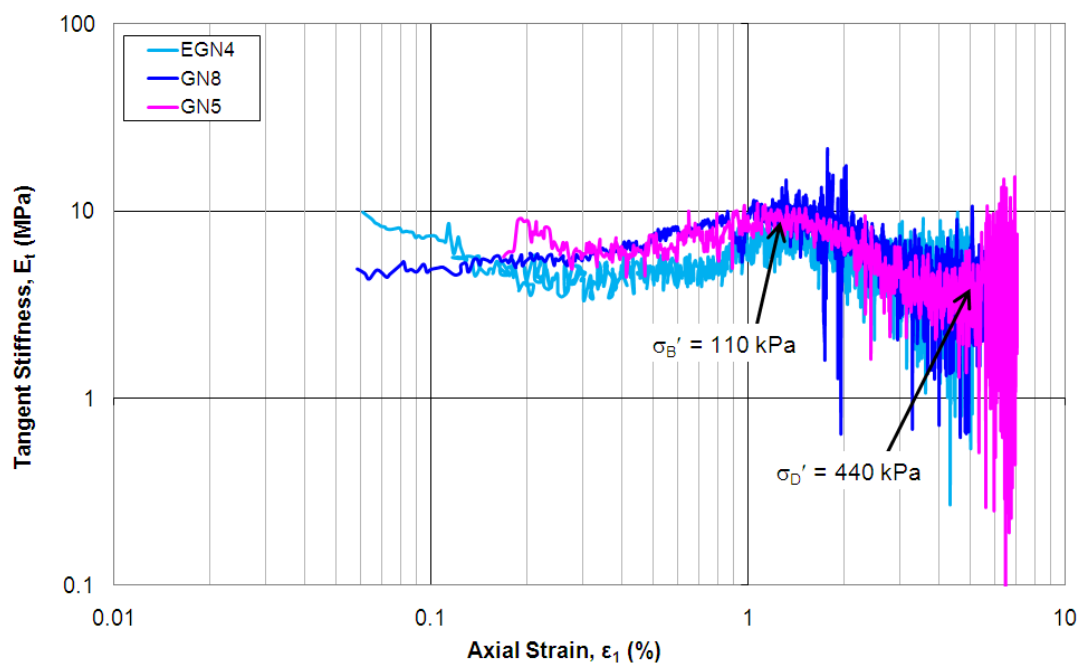


Figure 4.4(b)

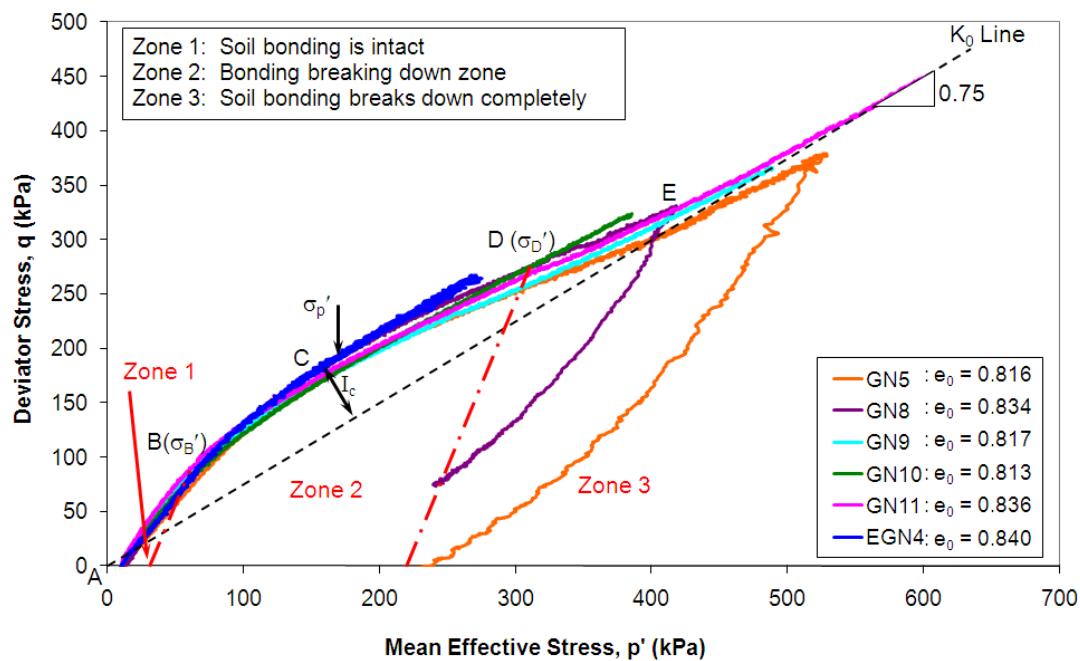


Figure 4.4(c)

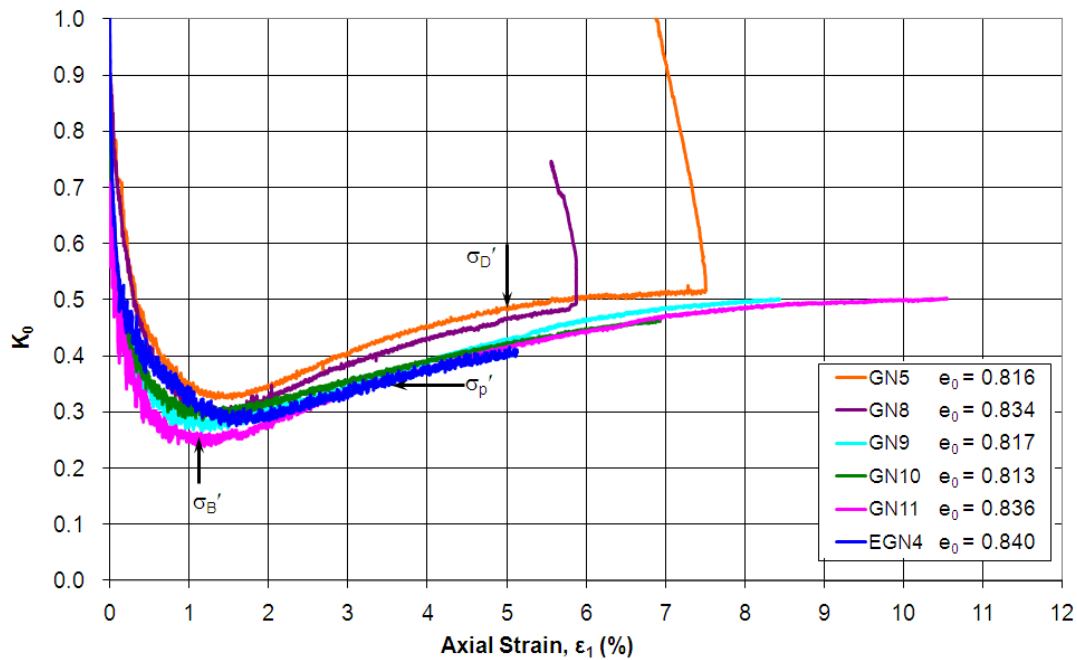


Figure 4.4(d)

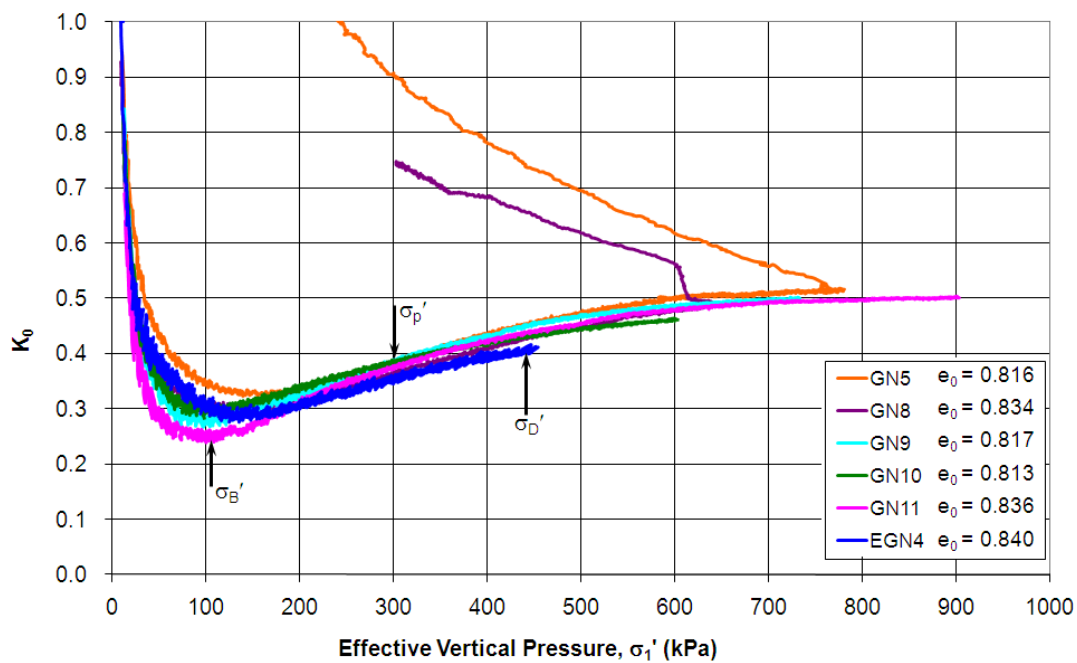


Figure 4.4(e)

Figure 4.4 K_0 Consolidation on Vertically Cut Intact Residual Soil under Triaxial Condition: (a) Stress-Strain Curves; (b) E_t vs. $\log(\epsilon_1\%)$ Curves; (c) Effective Stress Paths; (d) K_0 vs. Axial Strain Curves; (e) K_0 vs. Effective Vertical Stress Curves

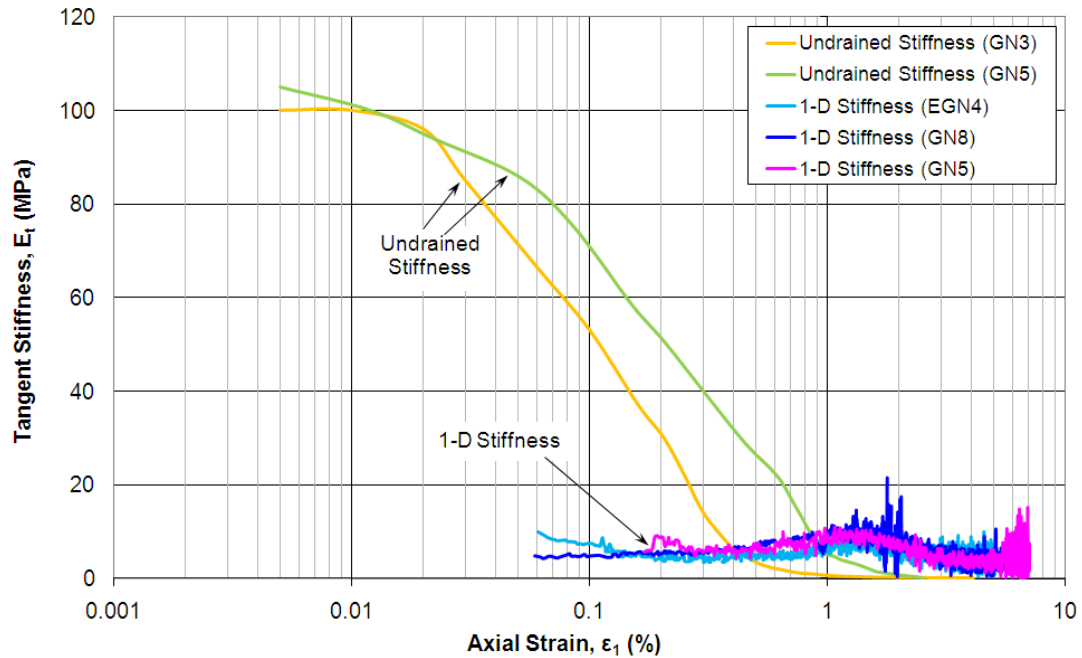


Figure 4.5 Comparison of Undrained Stiffness and Oedometer Stiffness

4.2.1.2 Using Plane-Strain Apparatuses

In order to study the strength and deformation behaviour of the intact residual soil under different stress conditions, a series of K_0 consolidated undrained plane-strain (CK₀UPS) tests were conducted, in addition to the CK₀UC tests. The consolidation tests carried out using plane-strain apparatus are summarized in Table 4.3.

It should be pointed out that the K_0 consolidation tests conducted using plane-strain apparatus should be the same as that using triaxial machine as the stress conditions imposed should be the same. As described in Chapter 3, the plane-strain apparatus used in this study allows the direct measurement of lateral pressure, σ_2 . This can be used as a check whether the CK₀UPS test results are reliable. The σ_2' versus $\varepsilon_1\%$ and σ_3' versus $\varepsilon_1\%$ curves obtained during K_0 consolidation for PSGN3 and PSGN6 tests are presented in Figure 4.6. It was observed that σ_2' was almost identical with σ_3' during K_0 consolidation under plane-strain conditions.

Table 4.3 Summary of K_0 Consolidation Tests Conducted on Vertically Cut Intact Residual Soil under Plane-Strain Conditions

Test No.		PSGN3	PSGN5	PSGN6	PSGN7
Cross-Sectional Area (mm)		60×60	60×60	60×60	60×60
Height (mm)		120	120	120	120
Weight (g)		792.50	770.76	801.47	813.64
Water Content (%)		31.32	30.12	30.28	29.47
Initial Void Ratio, e_0		0.832	0.816	0.818	0.807
Type of Test		CK ₀ UPS	CK ₀ UPS	CK ₀ UPS	CK ₀ UPS
Pressure for Saturation (kPa)	Cell	860	860	860	860
	Back	850	850	850	850
Consolidation Rate (mm/min)		0.01	0.01	0.01	0.01
Maximum Consolidation Pressure (kPa)	σ_1'	802.55	449.13	548.67	205.06
	q	399.40	238.40	286.52	129.81
	p'	536.40	290.24	357.77	118.53

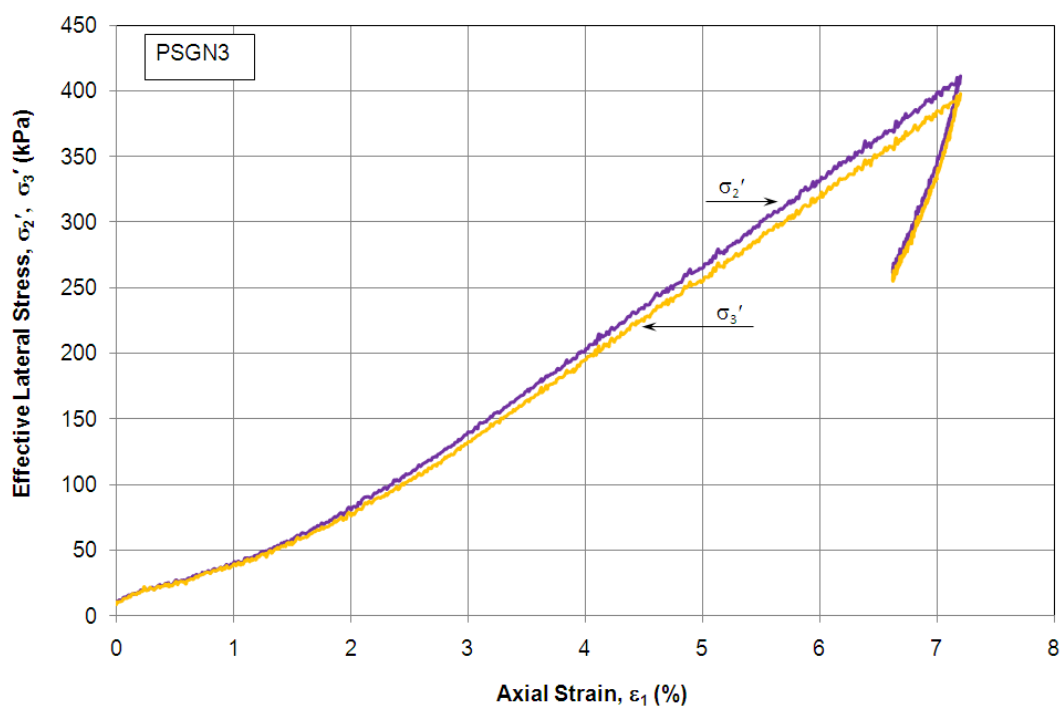


Figure 4.6(a)

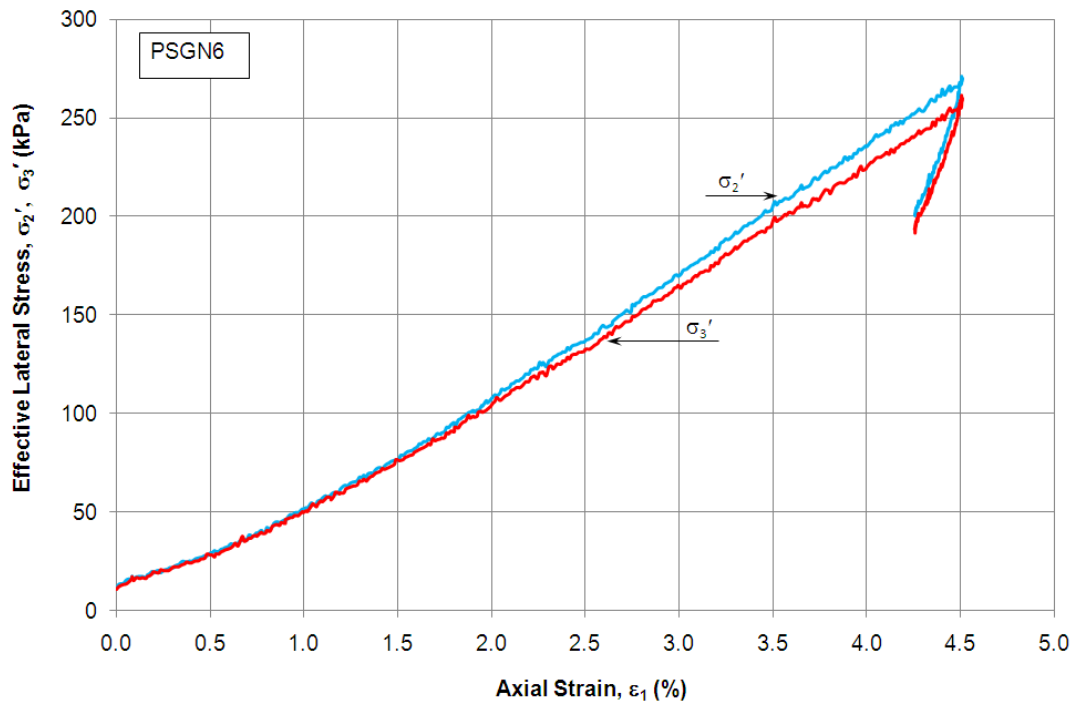


Figure 4.6(b)

Figure 4.6 Variations of Effective Lateral Stress during K_0 Consolidation under Plane-Strain Condition: (a) Test PSGN3; (b) Test PSGN6

From the e - $\log \sigma'_1$ and Δu - $\log \sigma'_1$ curves shown in Figure 4.7(a), a pre-consolidation pressure of 300 kPa was determined. This value is the same as the one obtained under triaxial condition. It is also noted from Figure 4.7(a) that the initial void ratio varied from 0.81 to 0.83. This is due to the natural variation of the intact residual soil.

Similar to the K_0 -behaviour observed under the triaxial condition, a de-structuring process was also observed under plane-strain condition. Two yielding points were identified at 1% and 4% of strain rate, based on the changes of 1-D stiffness shown in Figure 4.7(b). The corresponding zones of behaviour were defined in Figure 4.7. An elastic response can be observed in Figure 4.7(c) at a stress level below σ'_B (in Zone 1). Zone 2 can be considered as a bonding breaking down zone. The de-structuring process associated with large plastic strain development. The soil bonding broke down completely when 2nd yielding occurred at σ'_D .

A cementation index (I_c) of 0.44 was calculated from the effective stress paths shown in Figure 4.7(d). It is slightly lower than the one under triaxial conditions (0.48). This could be due to the variations of void ratio and water content of the intact residual soil. It is also noted from Figure 4.7(d) that the pre-consolidation stress (σ'_p) coincided with the point furthest away from the K_0 line. This is in good agreement with the data obtained under triaxial conditions. It indicated that σ'_p was related to the cementation of the intact residual soil.

The variations of K_0 value during consolidation are presented in Figure 4.7(e). The K_0 value decreased rapidly once consolidation started. It reached the lowest value (0.36-0.38) then bounced back as the soil structures broke down. Finally, the K_0 value reached a constant value of 0.49 at end of K_0 consolidation. The recovery in K_0 value could be related to the destruction of soil bonds.

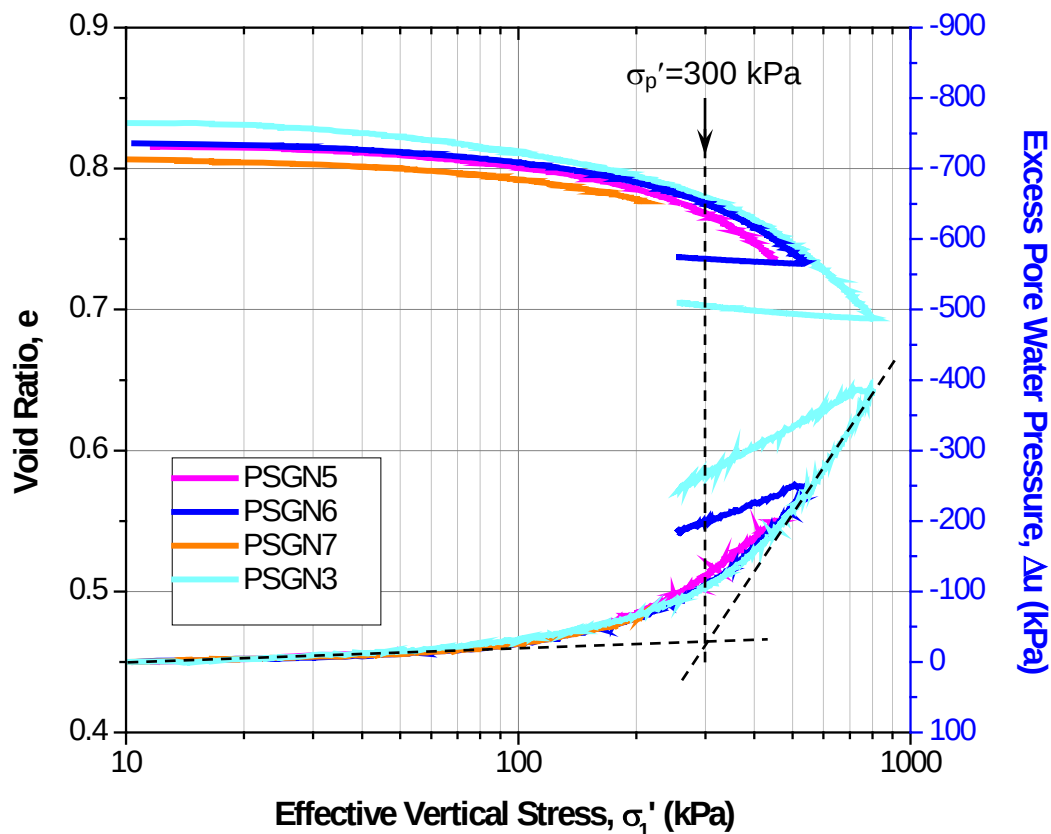


Figure 4.7(a)

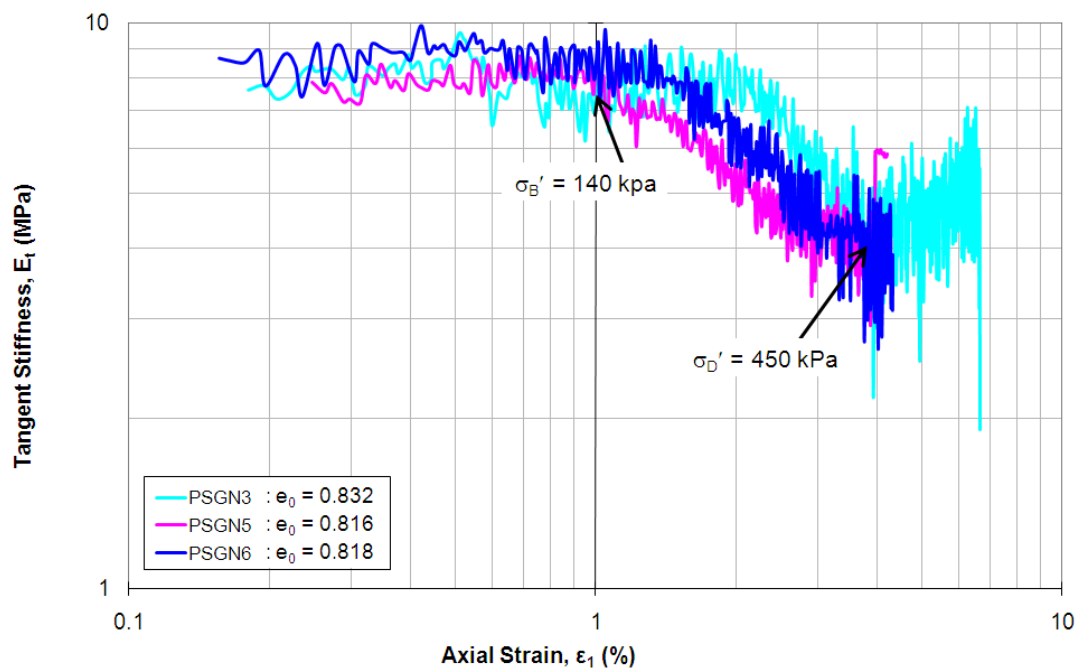


Figure 4.7 (b)

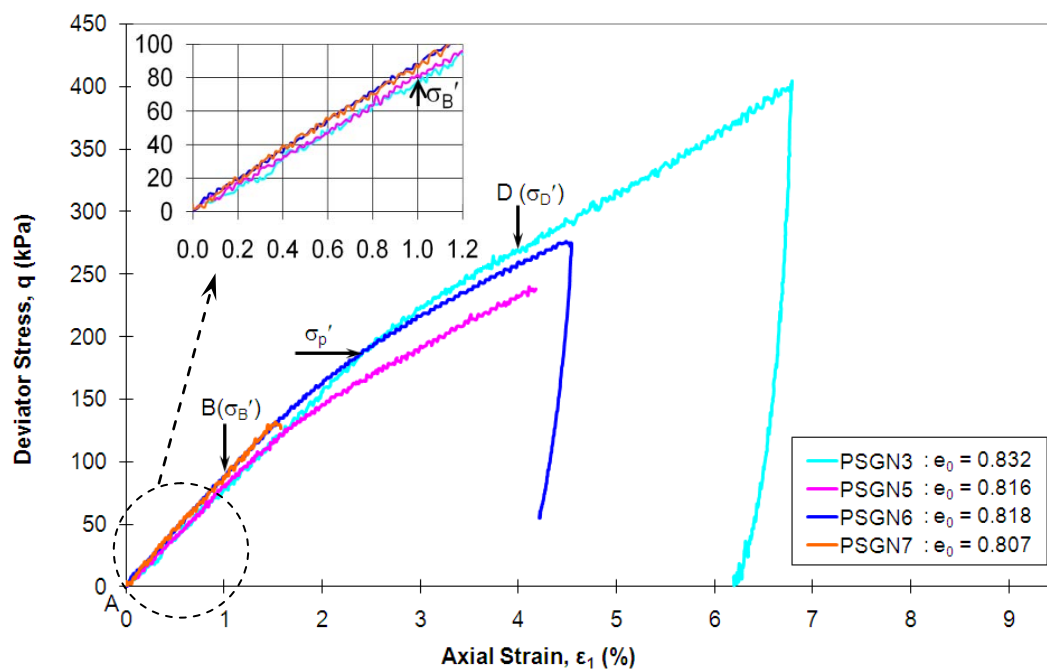


Figure 4.7(c)

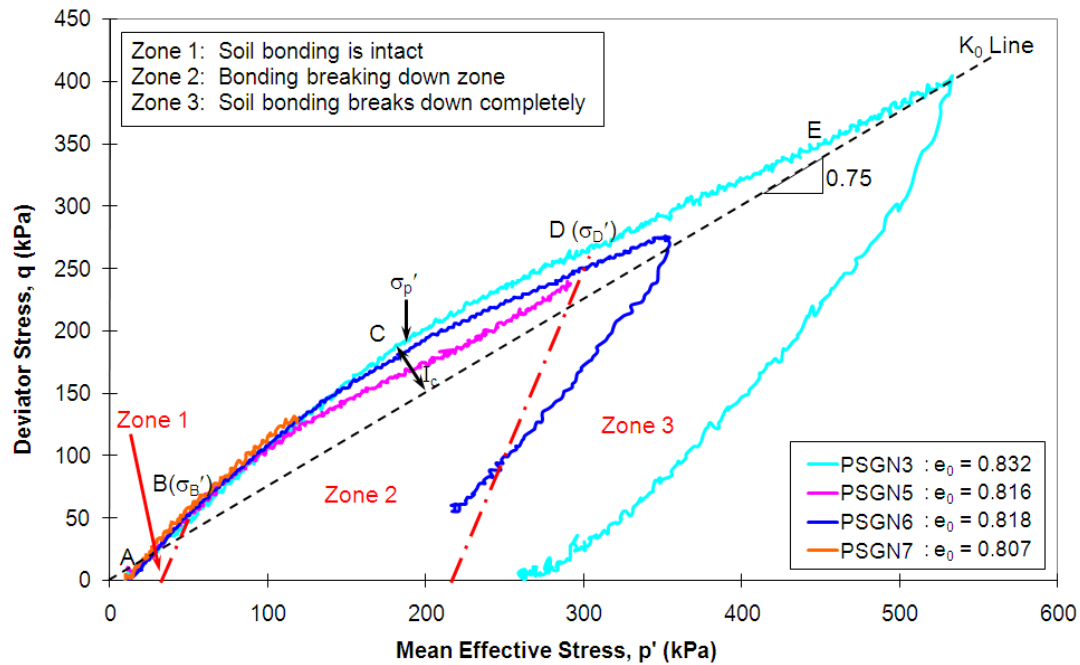


Figure 4.7(d)

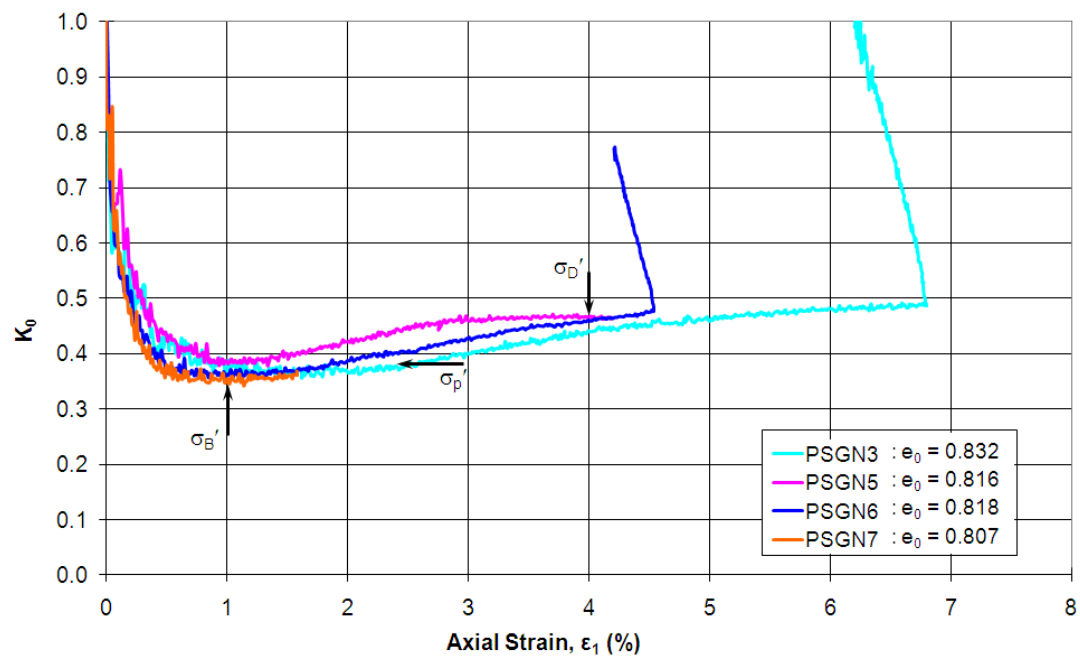


Figure 4.7(e)

Figure 4.7 K_0 Consolidation on Vertically Cut Intact Residual Soil under Plane-Strain Conditions: (a) e and Δu vs. $\log \sigma_1'$ Curves; (b) Et vs. $\log(\varepsilon_1\%)$ Curves; (c) Stress-Strain Curves; (d) Effective Stress Paths; (e) K_0 vs. $\varepsilon_1\%$ Curves

4.2.1.3 Comparisons

K_0 consolidation behaviour of the vertically cut intact residual soil under triaxial and plane-strain condition are compared in Figure 4.8. Very similar behaviour was observed although there were differences in the initial void ratio and water content. For a natural soil, such as the intact residual soil used for this project, it is hard to acquire specimens that are identical, even when block samples were used. It can be seen from Figure 4.8(a), the specimens in triaxial cell were more compressible at low stress level (before pre-consolidation stress). This could be due to the difference in the initial void ratio as shown in Figure 4.8(b). Base on the Δu versus $\log \sigma_1'$ curves, the same pre-consolidation stress (300kPa) was determined under triaxial and plane-strain conditions, as presented in Figure 4.8(c).

The effective stress paths determined from both testing conditions are presented in Figure 4.8(d). All the stress paths reached the same K_0 line with a slope of 0.75. The variation of K_0 values with $\varepsilon_1\%$ is also compared in Figure 4.8(e). Similar behaviour was observed. Under plane-strain condition, the lowest K_0 value reached is smaller. This may be affected by the initial soil conditions.

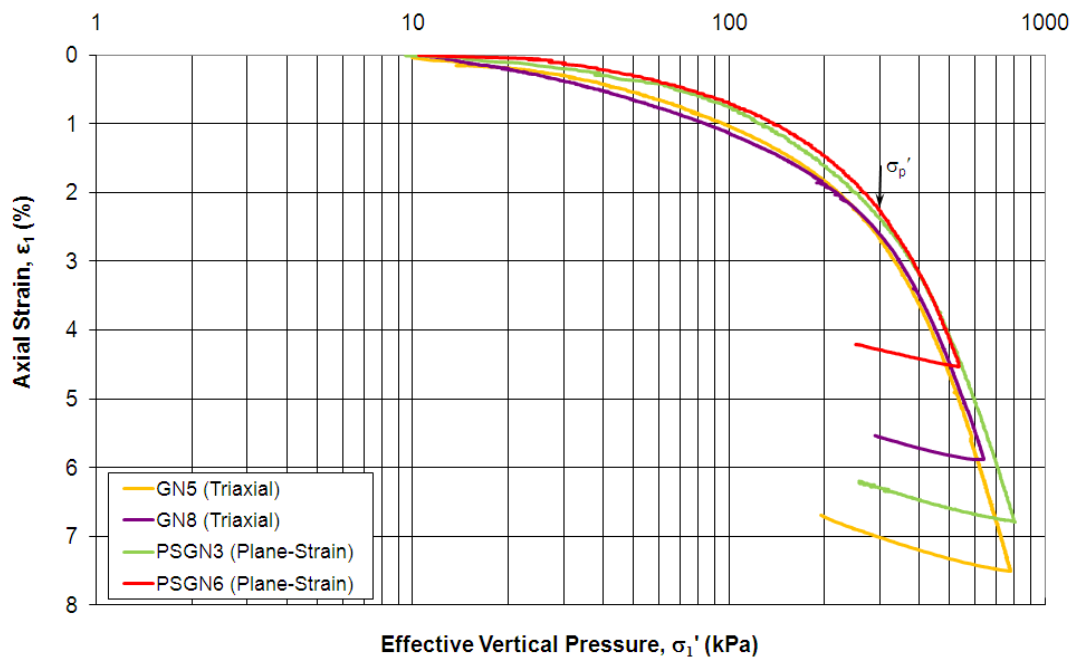


Figure 4.8(a)

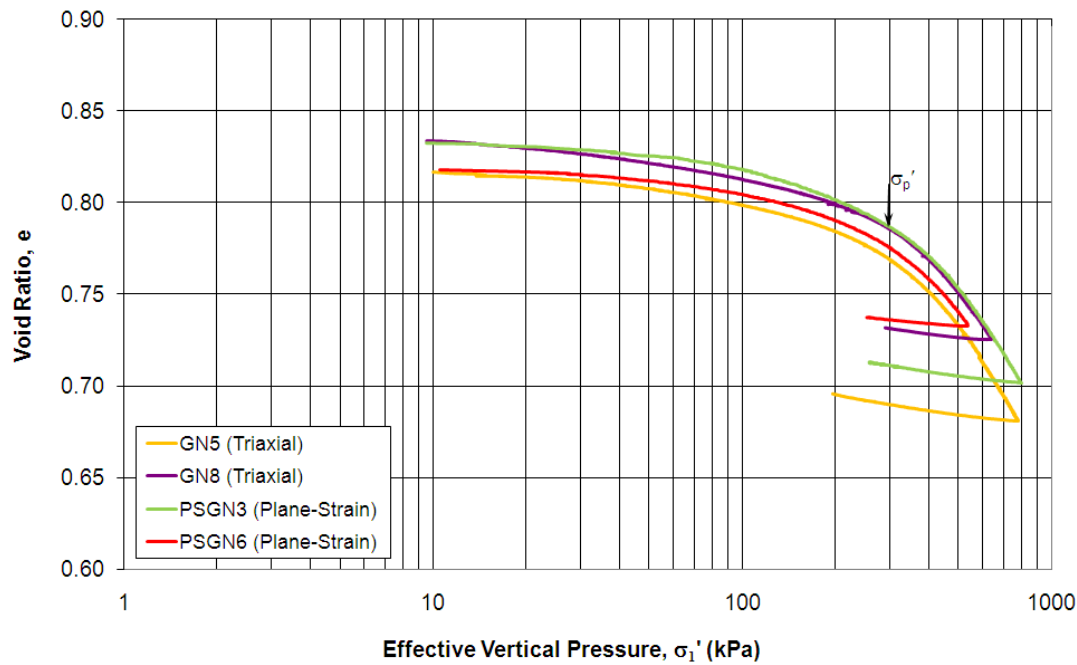


Figure 4.8 (b)

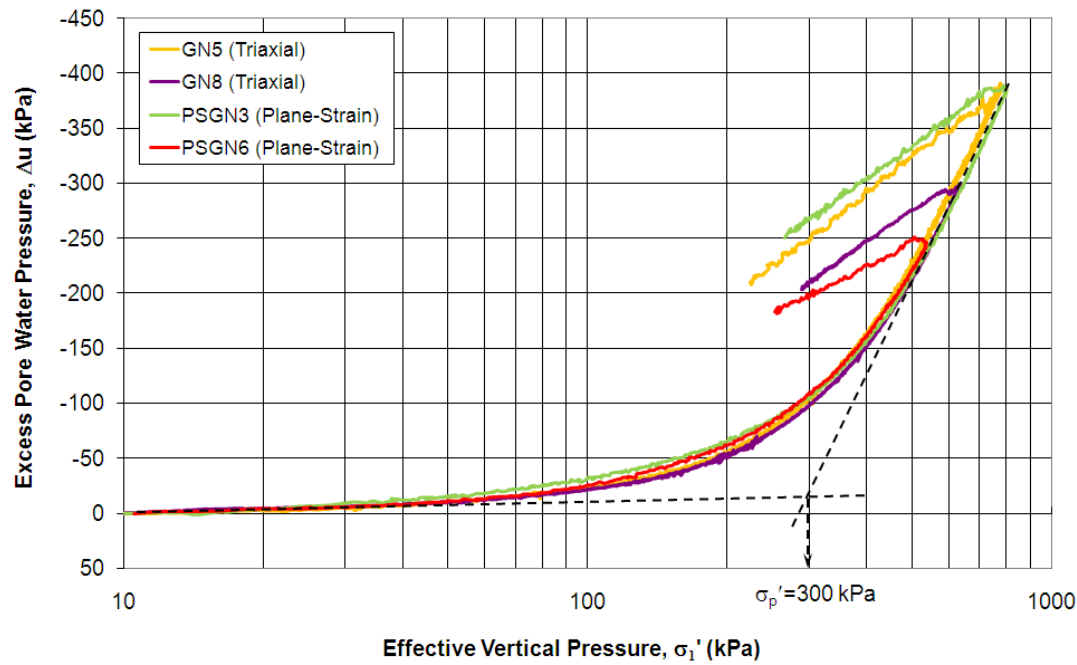


Figure 4.8 (c)

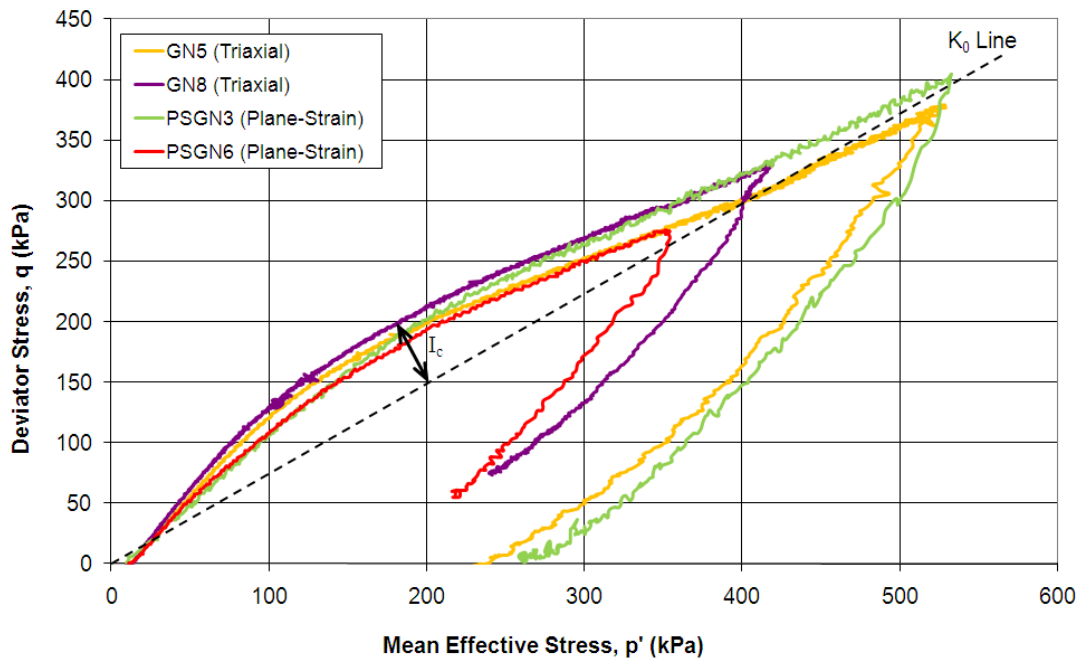


Figure 4.8(d)

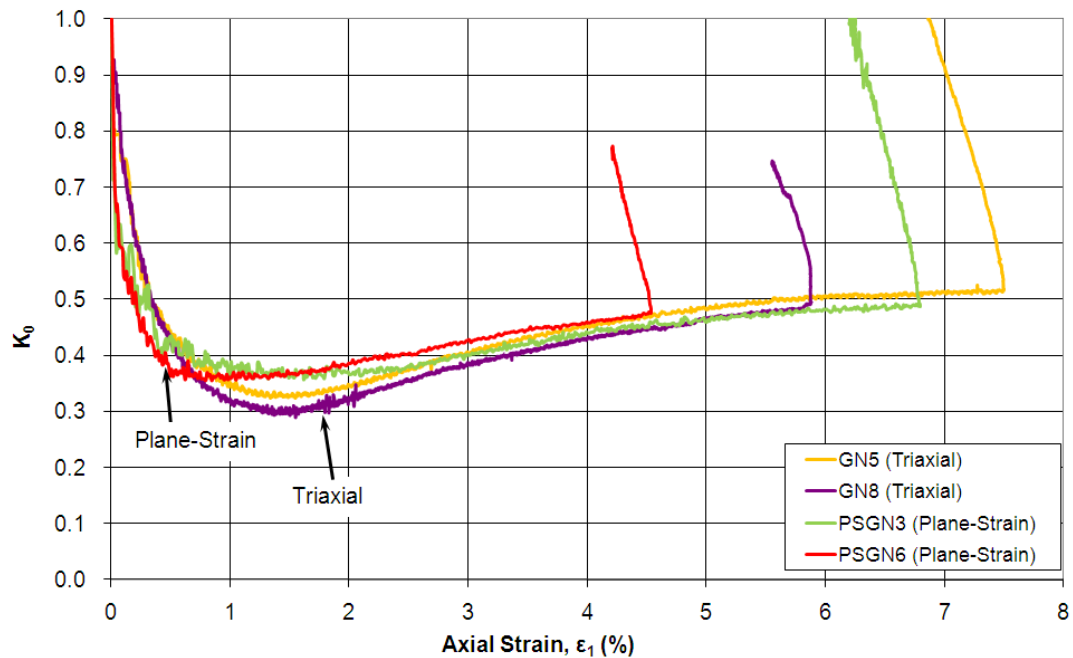


Figure 4.8(e)

Figure 4.8 Comparisons of K_0 Consolidation under Triaxial Condition and Plane-Strain Condition: (a) $\varepsilon_1\%$ vs. $\log \sigma_1'$ Curves; (b) e vs. $\log \sigma_1'$ Curves; (c) Δu vs. $\log \sigma_1'$ Curves; (d) Effective Stress Paths; (e) Variation of K_0 with Axial Strain

In addition, another comparison was made between the predict K_0 value and laboratory measurement. The coefficient of lateral pressure at rest, K_0 , as an important design parameter in geotechnical engineering is usually estimated by using Jaky's equation (Jaky, 1994) for NC clay:

$$K_0 = 1 - \sin\phi' \quad (4-2)$$

where ϕ' is the peak effective friction angle of the soil. In project design, the friction angle at critical state (ϕ'_{cs}) is normally used in the above equation for K_0 estimation. In Table 4.4, the estimated K_0 values using equation (4-2) were compared with the measured ones during K_0 consolidations under both axisymmetric and plane-strain conditions. The determination of ϕ'_{cs} will be explained in Chapter 5. It is noted that the prediction of K_0 values are different for axisymmetric conditions. However, the values measured from K_0 consolidation tests are almost the same for both conditions. It is also observed that the prediction using equation (4-2) is quite consistent with the experimental measurement. The error is less than 10%.

Table 4.4 Comparisons of K_0 Values for Intact Residual Soil

Testing Condition	$K_0=1-\sin\phi'_{cs}$	Measurement	Difference
Axisymmetric / Triaxial	0.4523	0.5	-9.54 %
Plane-Strain	0.4828	0.49	-1.47 %

4.2.2 Horizontally Cut Specimens

To study the strength anisotropy of the intact residual soil, a series of CK₀UC and CK₀UE tests were conducted on horizontally cut specimens. The consolidation behaviour of these specimens is presented in Figure 4.9. The details of the K_0 consolidated tests can be found in Table 4.5. The cutting orientation of intact residual soil was illustrated in Figure 3.4.

Table 4.5 Summary of K_0 Consolidation Tests Conducted on Horizontally Cut Intact Residual Soil under Triaxial Conditions

Test No.		GN3H	EGN3H	EGN4H
Diameter (mm)		70	70	70
Height (mm)		140	140	140
Weight (g)		1018.14	1024.07	1035.06
Water Content (%)		30.83	31.72	31.39
Initial Void Ratio, e_0		0.837	0.841	0.844
Type of Test		CK ₀ UC	CK ₀ UE	CK ₀ UE
Pressure for Saturation (kPa)	cell	860	1010	1010
	back	850	1000	1000
Consolidation Rate (mm/min)		0.007	0.007	0.007
Maximum Consolidation Pressure (kPa)	σ_1'	445.07	781.29	447.17
	q	240.57	385.29	248.67
	p'	283.69	524.43	281.39

The e vs. $\log \sigma_1'$ and Δu vs. $\log \sigma_1'$ curves obtained from K_0 consolidation tests on horizontally cut residual soil specimens are presented in Figure 4.9(a). The pre-consolidation pressure on horizontal direction was about 300 kPa. This value is the same as the one on the vertical direction. This indicated that the intact residual soil of Bukit Timah granite is isotropically consolidated during formation. No inherent strength anisotropy was observed.

The stress-strain behaviour established from K_0 consolidation on horizontally cut intact residual soil is presented in Figure 4.9(b). Differences in the stress-strain curves were observed within the elastic response range (before σ_B'). This could be due to the initial soil conditions. The low water content and low void ratio of specimen GN3H led to a stiff soil behaviour, which consistent with a higher stress-strain curves as shown in the enlarge plot on Figure 4.9(a).

The tangent stiffness was determined from the stress-strain curves, and presented in Figure 4.9(c). Two yielding points were obtained at 0.8% and 4% of axial strain. Similar to the test results of vertically cut intact residual soil, the effective stress paths were divided into three zones of behaviour to the two yielding points, as

shown in Figure 4.9(d). In each zone, the specimens in different cutting orientation behaved similarly. The cementation index (I_c) was calculated for horizontally cut intact residual soil. It had a value of 0.5, which was almost the same as the value for vertically cut intact residual (0.48). This indicated that the I_c value is not sensitive to the cutting orientation of soil specimens.

Figure 4.9(e) and (f) present the variations of K_0 value with $\varepsilon_1\%$ and σ_1' respectively. The K_0 values were both stress and strain dependent. It decreased with increasing $\varepsilon_1\%$ and σ_1' , and reached an ultimate value of 0.51 after the soil bonds broke down completely.

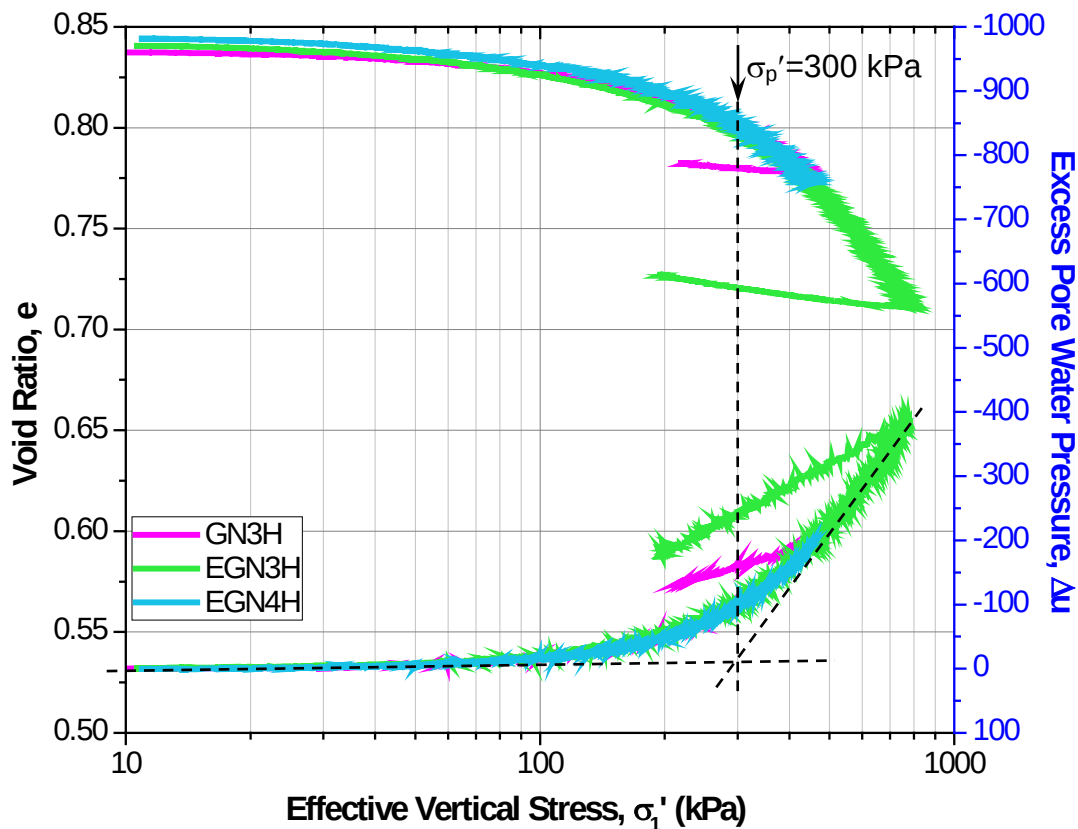


Figure 4.9(a)

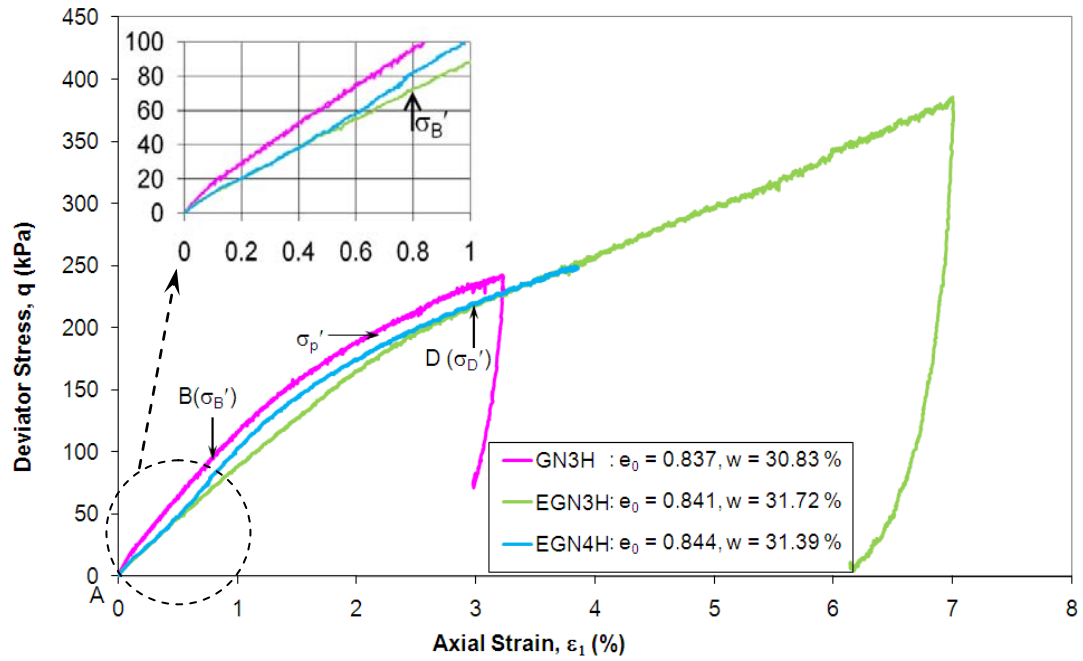


Figure 4.9 (b)

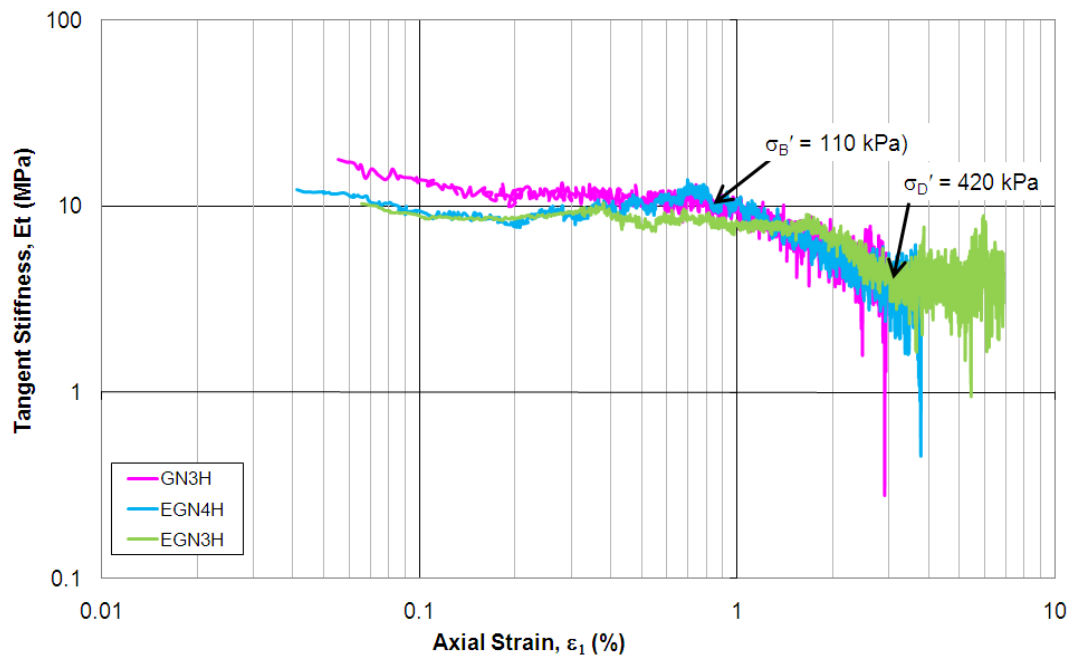


Figure 4.9(c)

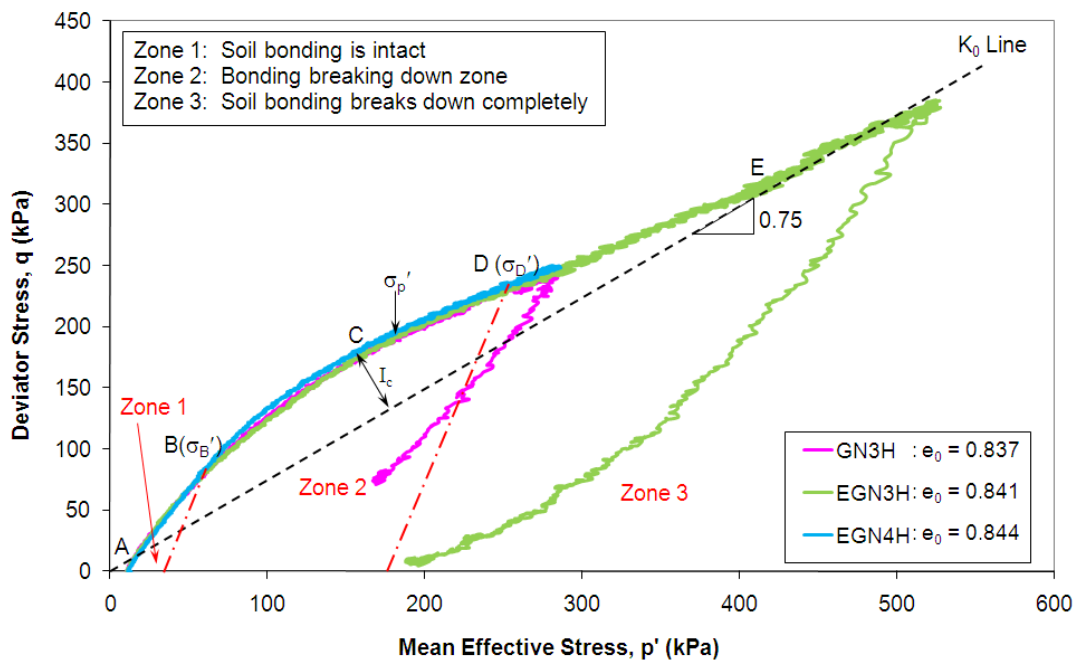


Figure 4.9(d)

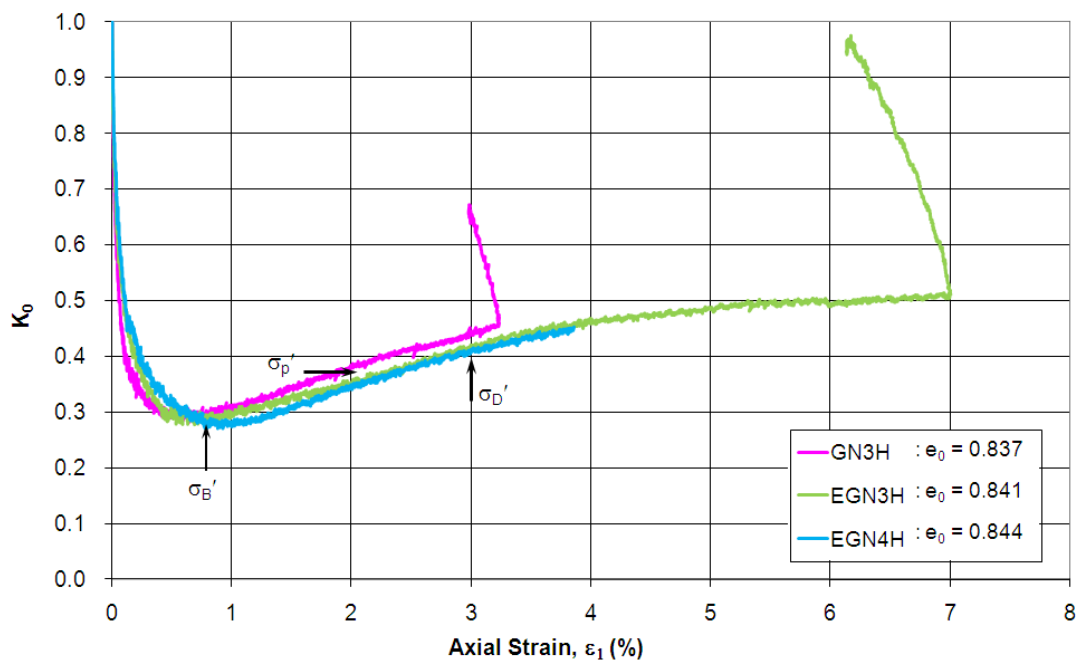


Figure 4.9(e)

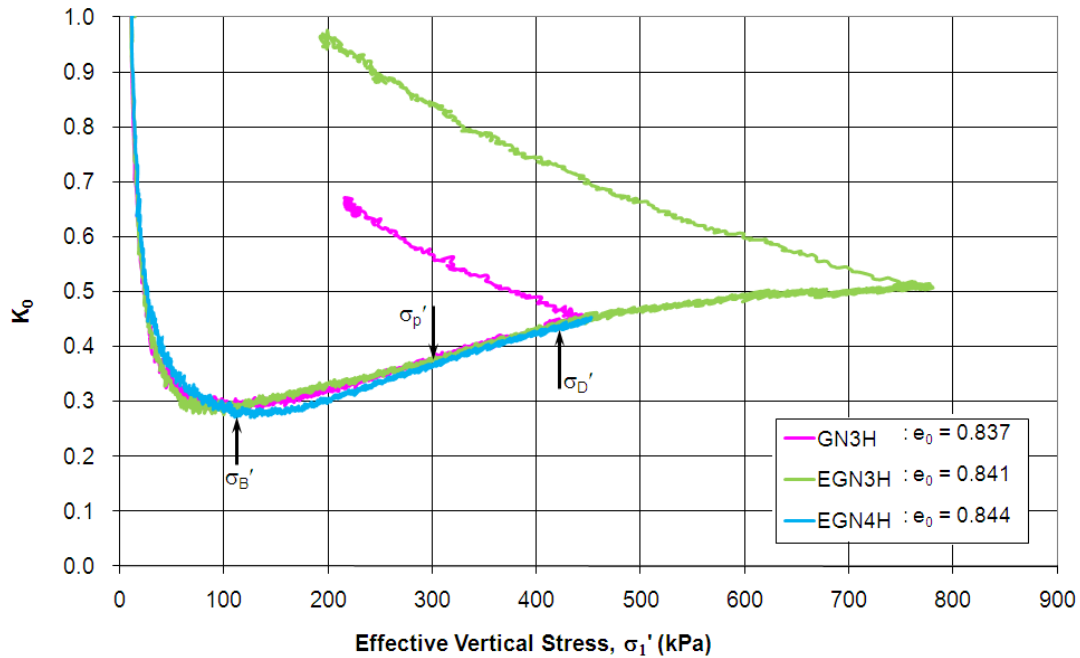


Figure 4.9(f)

Figure 4.9 K_0 Consolidation on Horizontally Cut Intact Residual Soil under Triaxial Condition: (a) $\varepsilon_1\%$ vs. $\log \sigma_1'$ Curves; (b) e vs. $\log \sigma_1'$ Curves; (c) Stress-Strain Curves; (d) E_t vs. $\log(\varepsilon_1\%)$ Curves; (e) Effective Stress Paths; (f) K_0 vs. $\varepsilon_1\%$ Curves; (g) K_0 vs. σ_1' Curves

4.2.3 Comparisons of Behaviour of Vertically Cut and Horizontally Cut Specimens

The consolidation behaviour of vertically cut and horizontally cut intact residual soil specimens is compared in Figure 4.10. As determined from Figure 4.10(a) that the pre-consolidation stress in both vertical and horizontal direction is the same. It is noted from Figure 4.10(b) that the vertically cut intact residual soil specimen was more compressible than horizontally cut specimens. This may be affected by the initial void ratios of intact residual soil, as presented in Figure 4.10(c).

Figure 4.10(d) compares the effective stress paths determined from K_0 consolidation tests on vertically cut and horizontally cut intact residual soil specimens. No significant difference was observed. All the effective stress paths reached a unique

K_0 line with a slope of 0.75. The cementation index for both vertically cut and horizontally cut specimens was the same (about 0.5). This indicated that the effective stress paths are independent of cutting orientation of specimens and the variation of initial void ratio.

The variations K_0 are also compared between vertically cut and horizontally cut intact residual soil specimens. As shown in Figure 4.10(e), the horizontally cut specimens reached the lowest K_0 at a smaller axial strain (0.7%) than vertically cut specimen (1.3%). Although a same ultimate K_0 value (0.5) was achieved, the horizontally cut specimen reached the ultimate value much faster.

The yielding stresses determined for specimens in different cutting orientations under triaxial and plane-strain conditions are compared in Figure 4.6. It is concluded that within a close range of I_c , no significant difference was observed between the yielding stresses.

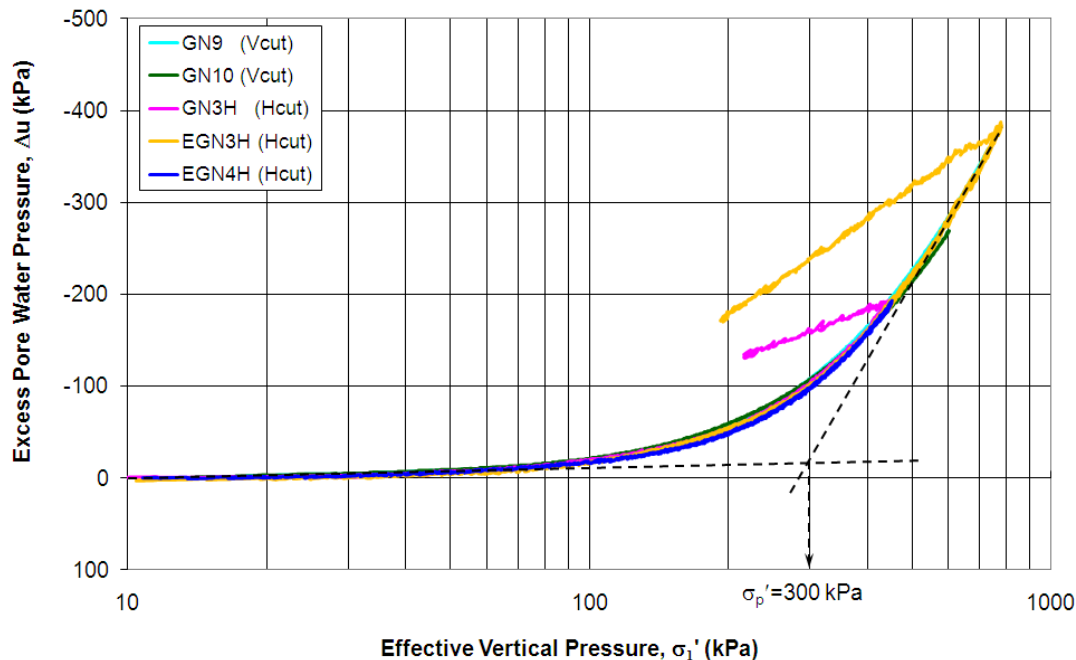


Figure 4.10 (a)

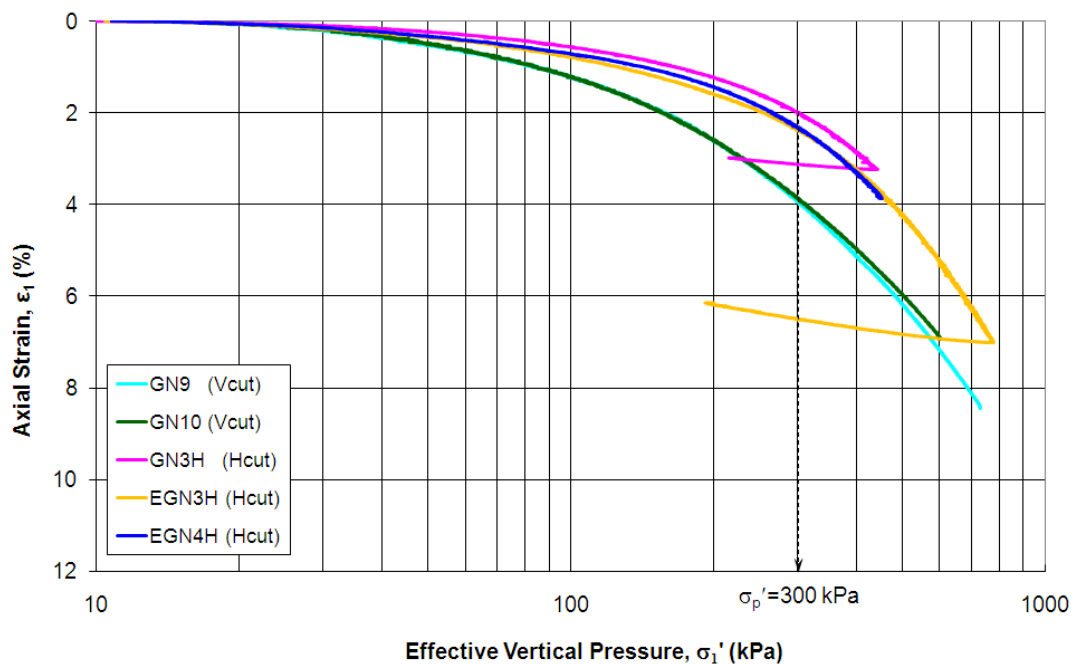


Figure 4.10(b)

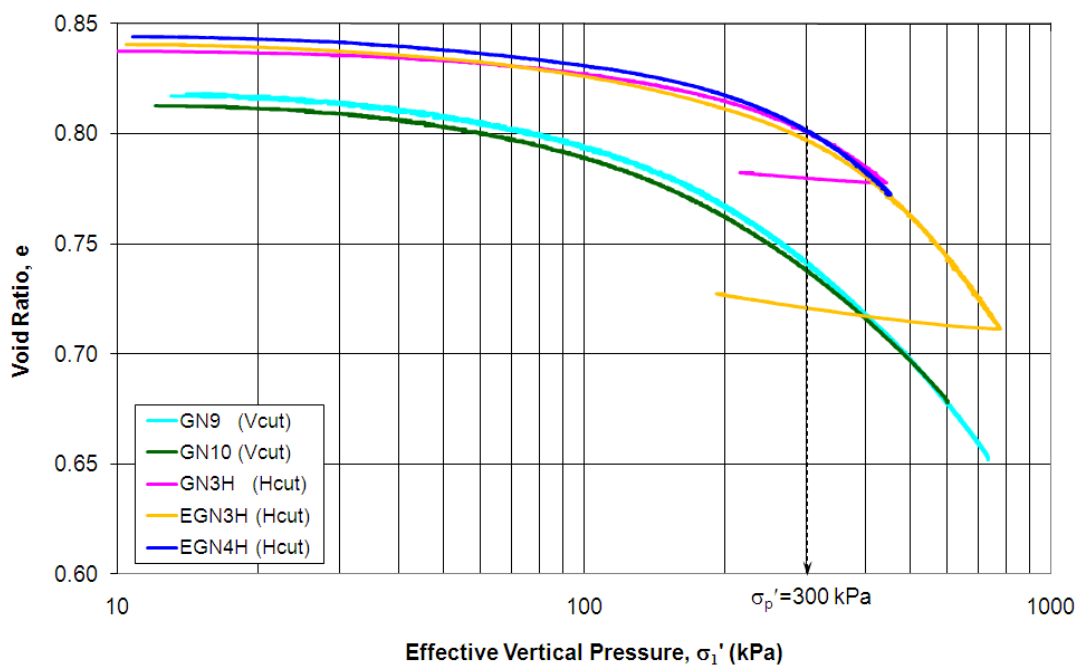


Figure 4.10(c)

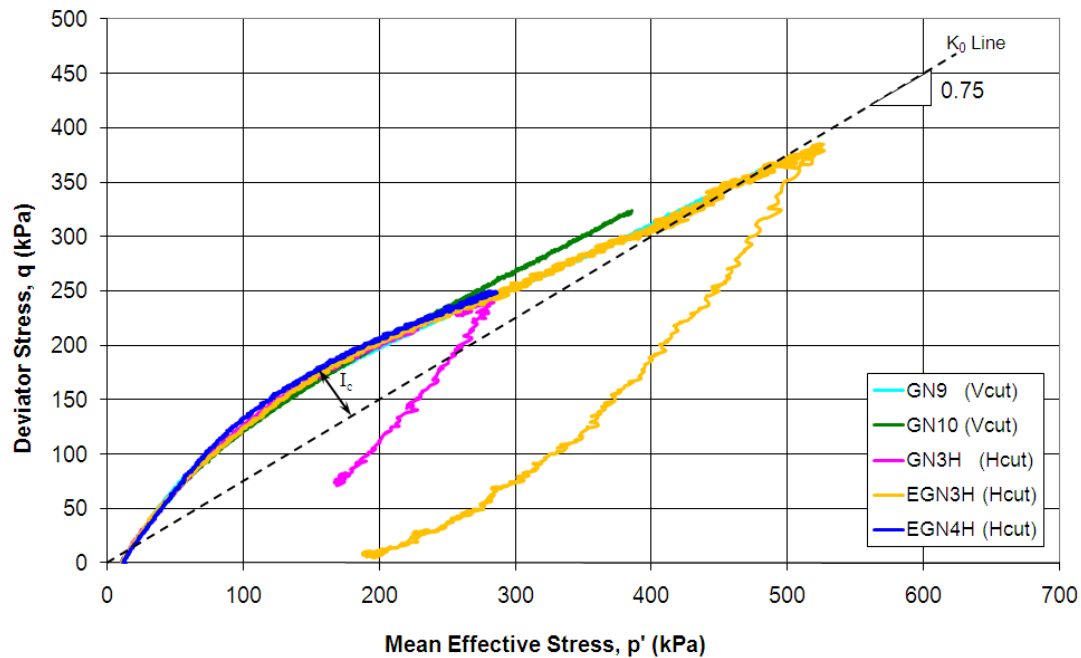


Figure 4.10(d)

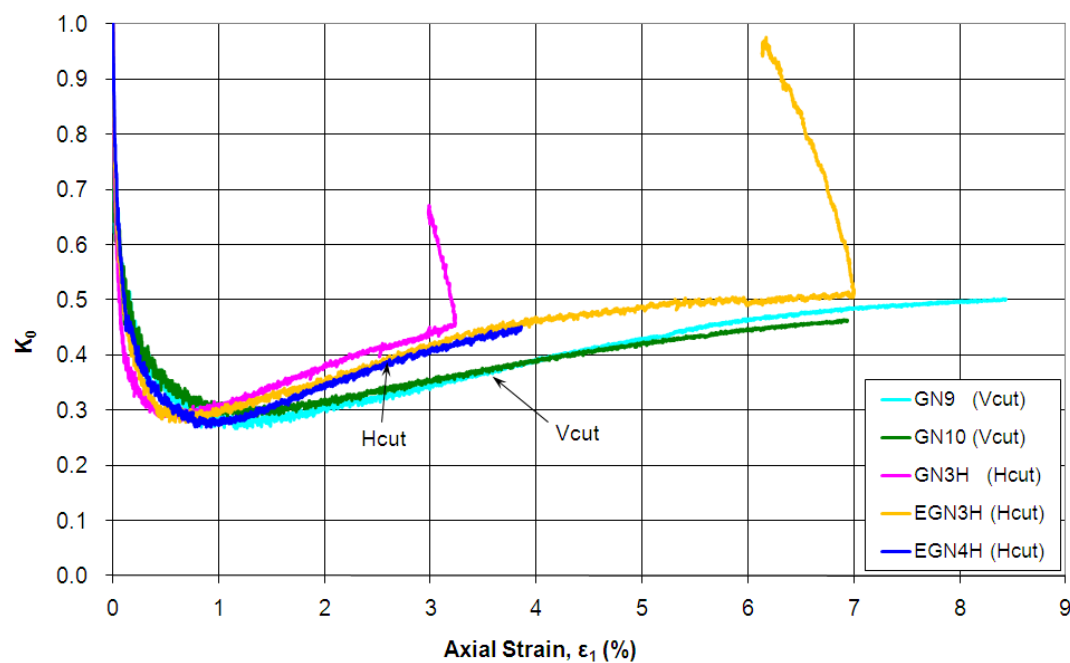


Figure 4.10 (e)

Figure 4.10 Comparisons of K_0 Consolidation on Vertically Cut and Horizontally Cut Intact Residual Soil under Triaxial Condition: (a) Δu vs. $\log \sigma_1'$ Curves; (b) $\varepsilon_1\%$ vs. $\log \sigma_1'$ Curves; (c) e vs. $\log \sigma_1'$ Curves; (d) Effective Stress Paths; (e) K_0 vs. $\varepsilon_1\%$ Curves

Table 4.6 Comparison of Yielding Stresses

Testing Conditions	Cutting Orientation	I_c	σ'_B (kPa)	σ'_D (kPa)
Triaxial	Vertical	0.50	110	440
Triaxial	Horizontal	0.48	110	420
Plane-Strain	Vertical	0.44	140	450

4.3 K_0 Consolidation of Compacted Residual Soil

4.3.1 Using Triaxial Cell

In this study, the trimming of the intact residual soil was compacted and tested to investigate the deformation and instability behaviour of the residual soil of Bukit Timah Formation. The compaction process can provide more homogeneous specimens than the intact residual soil. However, since the specimens were compacted individually, it was hard to achieve identical void ratio for each specimen.

The K_0 consolidation behaviour of compacted residual soil under triaxial condition is presented in Figure 4.11. The tests conducted are summarized in Table 4.7.

The pre-consolidation stress (σ'_p) was determined from the e and Δu versus $\log \sigma'_1$ curves as mentioned earlier, which is 300 kPa as shown in Figure 4.11(a). The initial void ratio varied from 0.73 to 0.76, this was due to the specimen compaction process and also the variation of water content of soil mixture. The consistency in the shapes of e vs. $\log \sigma'_1$ curves eliminated the possibility of testing errors.

The stress-strain curves determined from consolidation tests on compacted residual soil are presented in Figure 4.11(b). The deviator stress increased gradually with axial strain. The difference in the stress-strain curves could be caused by the variations in void ratio and water content.

The effective stress paths are shown in Figure 4.11(c). After a short transition from isotropic state to K_0 state, the effective stress paths remained on the K_0 line thereafter. Since the natural structure of residual soil was destroyed during the reconstitution process, the effective stress paths were not beyond the K_0 line as what intact residual soil did. No cementation index (I_c) was calculated for compacted residual soil, since the distance between the effective stress path and K_0 line was small enough to be ignored. The absence of soil bonds was also verified by K_0 vs. ε_1 % curves presented in Figure 4.11(d). As the K_0 -path initiated from an isotropic state ($\sigma_3' = \sigma_1' = 10$ kPa), the curve started from $K_0 = 1$ and dropped rapidly at the beginning. It maintained at an ultimate value of 0.44 after reaching the trough (0.36-0.41). The difference between the lowest K_0 and the ultimate value (0.03-0.08) was much smaller than the one obtained for the intact residual soil under the same testing conditions (0.17-0.25). This is an indication of lack of soil bonds in the compacted residual soil.

Table 4.7 Summary of K_0 Consolidation Tests Conducted on Compacted Residual Soil under Triaxial Conditions

Test No.		TXLQ1	TXLQ3	CGN1	CGN2
Diameter (mm)		50	50	50	50
Height (mm)		100	100	100	100
Weight (g)		378.94	377.51	381.25	379.69
Water Content (%)		22.94	22.69	24.32	23.94
Initial Void Ratio, e_0		0.73	0.76	0.74	0.75
Type of Test		CSD	CSD	CK ₀ UC	CK ₀ UC
Pressure for Saturation (kPa)	Cell	860	860	860	860
	Back	850	850	850	850
Consolidation Rate (mm/min)		0.005	0.005	0.005	0.005
Maximum Consolidation Pressure (kPa)	σ_1'	591.02	623.22	786.94	958.76
	q	330.02	341.22	434.94	534.76
	p'	371.01	395.74	496.98	602.25

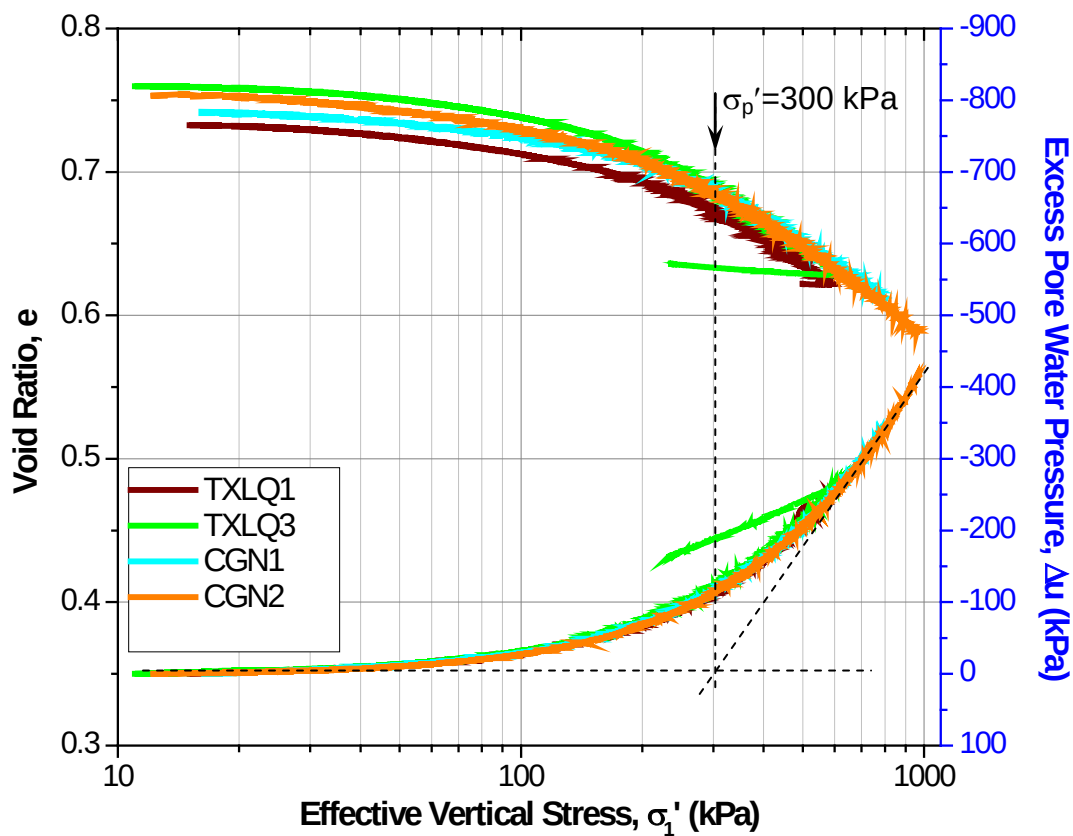


Figure 4.11(a)

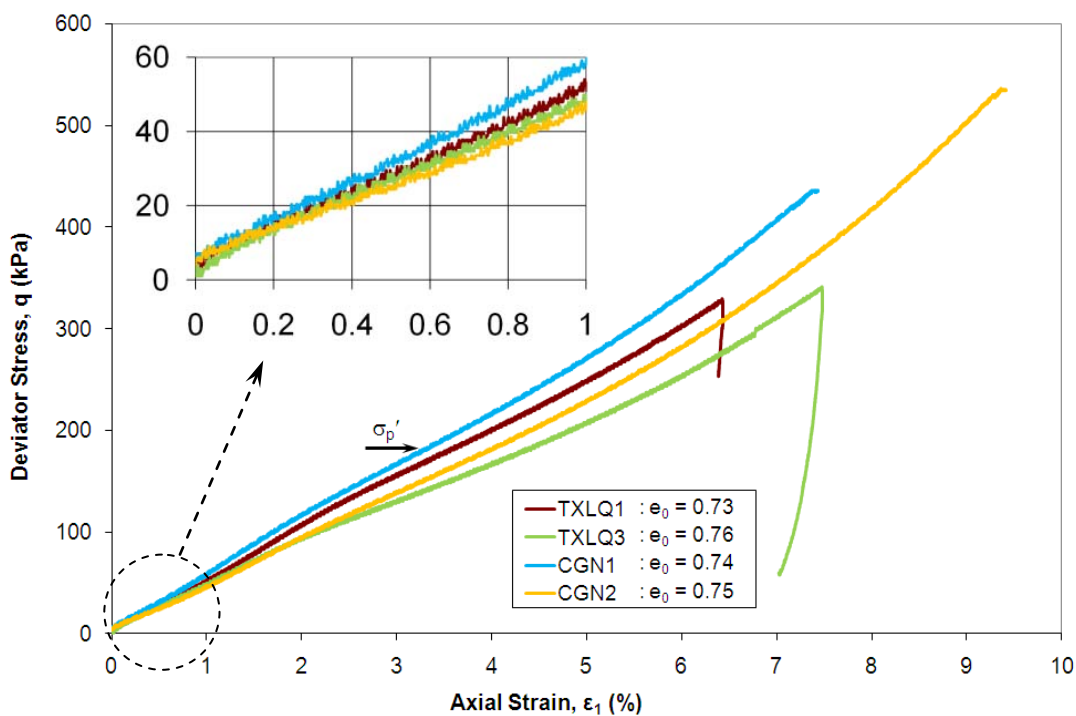


Figure 4.11 (b)

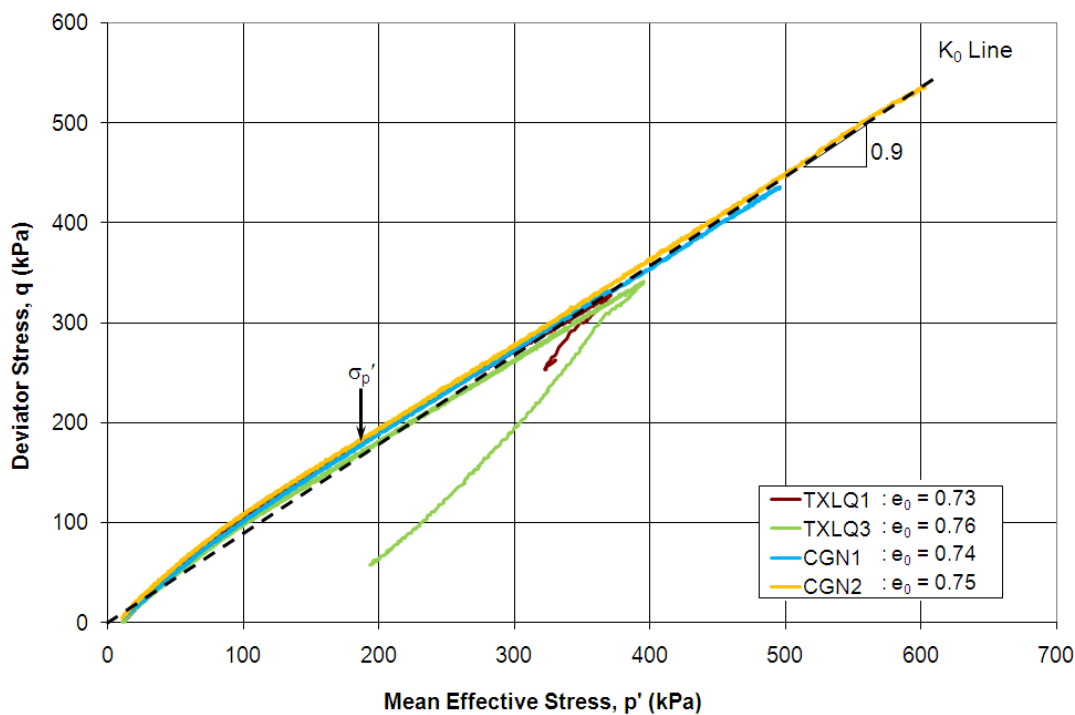


Figure 4.11(c)

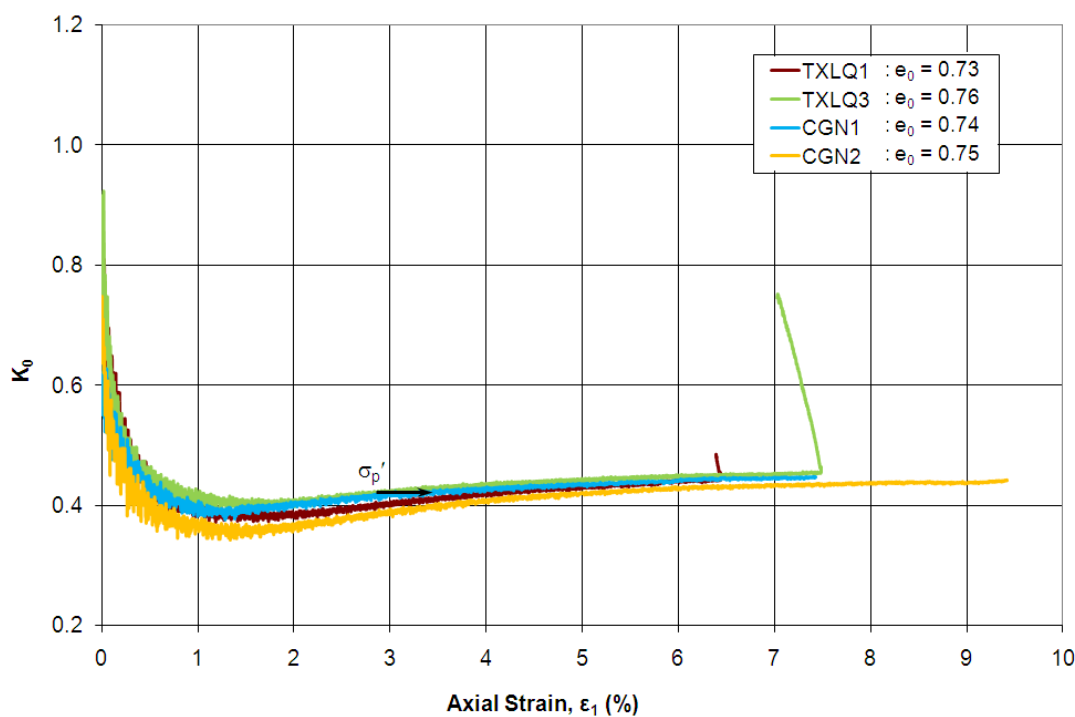


Figure 4.11(d)

Figure 4.11 K_0 Consolidation on Compacted Residual Soil under Triaxial Condition: (a) e and Δu versus $\log \sigma_1'$ Curves; (b) Stress-Strain Curves; (c) Effective Stress Paths; (d) K_0 versus $\epsilon_1\%$ Curves

A comparison between the estimated K_0 value and measured one is shown in Table 4.8. Similar K_0 were obtained from different estimation approaches. This is consistent with the observation made earlier on the intact residual soil.

Table 4.8 Comparison of K_0 Values for Compacted Residual Soil

Testing Condition	$K_0=1-\sin\phi'_{cs}$	Measurement	Difference
Triaxial	0.449	0.44	2.04 %

4.3.2 Using Plane-Strain Apparatuses

To compare the strength and deformation behaviour under axisymmetric and plane-strain conditions, a series of CK_0 UPS tests were also conducted on the compacted residual soil specimens. Table 4.9 summarized the K_0 consolidation tests conducted by using plane-strain apparatus. The consolidation data are presented as following in Figure 4.13.

Table 4.9 Summary of K_0 Consolidation Tests Conducted on Compacted Residual Soil under Plane-Strain Conditions

Test No.		HFGN3	HFGN5	HFGN6
Cross-Sectional Area (mm)		60×60	60×60	60×60
Height (mm)		120	120	120
Weight (g)		825.63	829.51	834.1
Water Content (%)		23.24	23.82	23.51
Initial Void Ratio, e_0		0.72	0.76	0.76
Type of Test		CSD	CSD	CSD
Pressure for Saturation (kPa)	Cell	860	860	860
	Back	850	850	850
Consolidation Rate (mm/min)		0.01	0.01	0.01
Maximum Consolidation Pressure (kPa)	σ'_1	779.61	315.94	136.72
	q	426.45	185.75	85.90
	p'	495.57	192.11	79.51

The lateral pressure, σ_2 was measured directly during the K_0 consolidation by plane-strain apparatus. The σ_2' versus $\varepsilon_1\%$ and σ_3' versus $\varepsilon_1\%$ curves obtained from each test are presented in Figure 4.12. It was observed that σ_2' was almost identical with σ_3' during K_0 consolidation under plane-strain condition. The consolidation behaviour should be consistent under triaxial and plane-strain conditions. The estimation will be verified later in Section 4.3.3.

A pre-consolidation stress of 300 kPa was determined as shown in Figure 4.13(a), which is consistent with the test data obtained from triaxial cell. The difference in initial void ratio is due to the variation of water content of the soil mixture, since the compaction process was only controlled by the volume of specimen. The variation of initial void ratio also happened on compacted triaxial specimens. The stress-strain curves determined under plane-strain conditions are presented in Figure 4.13(a). Similar to the tests conducted using triaxial cell, the deviator stress increased almost linearly with axial strain. Figure 4.13(b) presents the effective stress paths determined under plane-strain conditions. It is noted that the effective stress paths migrated to the K_0 line after the transition phase from an isotropic state. The K_0 value tended to be constant after the insignificant trough as shown in Figure 4.13(c). No destruction of the soil bonds was noted on compacted residual soil.

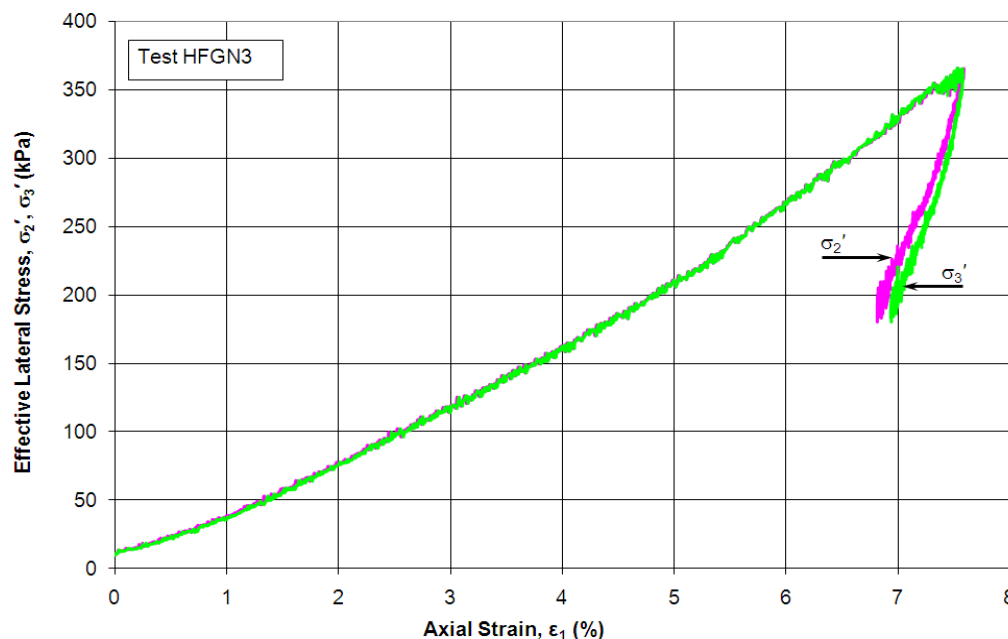


Figure 4.12(a)

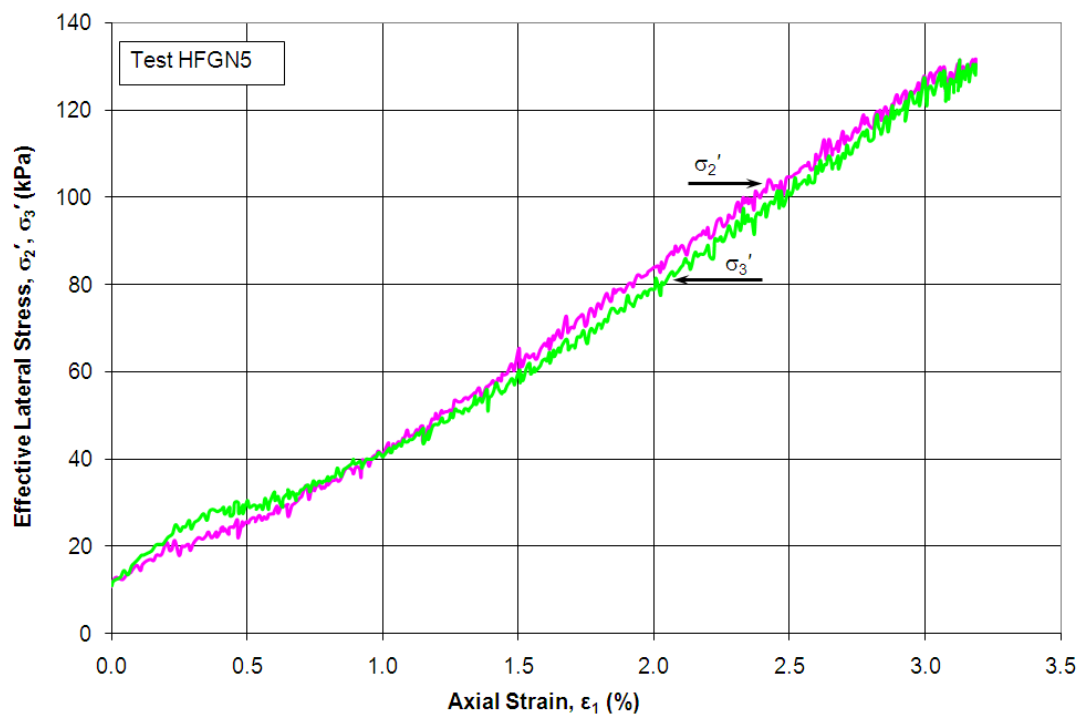


Figure 4.12(b)

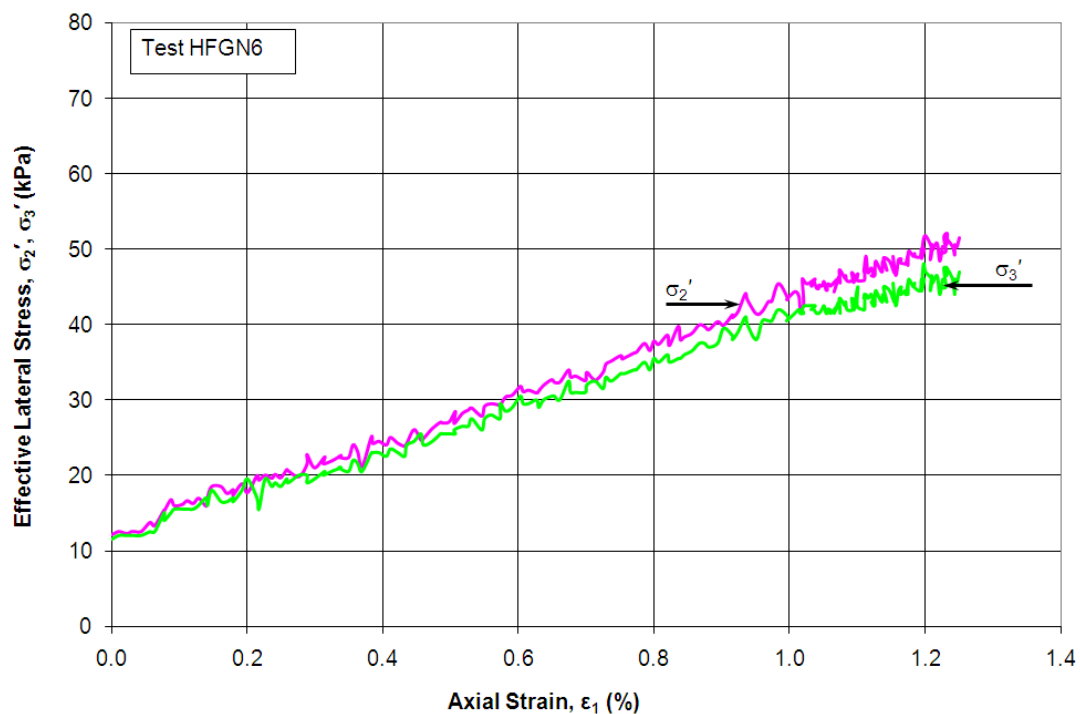


Figure 4.12 (c)

Figure 4.12 Variations of Effective Lateral Stress during K_0 Consolidation under Plane-Strain Condition: (a) Test HFGN3; (b) Test HFGN5; (c) Test HFGN6

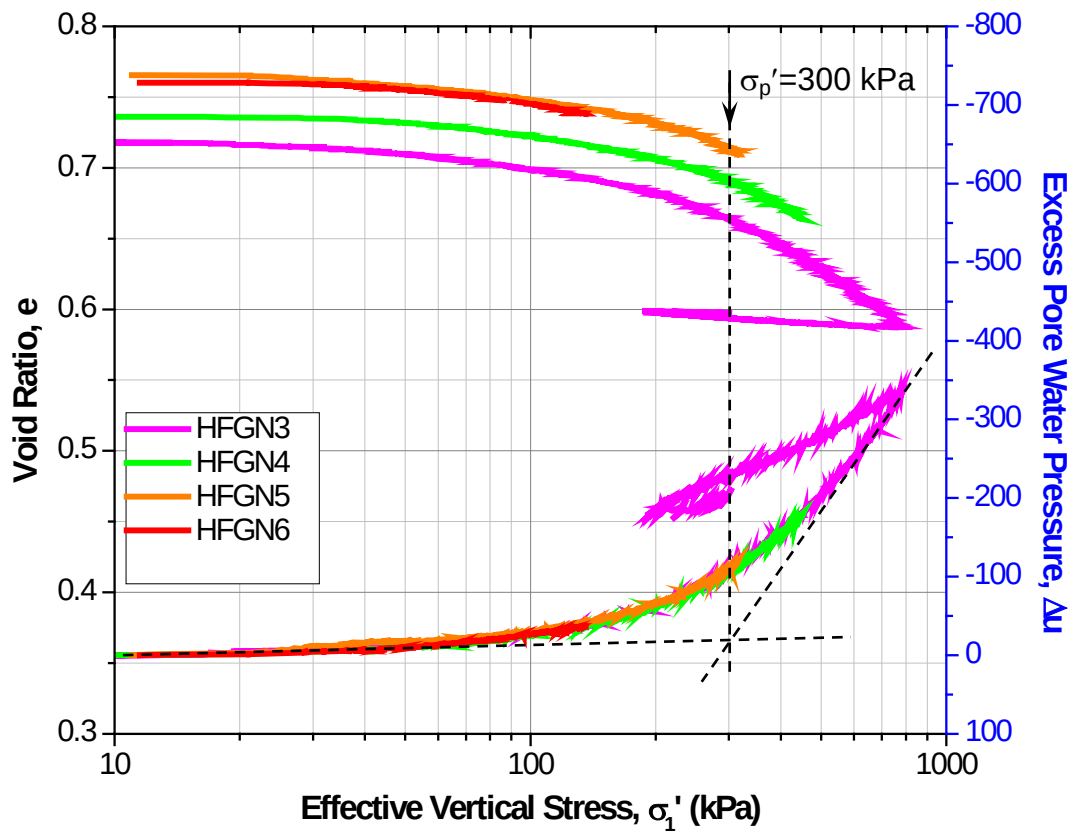


Figure 4.13(a)

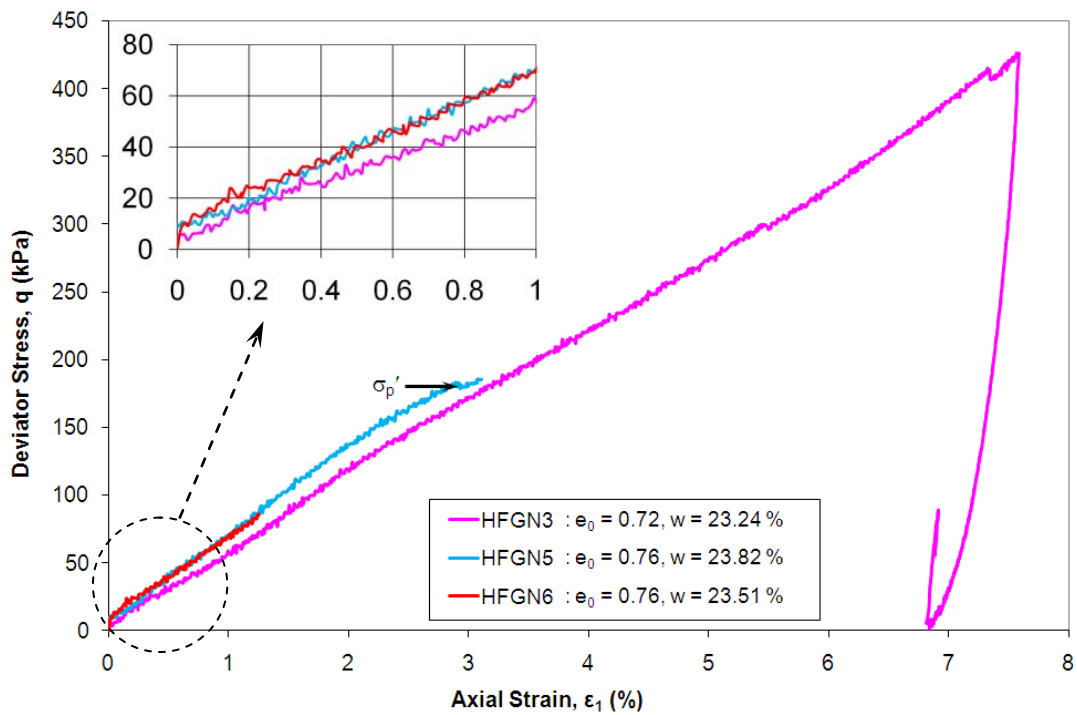


Figure 4.13 (b)

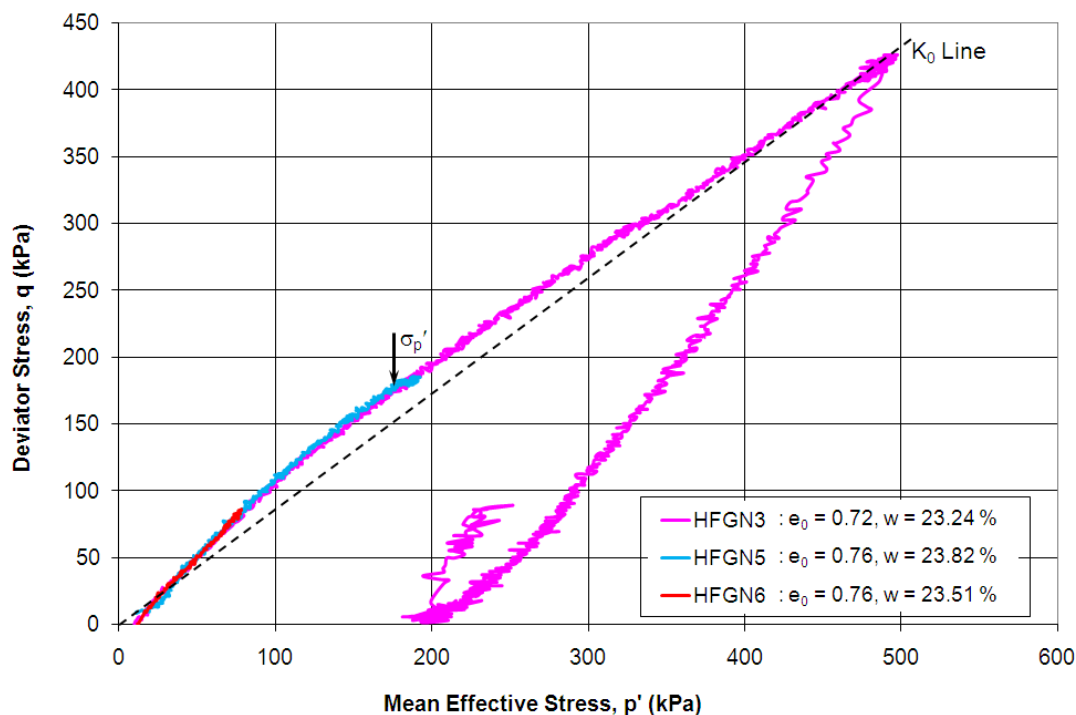


Figure 4.13(c)

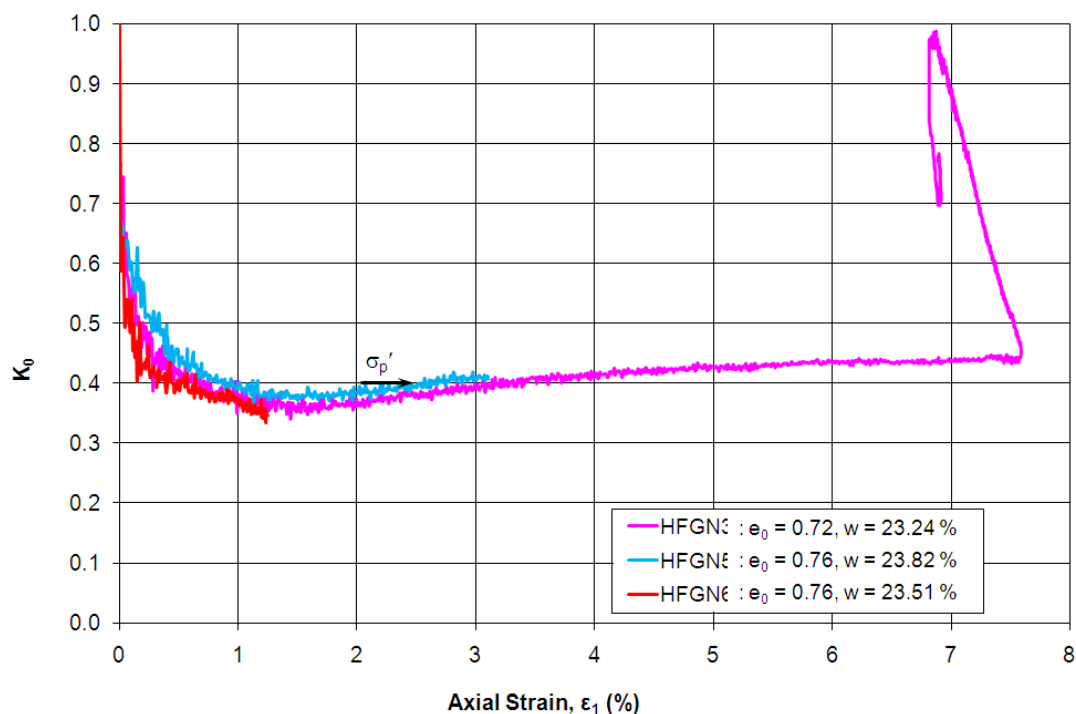


Figure 4.13(d)

Figure 4.13 K_0 Consolidation on Compacted Residual Soil under Plane-Strain Condition: (a) e and Δu vs. $\log \sigma_1'$ Curves; (b) Stress-Strain Curves; (c) Effective Stress Paths; (d) K_0 vs. $\epsilon_1\%$ Curves

4.3.3 Comparisons

The consolidation behaviour of compacted residual soil under triaxial and plane-strain conditions is compared in Figure 4.14. The Δu versus $\log \sigma_1'$ and $\varepsilon_1\%$ versus $\log \sigma_1'$ curves were consistent under both testing conditions, as shown in Figure 4.14(a) and (b). The e versus $\log \sigma_1'$ curves presented in Figure 4.14(c) were in the same shape. The difference in the initial void ratio may be affected by the variation in the water content of soil.

As shown in Figure 4.14(d) and (e), the effective stress paths and the K_0 versus $\varepsilon_1\%$ curves were consistent under both triaxial and plane-strain conditions. The cementation index (I_c) as proposed for cementation measurement was small enough to be ignored for the compacted residual soil. No sign of de-structure process was observed under both triaxial and plane-strain conditions.

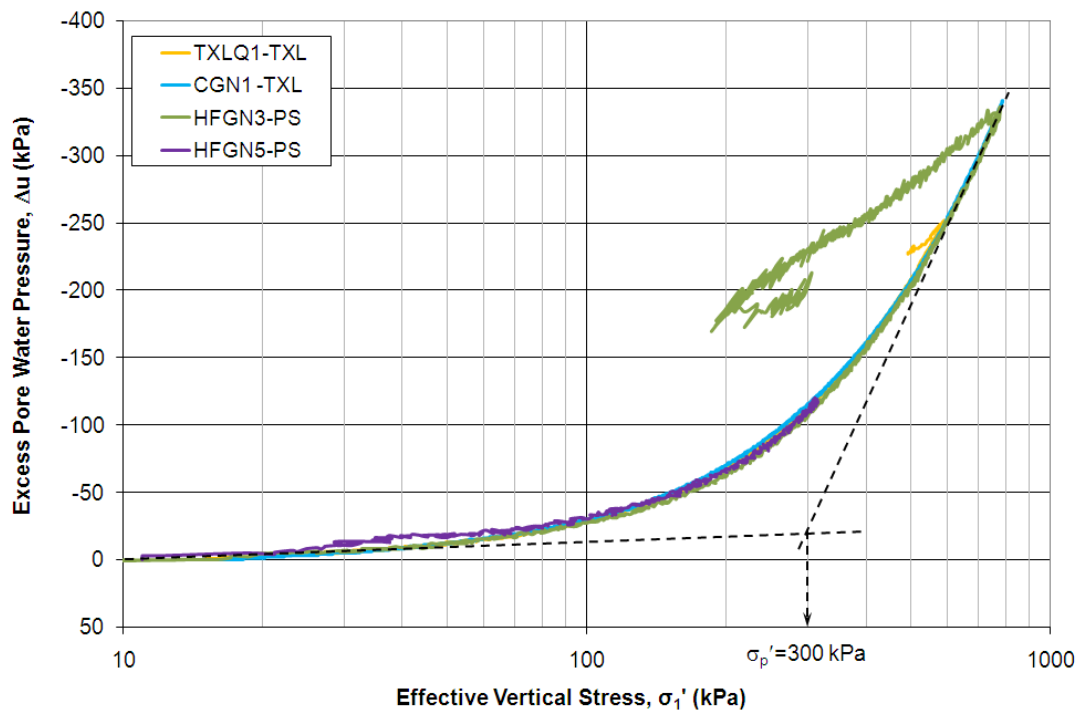


Figure 4.14 (a)

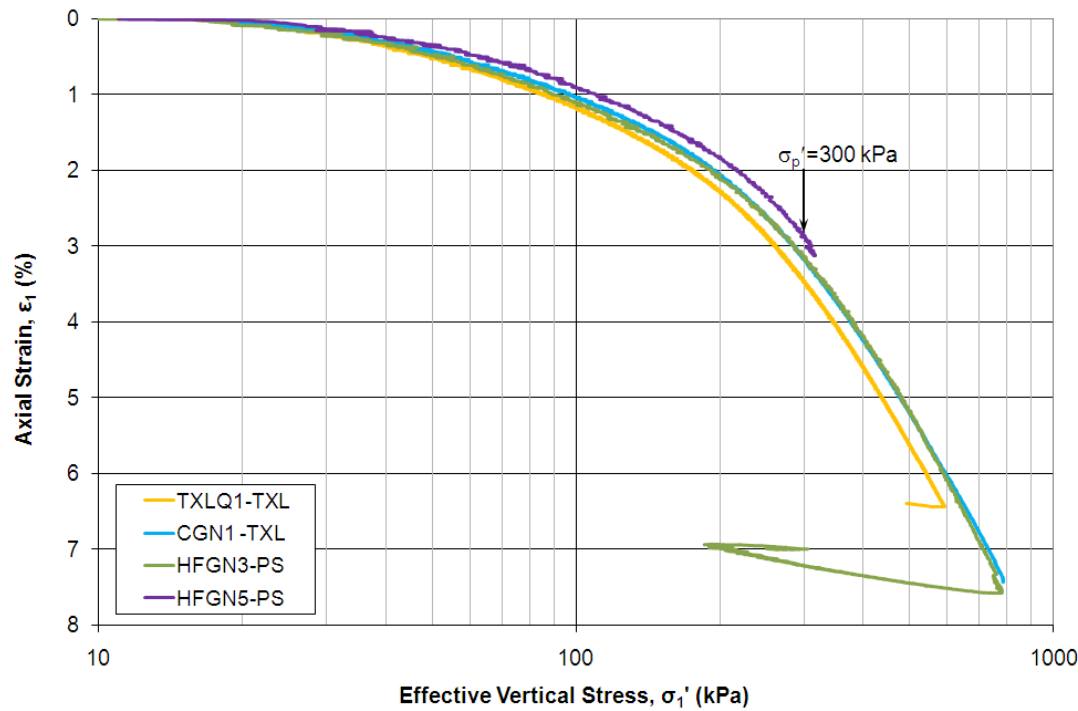


Figure 4.14(b)

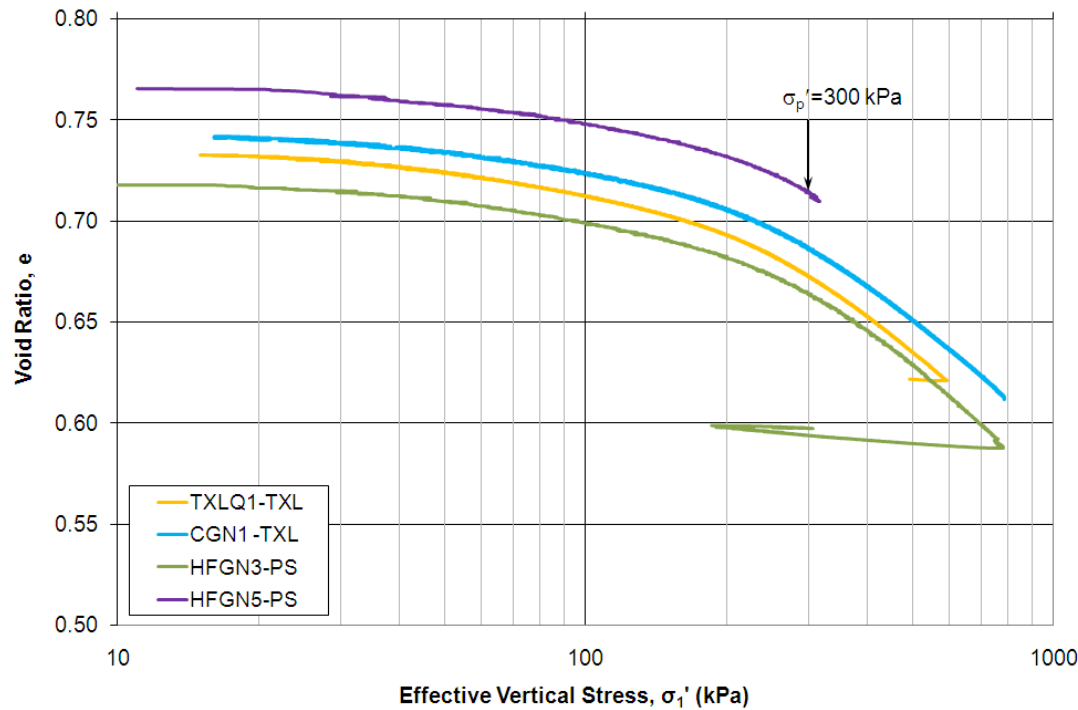


Figure 4.14 (c)

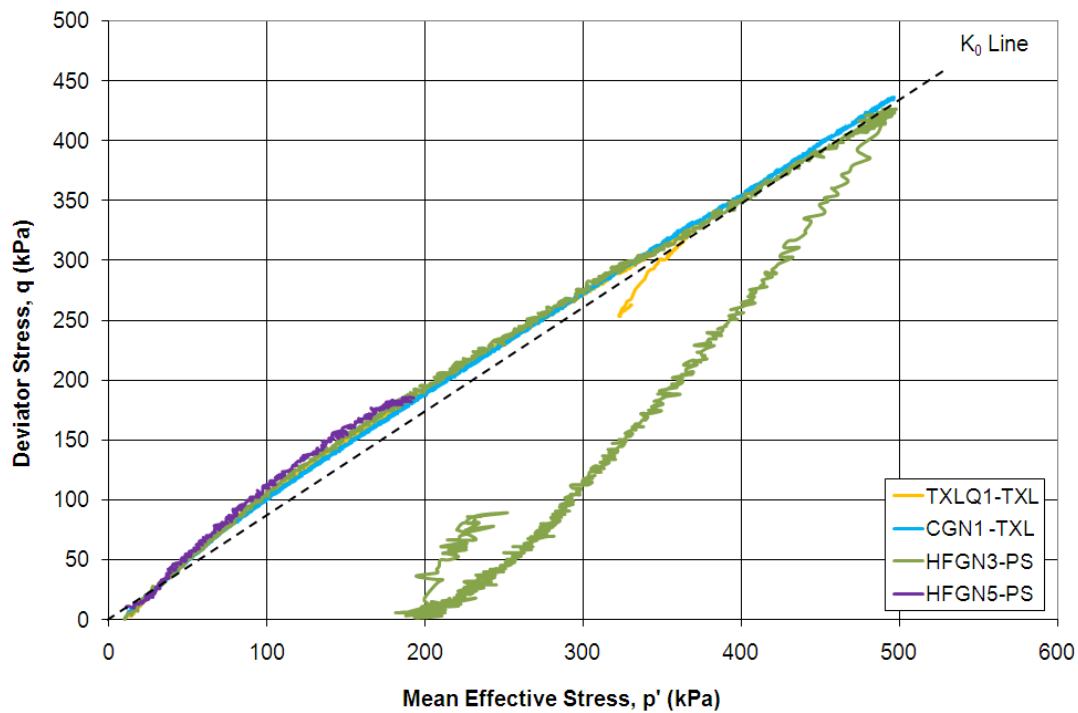


Figure 4.14(d)

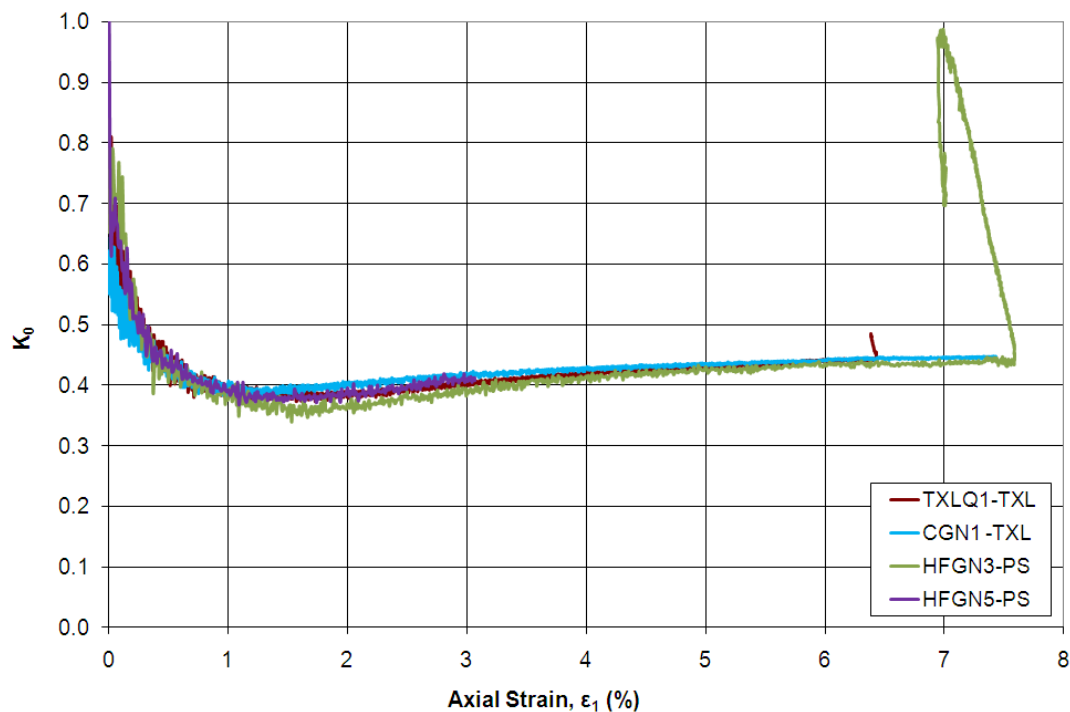


Figure 4.14(e)

Figure 4.14 Comparisons of K_0 Consolidation Tests under Triaxial Conditions and Plane-Strain Conditions: (a) Δu vs. $\log \sigma_1'$ Curves; (b) $\varepsilon_1\%$ vs. $\log \sigma_1'$ Curves; (c) e vs. $\log \sigma_1'$ Curves; (d) Effective Stress Paths; (e) K_0 vs. $\varepsilon_1\%$ Curves

4.4 K_0 Consolidation of Artificially Cemented Soil

4.4.1 Using Triaxial Cell

As mentioned in Chapter 3, the residual soil of Bukit Timah granite used in this study had a true cohesion that is small enough to be ignored. Therefore, an artificially cemented soil was tested to study the cementation effect on the shear strength and deformation behaviour. The artificially cemented soil was prepared by burning a compacted soil specimen, which made from kaolin-changi sand mixture, at 500°C. According to Grim (1953), firing of kaolin causes an endothermic reaction, in which could result in bonding between soil particles. The consolidation behaviour under triaxial conditions was presented as following. The details of the K_0 consolidation tests can be found in Table 4.10.

Table 4.10 Summary of K_0 Consolidation Tests Conducted on Artificially Cemented Soil under Triaxial Conditions

Test No.		AGN3	AGN4	AGN11
Diameter (mm)		50	50	50
Height (mm)		100	100	100
Weight (g) (after burn)		271.91	274.68	276.27
Initial Void Ratio, e_0		0.95	0.93	0.89
Type of Test		CK ₀ UC	CK ₀ UC	CSD
Pressure for Saturation (kPa)	Cell	860	860	860
	Back	850	850	850
Consolidation Rate (mm/min)		0.005	0.005	0.005
Maximum Consolidation Pressure (kPa)	σ_1'	1150.06	1012.42	1250.77
	q	680.06	618.42	748.77
	p'	696.35	600.137	751.59

As presented in Figure 4.15(a), a sharp change was observed from e versus $\log \sigma_1'$ and Δu versus $\log \sigma_1'$ curves. A pre-consolidation stress of 600 kPa was determined. Since the artificially cemented soil specimens were prepared individually, the void

ratio of soil specimen may vary in a small range. The variation was also observed for the compacted residual soil.

Similar to the consolidation behaviour of the intact residual soil, a typical destruction of soil structure was observed for artificially cemented soil. As shown in Figure 4.15(b), an elastic response at beginning to consolidation was observed clearly from the enlarged plot. A significant change in the slope of the stress-strain curves was also noted. The corresponding tangent stiffness was calculated and plotted against the axial strain in Figure 4.15(c). Two yielding points were identified at 0.7% and 2% of axial strain. Based on the location of yielding points, the effective stress path was divided into three zones of behaviour, as shown in Figure 4.15(e). The artificially cemented soil behaved similarly as the intact residual soil in each zone.

The cementation index (I_c) was calculated for the artificially cemented soil from Figure 4.15(e). A value of 1.16 was obtained, which was much higher than the value obtained for the intact residual soil (0.48 for vertically cut specimen under triaxial conditions). This led to much higher yielding stresses of 360 kPa and 680 kPa, as shown in Figure 4.14 (c).

The K_0 value during consolidation tests are plotted against the axial strain in Figure 4.15(g). It reached the lowest point (0.13-0.15) first, then followed by a notable amount of recovery (0.25-0.27). An ultimate value of 0.4 was approached towards the end of K_0 consolidation. The increment of K_0 value after the trough was increased with I_c value. It is much higher for artificially cemented soil than the one for the intact residual soil (0.17-0.25). It is also observed that the first yielding occurs at the lowest K_0 . It may be also concluded that the large amount of recovery in K_0 was an indication of destruction process of cemented of soil.

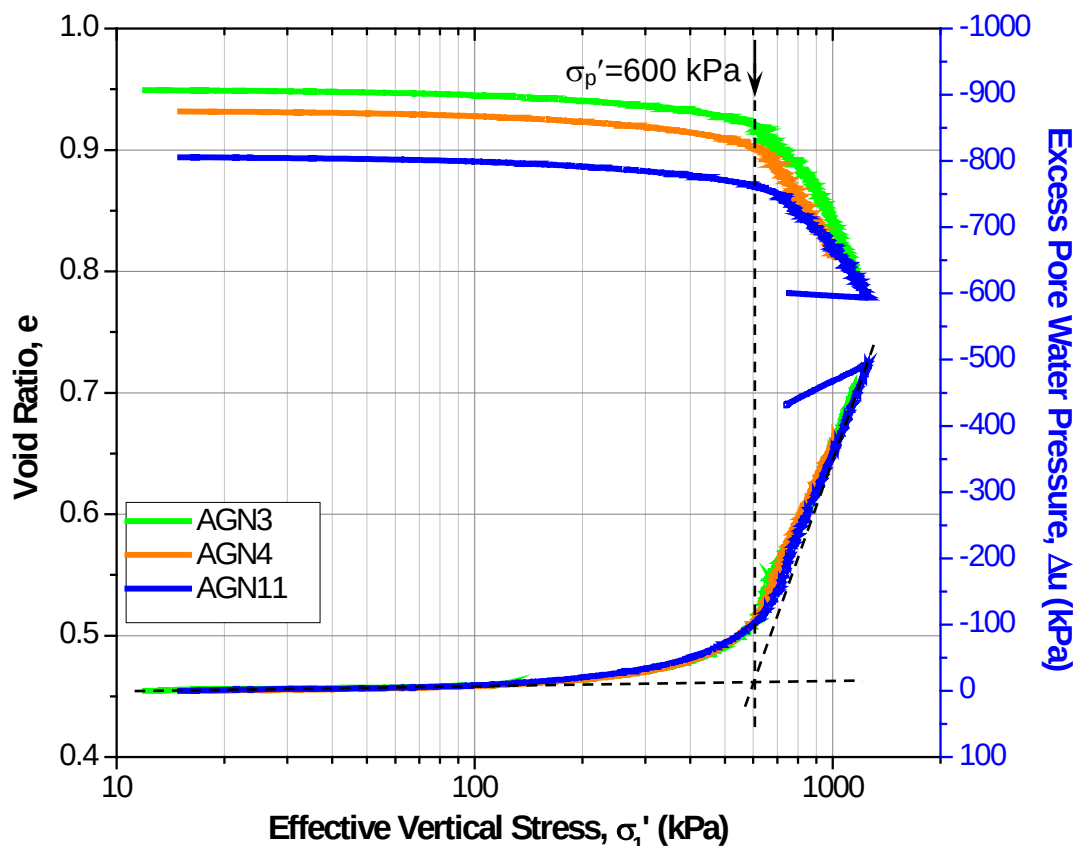


Figure 4.15(a)

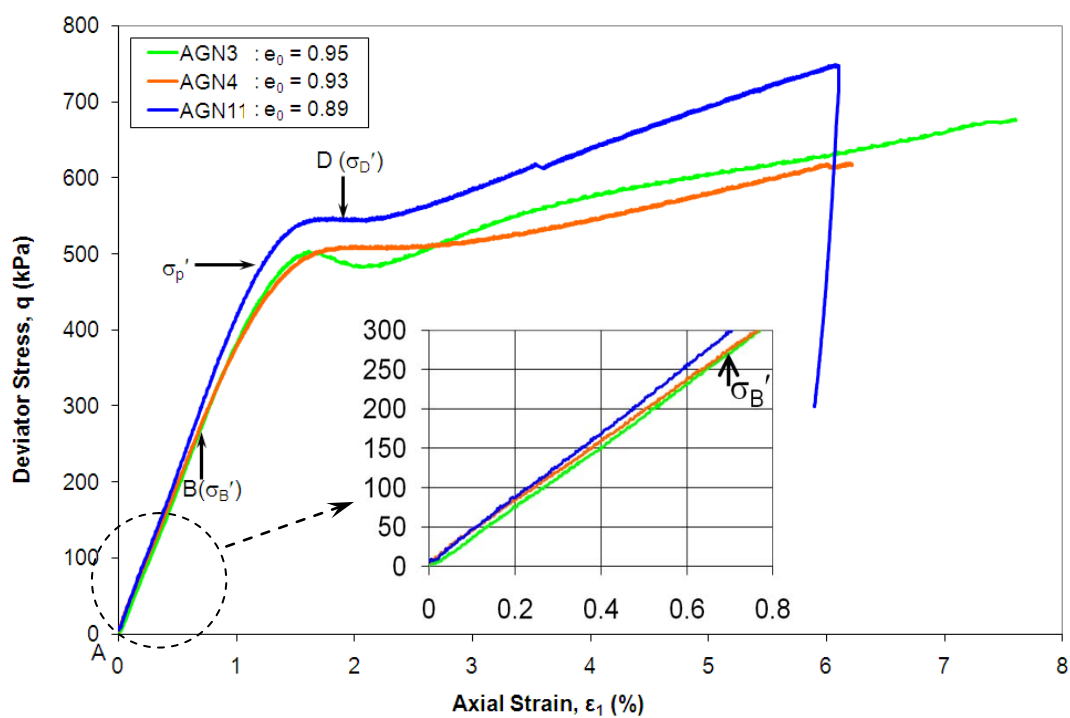
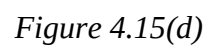
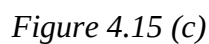


Figure 4.15 (b)



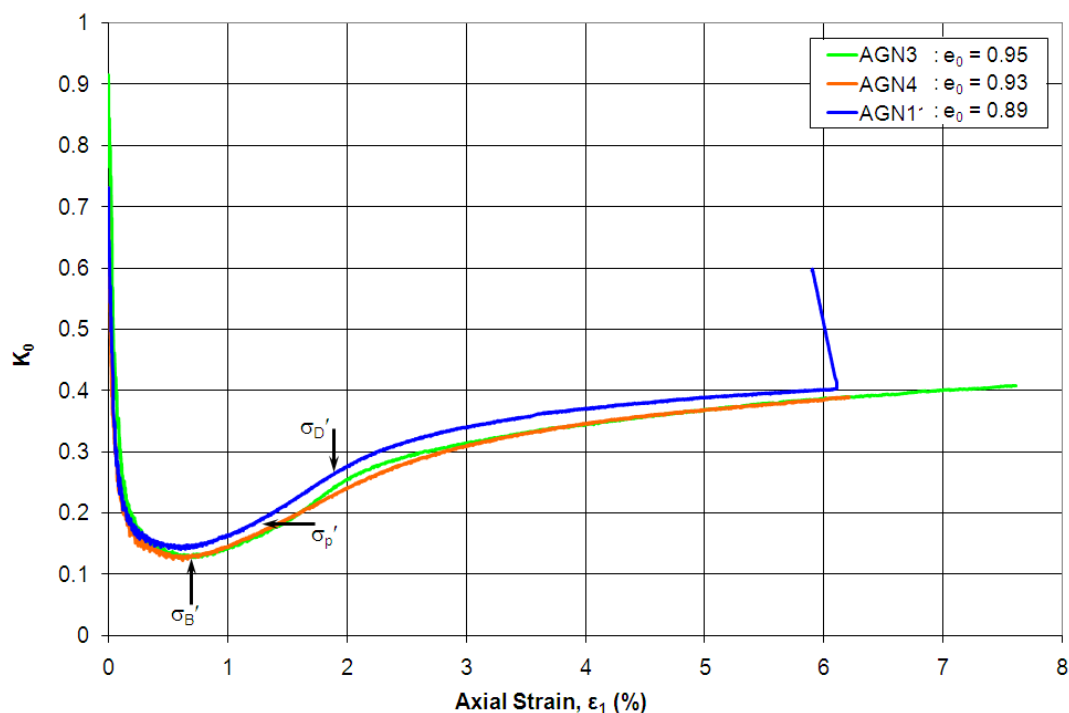


Figure 4.15(e)

Figure 4.15 K_0 Consolidation on Artificially Cemented Soil under Triaxial Condition: (a) e and Δu vs. $\log \sigma_1'$ Curves; (b) Stress-Strain Curves; (c) Et vs. $\log(\varepsilon_1\%)$ Curves; (d) Effective Stress Paths; (e) K_0 vs. $\varepsilon_1\%$ Curves

A comparison between estimated K_0 and measured K_0 is also carried out for the artificially cemented soil, as presented in Table 4.11. The prediction is consistent with experimental measurement. Similar observations have been made for both intact and compacted residual soils.

Table 4.11 Comparison of K_0 Values for Artificially Cemented Soil

Testing Condition	$K_0 = 1 - \sin \phi_{cs}'$	Measurement	Difference
Triaxial	0.408	0.4	2.01 %

4.5 Conclusions

The one-dimensional consolidation behaviour of residual soils and artificially cemented soil were presented in this chapter. The K_0 consolidation tests were conducted by using both triaxial cell and plane-strain apparatuses. The results presented in Chapter 4 can be summarized as follows:

1. The pre-consolidation stress (σ_p'), is normally determined from the laboratory one-dimensional consolidation (oedometer) tests. However, as pointed out by Chu et al. (2005), oedometer tests are not suitable for stiff to hard soils, such as residual soil, because of the large bedding errors and wall frictions. Chu et al. (2005) suggested conducting K_0 consolidation test using triaxial cell to overcome the difficulties encountered in oedometer test. To adopt a more consistent method to determine σ_p' , a method similar to the one proposed by DeGroot et al. (2007) was adopted in this study. As pore water pressure (u) is generated only as a response of plastic yield, the value of σ_p' could be obtained from the Δu vs. $\log \sigma_1'$ curves. By comparing the σ_p' values among the intact residual soil, compacted residual soil and artificially cemented soil, it is verified that the σ_p' is significantly affected by soil cementation (Gutiérrez et al., 2008).
2. For intact residual soil, two yielding points σ_B' , σ_D' were determined based on the variation of tangent stiffness with axial strain. Consequently, three zones of behaviour were identified on the effective stress paths as proposed by Malandraki & Toll (1996). The intact residual soil behaved elastically in Zone 1. The soil bonding was still intact when stress level is below σ_B' . The de-structure process carried out in Zone 2. Essentially bonding was broke down under σ_p' , which resulted in large plastic deformation. The soil bonding was completely destroyed at σ_D' . In Zone 3, the bonded material behaviour similarly to the uncemented soil. The effective stress path migrated to the K_0 line and maintained there until a pre-determined

consolidation stress was achieved. Similar behaviour was also observed for the artificially cemented soil.

3. As concluded by many researchers (Maccarini, 1987; Vaughan, 1988; Leroueil & Vaughan, 1990; Toll & Malandraki, 1993; Malandraki & Toll, 2000; Leroueil & Hight, 2003; Toll et al., 2006; Lo & Hinchberger, 2007), the presence of soil structure will lead the effective stress path to move along the yielding surface before yield occurs, which was above the K_0 line. The stress path migrates back to the K_0 line once soil structure was destroyed. Based on this observation, a cementation index (I_c) was proposed in this study to quantify the cementation of soil by measuring the distance between the effective stress path and the K_0 line. It follows:

$$I_c = \frac{|\overline{K_0} p'_m - q_m|}{\sqrt{\overline{K_0}^2 + 1}} / P_a \quad (4-1)$$

where p'_m and q_m are the coordinates of the point furthest away from K_0 line; $\overline{K_0}$ is the slope of the K_0 line, which can be expressed as $\overline{K_0} p' - q = 0$, and P_a is the Atmosphere pressure (101.325 kPa) used to eliminate the effect of units.

4. By evaluating the I_c value for the intact residual soil and artificially cemented soil, it is observed that the yielding stress, σ'_B and σ'_D increased with I_c for bonded materials. It is also noted that bonding of soils can be also reflected by the variation of K_0 value. For the intact residual soil and the artificially cemented soil, the K_0 value attained a lowest value first before it increases gradually, which could be a reflection of the de-structuring process of bonded materials. The first yield (σ'_B) was found to occur at the lowest K_0 value. A constant K_0 value was achieved at the end of a test, which may be an indication of completely breaking down of soil structures. It is also noted that the picking up of K_0 value after the trough was increased with the I_c value. In good agreement, for compacted residual soil with very low I_c , the

K_0 value remained constant after reaching the lowest value. Little recovery in K_0 was observed.

5. A comparison of K_0 values for NC soil between measured and predicted using $K_0 = 1 - \sin\phi_{cs}'$ was made. Good agreements are achieved for the intact, compacted residual soils, and artificially cemented soil.

CHAPTER 5

UNDRAINED BEHAVIOUR OF THE INTACT RESIDUAL SOIL

5.1 Introduction

To study the general strength and stress-strain behaviour of the intact residual soil of Bukit Timah granite, several series of K_0 consolidated undrained triaxial compression (CK_0UC), extension (CK_0UE) and direct simple shear (CK_0UDSS) tests, and plane-strain (CK_0UPS) tests were carried out. The anisotropy behaviour of the intact residual soil was also investigated by testing both horizontally cut and vertically cut specimens. The results obtained from these tests are presented in this chapter. Major findings from the tests are highlighted.

5.2 K_0 Consolidated Undrained Triaxial Compression (CK_0UC) Tests

5.2.1 Vertically Cut Specimens

The CK_0UC tests conducted on vertically cut intact residual soil specimens are summarized in Table 5.1. In all the tests, the specimens were K_0 consolidated before being sheared undrained. The method adopted for K_0 consolidation was described in Chapter 3. To achieve a normally consolidated (NC) state, the specimen was consolidated to a certain value of effective vertical stress, σ_1' , which was far away from the estimated pre-consolidation pressure (e.g. 600kPa for specimen GN10 and 900kPa for specimen GN11). To achieve a certain overconsolidation ratio (OCR), such as $OCR = 2$ in test GN3, the specimen was consolidated to an effective vertical

Table 5.1 Summary of CK₀UC Tests on Vertically Cut Intact Residual Soil Specimens

Test No.		GN3	GN5	GN6	GN7	GN8	GN9	GN10	GN11
Pressure for Saturation (kPa)	Cell	860	1010	810	810	860	860	860	860
	Back	850	1000	800	800	850	850	850	850
OCR		2	4	1.54	1	2.2	1	1	1
Type of Test		CK ₀ UC	CK ₀ UC	CK ₀ UC	CK ₀ UC	CSD	CK ₀ UC	CK ₀ UC	CK ₀ UC
Shearing Rate (mm/min)		0.01	0.005	0.005	0.007	0.005	0.005	0.005	0.005
Weight (g)	Pre-Test	1035.76	1039.2	1078.96	1056.68	1037.63	351.86	352.95	357
	Post-Test	1024.67	1009.93	-	1036.84	1090.24	343.86	349	344.85
Water Content (Pre-Test) (%)	Average	32.51	30.18	-	31.87	31.41	30.22	29.91	31.56
Water Content (Post-Test) (%)	Average	30.41	28.20	-	30.16	38.16	28.55	29.35	28.19
	Top	30.58	27.91	-	29.46	36.32	28.33	29.63	28.8
	Middle	30.42	28.3	-	30.13	39.37	28.18	28.77	27.1
	Bottom	30.25	28.41	-	30.25	37.87	29.15	29.65	28.67
σ'_{10} (kPa)		194.35	195.27	195.19	453.52	288.98	731.56	599.79	901.07
q_0 (kPa)		74.35	0	132.19	270.02	74.48	364.56	321.79	448.07
p'_0 (kPa)		144.78	200.76	107.06	275.01	239.33	488.52	385.26	602.36
q_{peak} (kPa)		245.04	363.51	156.64	287.36	75	410.08	358.43	504.53
ε_1 at q_{peak} (%)		2.42	3.33	12.58	1.37	-	1.01	1.32	1.12
$(q/p')_{\text{peak}}$		1.41	1.34	1.54	1.35	2.19	1.35	1.34	1.31
$R_{\text{max}} = (\sigma'_1/\sigma'_3)_{\text{max}}$		3.67	3.43	4.15	3.45	9.11	3.44	3.41	3.33
ε_1 at $(q/p')_{\text{peak}}$ (%)		5.9	5.25	6.53	6.11	-	8.59	5.46	6.71
Secant Friction Angle, φ' (°)		34.87	33.27	37.71	33.41	53.34	33.34	33.13	32.55

stress (σ_1') of 390kPa, which was far beyond the pre-consolidation stress ($\sigma_p' = 300\text{kPa}$), and then unloaded to 195kPa (in-situ effective vertical stress). The same procedure was adopted for the specimen with OCR value of 4 (GN5). The specimen was consolidated to 780kPa and then unloaded to 195kPa. The initial effective stress at shearing stage ($\sigma_1'{}_0$) was listed in Table 5.1. It should be mentioned that, specimen GN6 was consolidated directly to the in-situ effective vertical stress of 195kPa, which was lower than σ_p' (300kPa) of the vertically cut intact residual soil. The OCR value of specimen GN6 was calculated as $OCR = \sigma_p' / \sigma_{10}' = 300/195 = 1.54$.

The failure envelope of the vertically cut intact residual soil is shown in Figure 5.1(a). It is determined from seven CK_0 UC tests and one constant-shear-drained (CSD) test (GN8), which will be discussed in more details in Chapter 7. The stress paths followed by K_0 consolidation are also shown as thin dash lines in Figure 5.1(a) for reference. The failure envelope was determined as an envelope to all the effective stress paths, and passes through the points of peak effective stress ratio, $(q/p')_{\text{peak}}$. It is straight within the tested stress range (BC in Figure 5.1(a)). As presented in Chapter 3, the intact residual soil disintegrated immediately after soaking into water. Therefore, it can assume that the true cohesion of the intact residual soil is small enough to be ignored. Thus, the failure envelope within the low stress range is assumed to pass through the origin, as shown by dotted curve AB in Figure 5.1(a).

The stress-strain behaviour of the vertically cut intact residual soil specimens determined from CK_0 UC tests is shown in Figure 5.1(b) for NC and OC specimens respectively. It was observed that the peak deviator stress (q_{peak}) was attained at a very small axial strain (1-3%) for most of the tests, and the peak indicated the failure of the specimens. A significant amount of strain softening was noted after the q_{peak} for NC clay. The q_{peak} increased with OCR for OC specimens and with the effective confining pressure for NC specimens. The peak effective stress ratio, $(q/p')_{\text{peak}}$ occurred right after the q_{peak} at about 5-6% of axial strain (as shown in Figure 5.1(c)). It should also point out that the q/p' measured in Test GN6 was

much higher than the rests. This may be because the specimen GN6 was consolidated to the lowest stress level, which was below the pre-consolidation stress. The soil bonds were not destroyed completely during consolidation as the others did. The existence of soil bonds may affect the variation of stress ratio significantly.

By comparing the variations of q/p' between NC and OC specimens in Figure 5.1(d), it is noted that the $(q/p')_{\text{peak}}$ decreased as OCR increased for OC clay. For NC clay, all the curves fell into a narrow band, indicating that $(q/p')_{\text{peak}}$ is not greatly affected by stress level for NC soil. .

Figure 5.1(e) presents the normalized stress-strain behaviour of vertically cut intact residual soil. The deviator stress is normalized against the initial effective vertical stress (σ'_{10}). For OC clay, the normalized peak deviator stress increased as OCR increased. For NC clay, a normalized behaviour was observed. The deviator stresses obtained from tests under different effective confining stresses tended to reach the same peak value. Therefore, the well-known c_u/σ'_{10} vs. OCR relationship shown below is applicable to residual soil.

$$\left(\frac{c_u}{\sigma'_{10}}\right)_{OC} = \left(\frac{c_u}{\sigma'_{10}}\right)_{NC} OCR^A \quad (5-1)$$

where $(c_u/\sigma'_{10})_{OC}$ and $(c_u/\sigma'_{10})_{NC}$ are the undrained shear strength ratio for OC and NC specimens respectively and A is a constant which varies from soil to soil. The c_u/σ'_{10} versus OCR relationship determined from CK₀UC tests on vertically cut specimens is plotted in Figure 5.2. As seen from the figure, $(c_u/\sigma'_{10})_{NC}$ had a average value of 0.294. The test results were fitted into the above relationship, a value of 0.874 was obtained for A . This relationship will be compared with the one determined from tests on horizontally cut intact residual soil specimens in section 5.2.3.

The variation of excess pore water pressure (Δu) during undrained shearing is shown in Figure 5.1(f). The excess pore water pressure increased drastically at beginning of shearing. For OC specimens, no significant drop was observed after

$(\Delta u)_{\text{peak}}$. The peak excess pore water pressure increased with OCR. For NC clay, the pore water pressure increased during undrained shearing, since the specimen was compressed as axial strain increased. The excess pore water pressure increased as effective confining pressure increased.

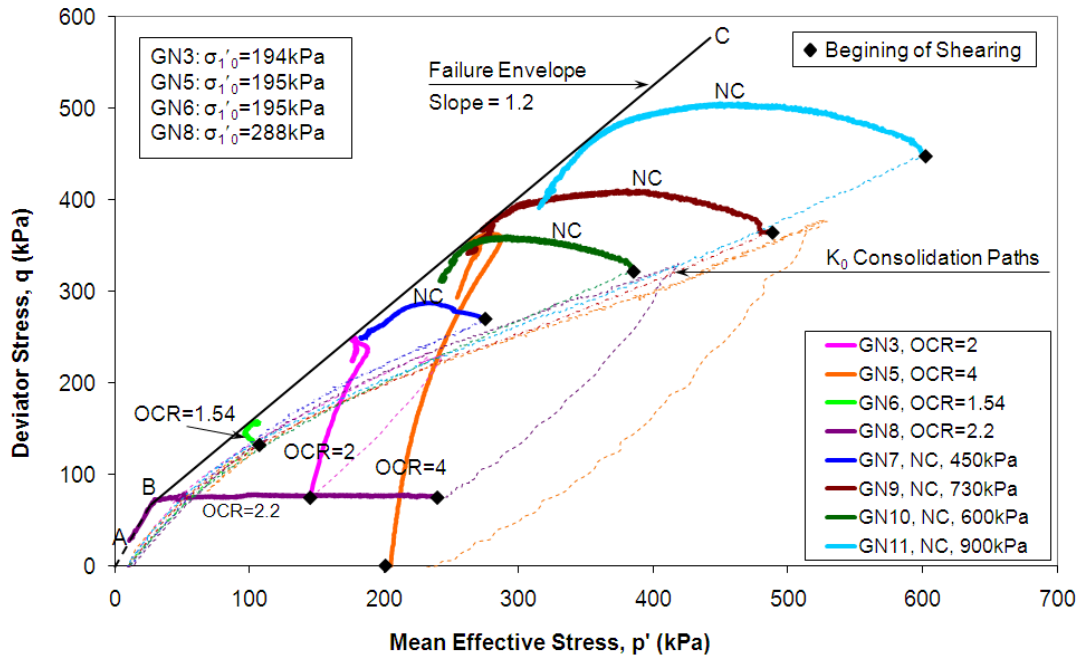


Figure 5.1(a)

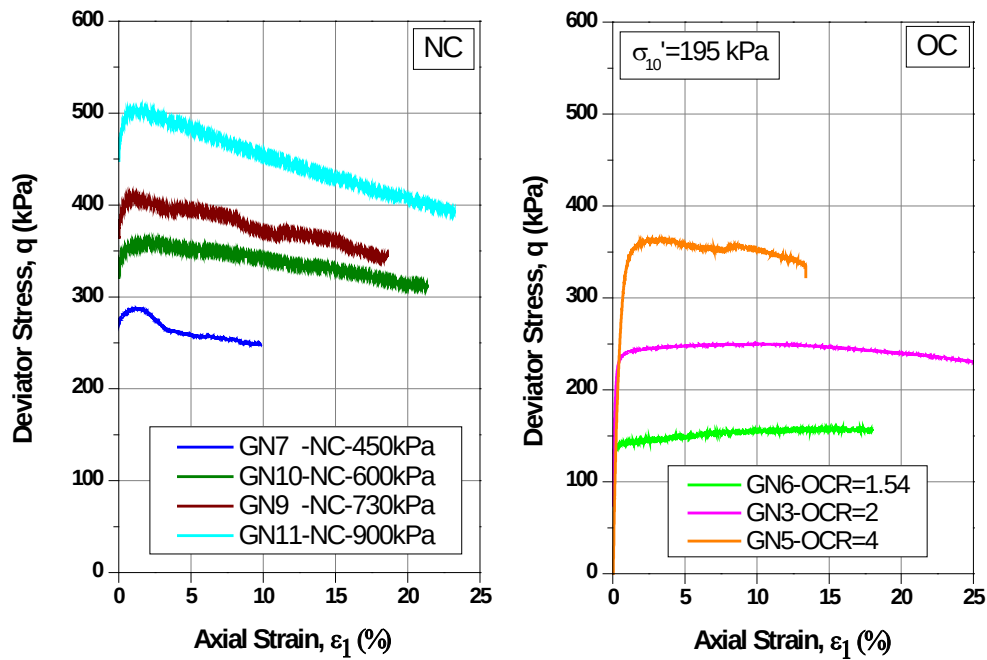


Figure 5.1 (b)

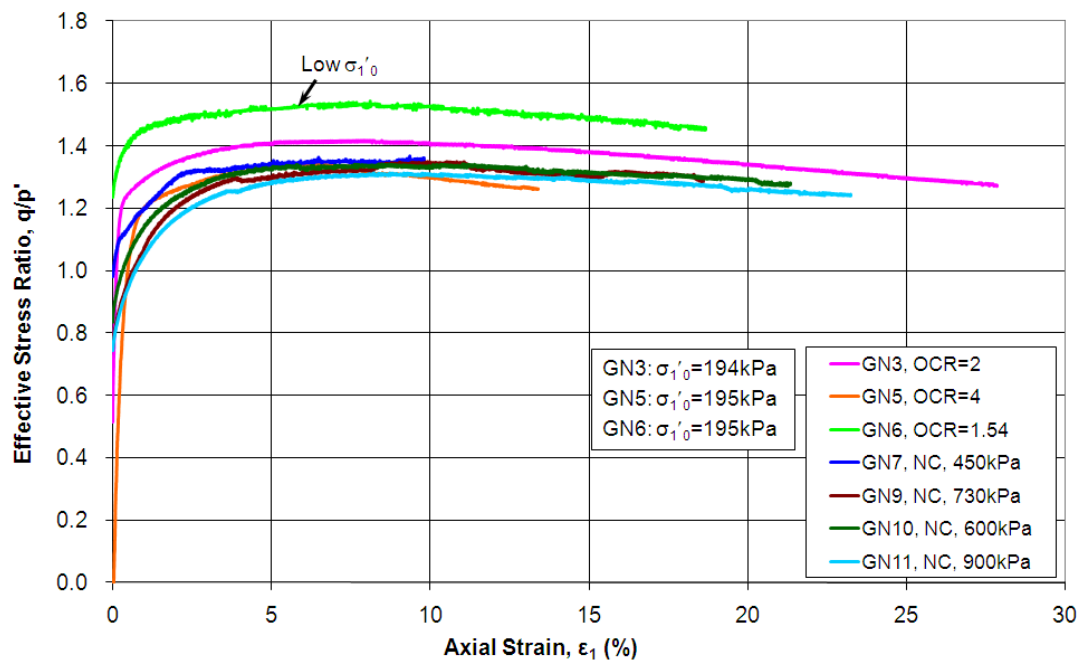


Figure 5.1 (c)

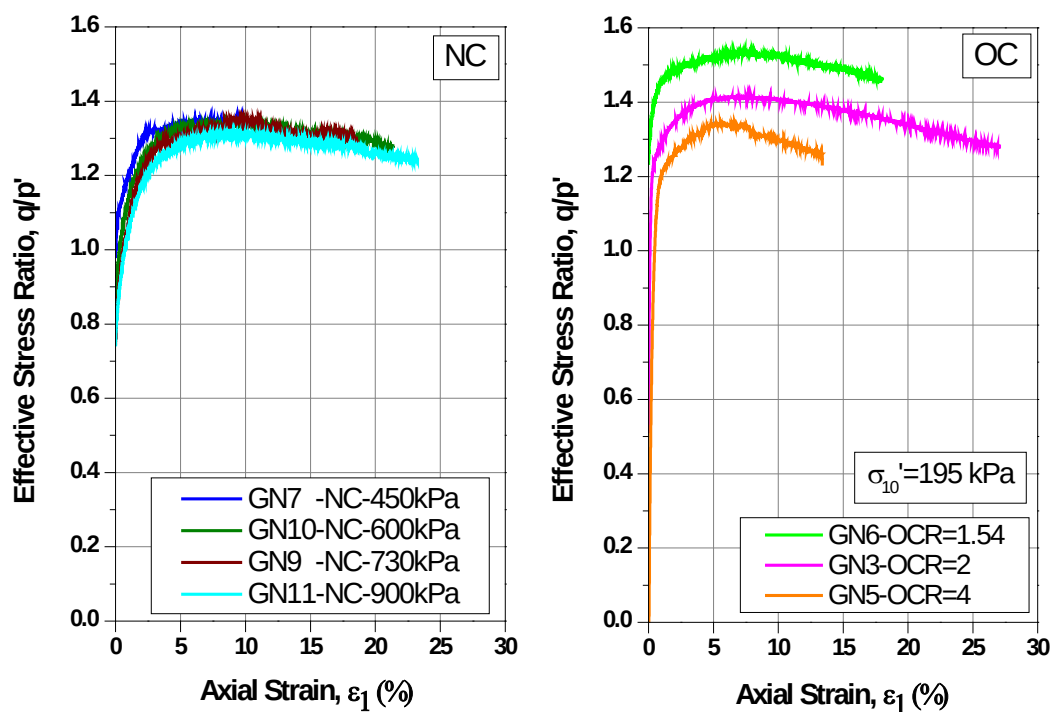


Figure 5.1(d)

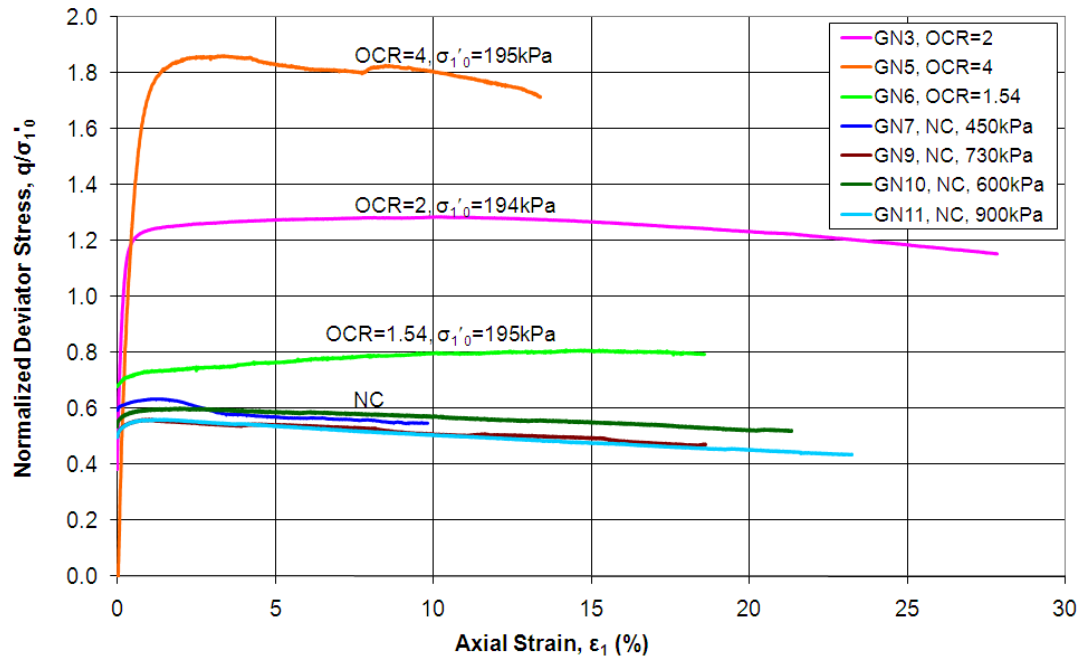


Figure 5.1 (e)

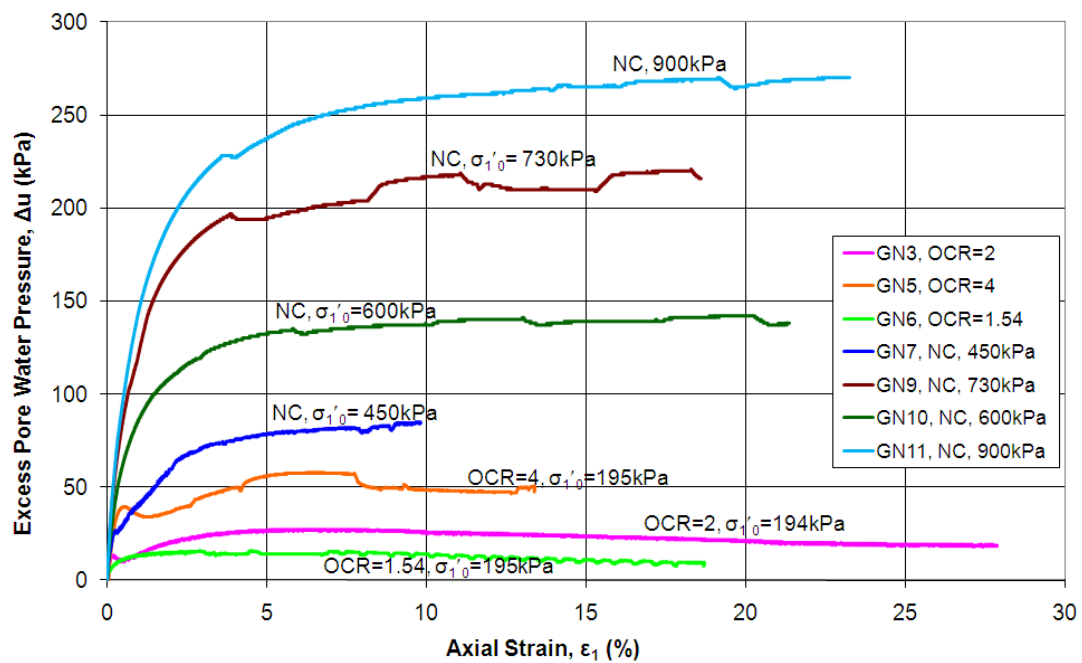


Figure 5.1(f)

Figure 5.1 Undrained Behaviour of Vertically Cut Intact Residual Soil from CK_0UC Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) q/p' vs Axial Strain Curves; (d) Comparison of (q/p') between NC and OC Specimens; (e) Normalized Stress-Strain Curves; (f) Excess Pore Water Pressure vs. Axial Strain Curves

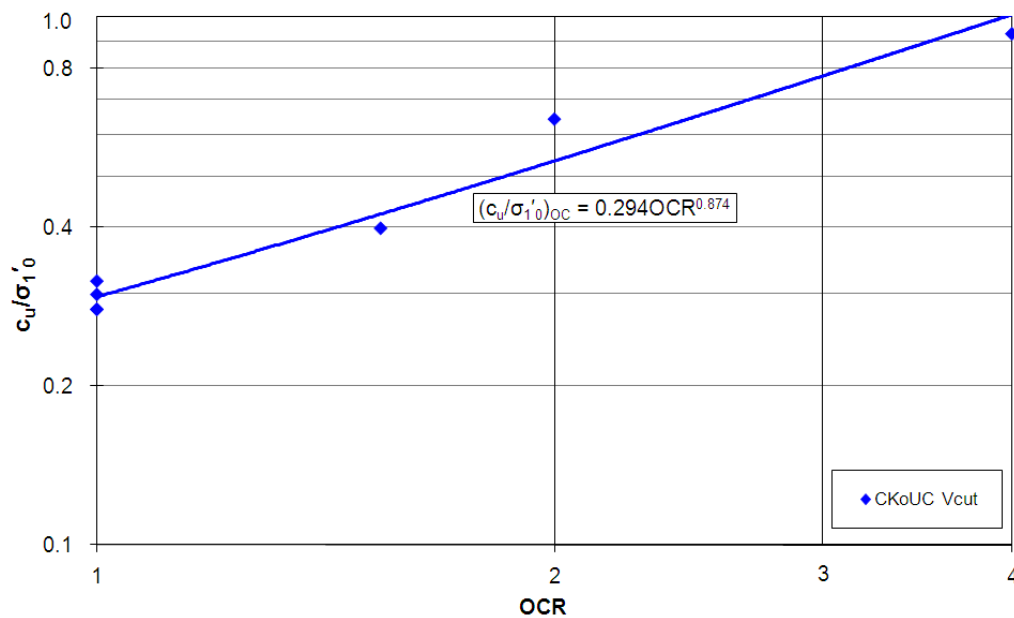


Figure 5.2 c_u/σ'_{10} vs. OCR Relationship Obtained from CK₀UC Tests on Vertically Cut Intact Residual Soil Specimens

Due to the inconsistency of effective confining stress, the stress paths resulting from CK₀UC tests on vertically cut intact residual soil specimens need also to be normalized. A coupled stress (Yao & Zhu, 2009) which takes the effects of both deviator stress (q) and mean effective stress (p') into consideration is chosen as the normalized parameter:

$$T = M^2 p' + q^2 / p' \quad (5-2)$$

where M is the slope of the critical state line. The critical state line determined from tests on NC specimen is shown by dotted line in Figure 5.3(a), with a slope of 1.34, i.e. $M=1.34$. The normalized stress paths determined from tests on NC specimens were consistent. It is indicative that the normalized stress path is independent of the stress level of NC specimen.

The variation of Skempton's pore pressure parameter A is presented in Figure 5.4(a). For CK₀UC tests conducted on fully saturated soil specimens under constant cell pressure (Skempton, 1954), it can be calculated as following:

$$A = \Delta u / \Delta \sigma_1 = \Delta u / \Delta q \quad (5-3)$$

where Δu is the change in pore water pressure, $\Delta \sigma_1$ is the change in vertical stress (or $\Delta \sigma_v$), and Δq is the change in the deviator stress. It was observed that, for NC clay, the A value kept increasing after the peak deviator stress; this was due to the significant amount of straining softening of NC specimens. However, for OC clay, the A value decreased as shearing continued, and reaches a constant after the deviator stress reaches the peak. This was caused by strain hardening and constant excess pore water pressure towards the end of the tests.

The Skempton's pore pressure parameter at failure (A_f) is plotted against OCR in Figure 5.4(b). According to Das (1998), the A_f was measured at the peak deviator stress (q_{peak}) during undrained shearing. As seen from Figure 5.4(b), the A_f value decreased drastically with OCR. It tended to reach a constant value when OCR was beyond two. In Figure 5.4(b), it also presents the relationships determined from previous studies on Oslo clay, Oslo and London clay and Weald clay (Simons, 1960). It should be pointed out that the relationships established by Simons (1960) were based on isotropically consolidated undrained (CIU) tests results. The variation of A_f with OCR for the intact residual soil was consistent with Simons' observation (Simons, 1960). However, the CK_0UC tests gave a much higher A_f value at NC state than Simons' data. This may be due to the difference in consolidation methods. In this study, the intact residual soil was K_0 consolidated, instead of isotropically consolidation. The high stress level resulted from K_0 consolidation led to a higher value of excess pore water pressure, consequently, a higher A_f value.

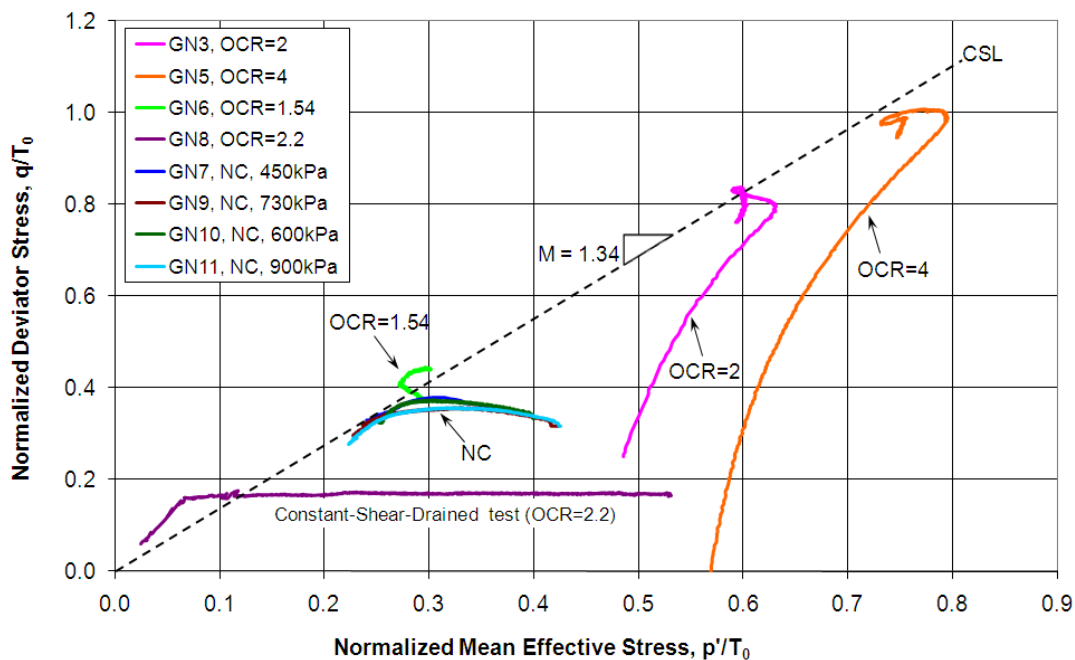


Figure 5.3 Normalized Stress Paths of Vertically Cut Intact Residual Soil from CK_0UC Tests

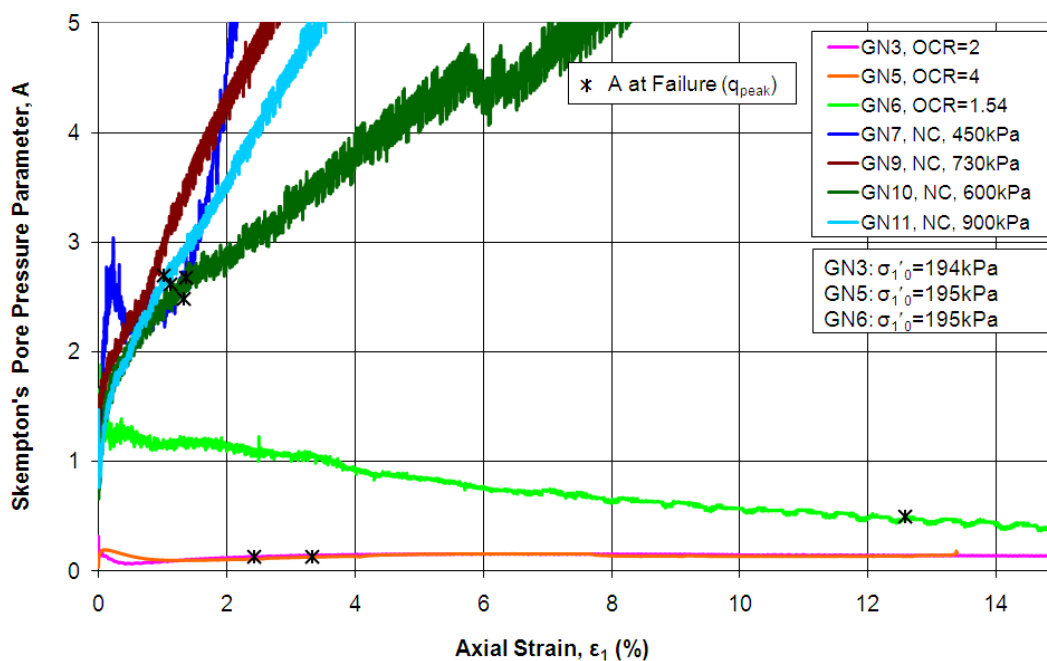


Figure 5.4 (a)

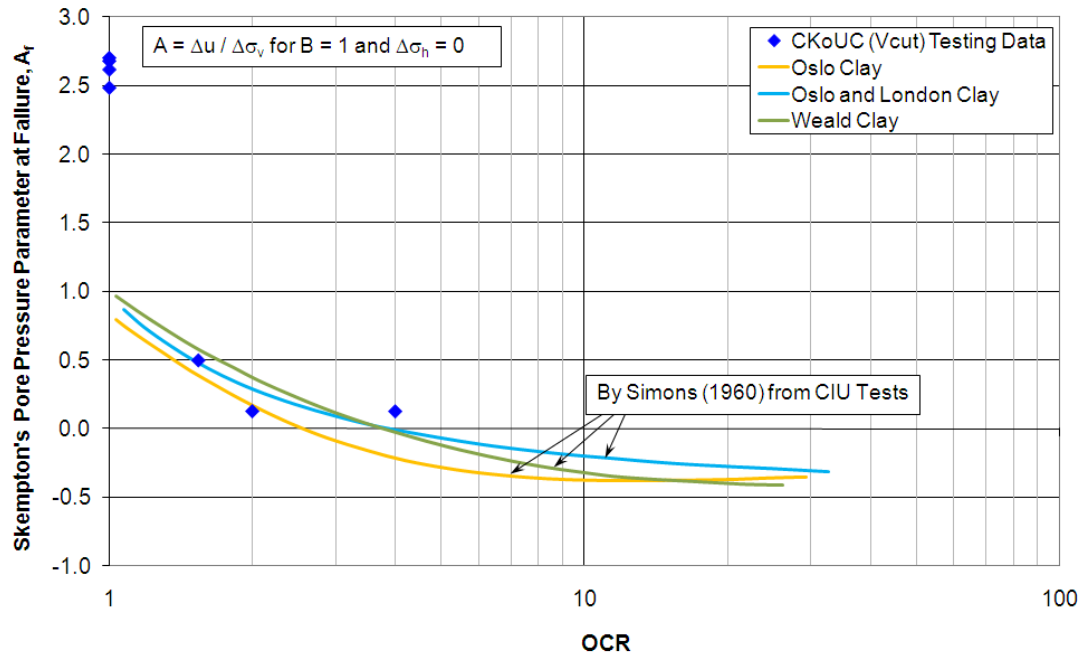


Figure 5.4 (b)

Figure 5.4 Variations of Skempton's Pore Pressure Parameter, A in CK_0UC Tests:

(a) A vs. Axial Strain Curves; (b) A_f vs. OCR Curves

5.2.2 Horizontally Cut Specimens

The CK_0UC tests conducted on horizontally cut intact residual soil specimens are summarized in Table 5.2. Figure 5.5 present the undrained behaviour determined from those tests.

The failure envelope of horizontally cut intact residual soil is determined from three CK_0UC tests. The specimen for each test was anisotropically (K_0) consolidated to OCR of 1, 2 and 4. The OCR value was imposed by adopting the same method as described in Section 5.2.1. The K_0 consolidation paths were shown by dotted curves in Figure 5.5(a). The failure envelope within the tested stress range is straight and passing through the origin with a slope of 1.4, as shown in Figure 5.5(a).

The stress-strain curves are shown in Figure 5.5(b). Similar to the results of vertically cut intact residual soil specimens, the deviator stress reached the peak at

small strain (less than 1%). For OC specimen, the peak deviator stress increased with OCR value. The variation of effective stress ratio (q/p') is presented in Figure 5.5(c). It is observed that the peak stress ratio of OC specimens increased as OCR decreased.

Table 5.2 Summary of CK_0UC Tests on Horizontally Cut Intact Residual Soil Specimens

Test No.		GN3H	GN5H	GN7H
Pressure for Saturation (kPa)	cell	860	1010	1010
	back	850	1000	1000
OCR		2	4	1
Shearing Rate (mm/min)		0.007	0.007	0.007
Weight (g)	Pre-Test	1018.14	1029.71	1035.06
	Post-Test	1022.38	1015.67	1032.33
Water Content (Pre-Test) (%)	Average	30.83	31.64	31.39
Water Content (Post-Test) (%)	Average	30.58	29.42	31.13
	Top	30.00	30.75	30.68
	Middle	30.66	28.75	31.17
	Bottom	31.08	28.78	31.25
	Shear Band	-	30.75	31.44
$\sigma_1'{}_0$ (kPa)		218.48	194.58	463.91
q_0 (kPa)		74.48	21.58	261.91
p'_0 (kPa)		168.83	180.09	289.3
q_{peak} (kPa)		258.27	361.16	288.19
ε_1 at q_{peak} (%)		7.07	2.37	0.46
$(q/p')_{peak}$		1.46	1.37	1.35
$R_{max} = (\sigma_1'/\sigma_3')_{max}$		3.86	3.51	3.46
ε_1 at $(q/p')_{peak}$ (%)		4.61	5.16	3.69
Secant Friction Angle, ϕ' (°)		36.05	33.82	33.47

To study the $c_u/\sigma_1'{}_0$ vs. OCR relationship of the intact residual soil in the horizontal direction, the stress-strain curved is normalized against the initial effective vertical stress ($\sigma_1'{}_0$) in Figure 5.5(d). It is clearly observed that the normalized deviator stress increased with OCR value. Figure 5.6 presents the corresponding $c_u/\sigma_1'{}_0$ vs.

OCR curve determined for horizontally cut intact residual soil. The test results fitted into the relationship very well. The $(c_u/\sigma'_{10})_{NC}$ was 0.310, and the power, A was 0.826. The generation of excess pore water pressure during CK_0UC tests on horizontally cut intact residual soil is shown in Figure 5.5(e). It is observed that the excess pore water pressure was affected by both OCR and consolidation stress.

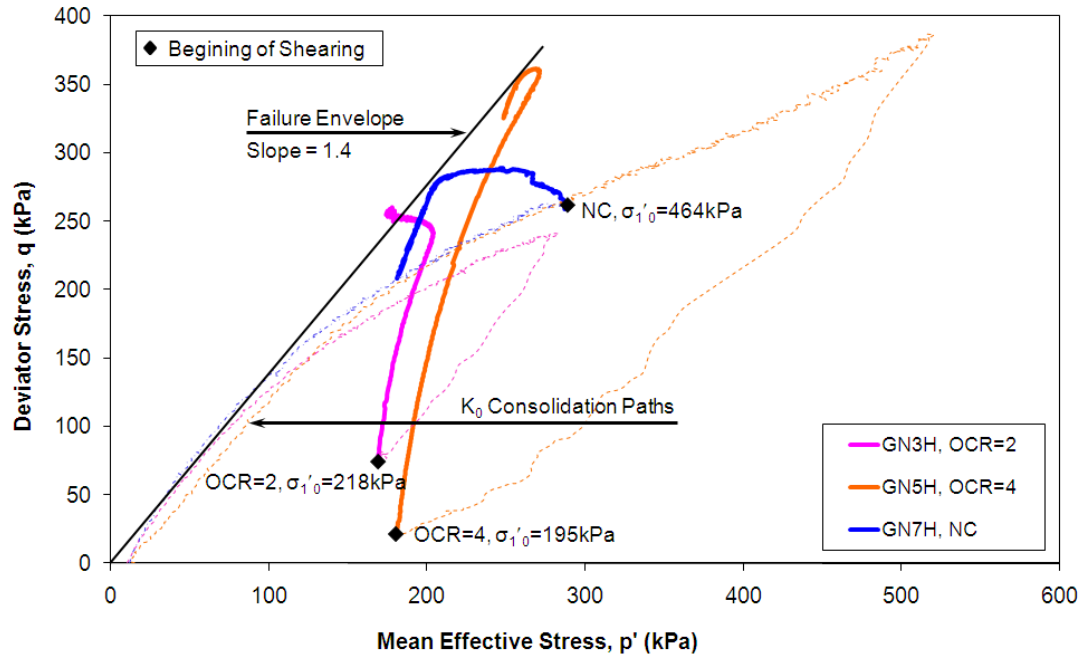


Figure 5.5 (a)

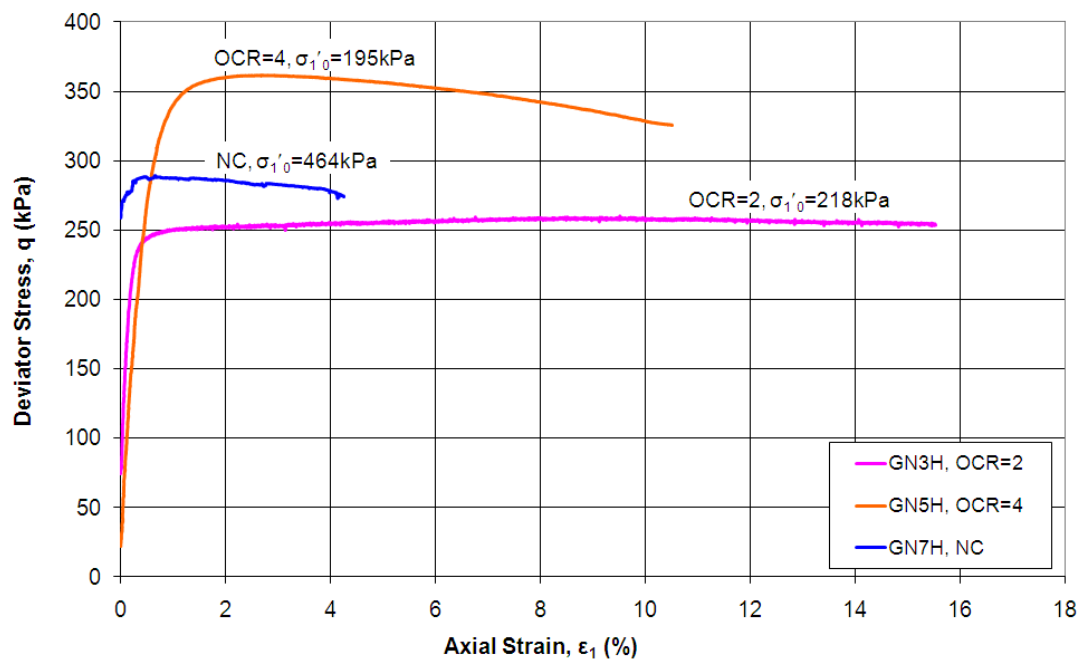


Figure 5.5 (b)

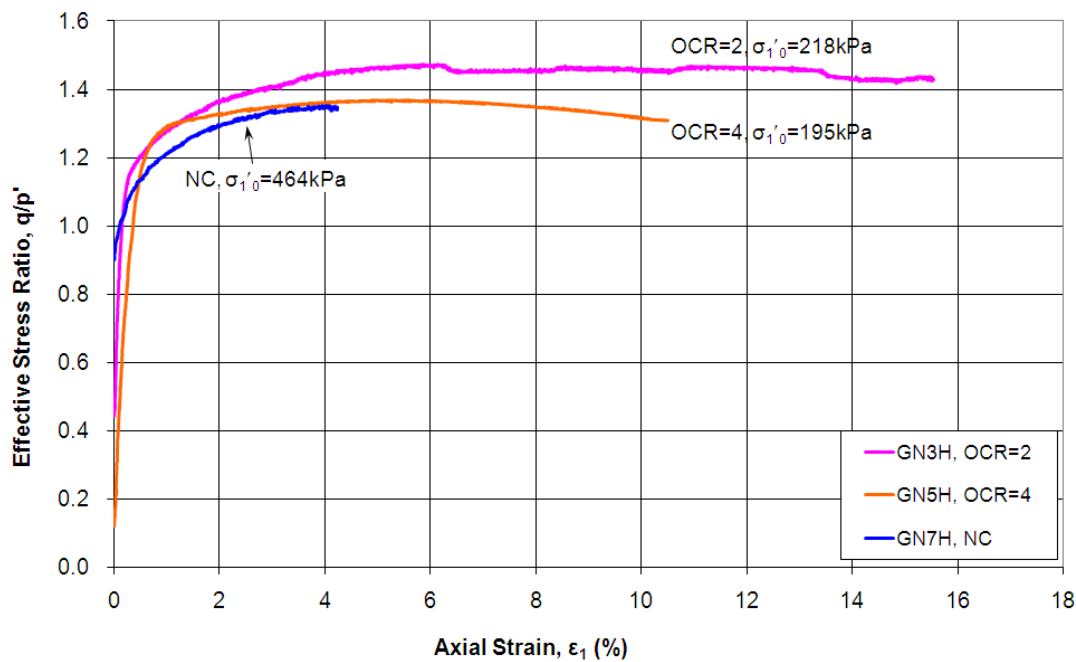


Figure 5.5(c)

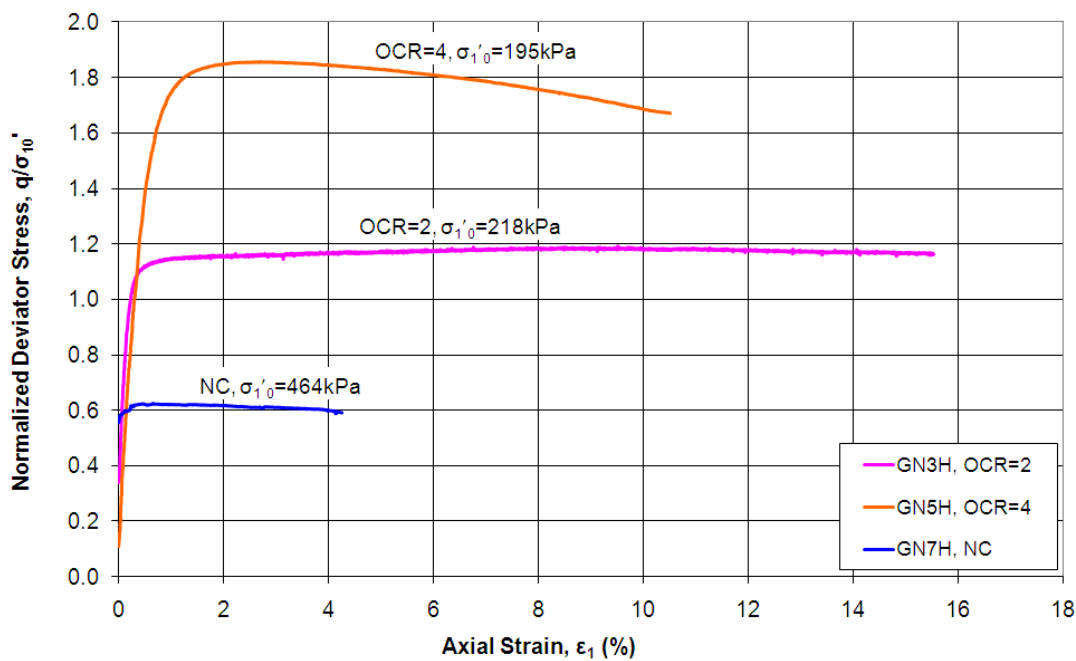


Figure 5.5 (d)

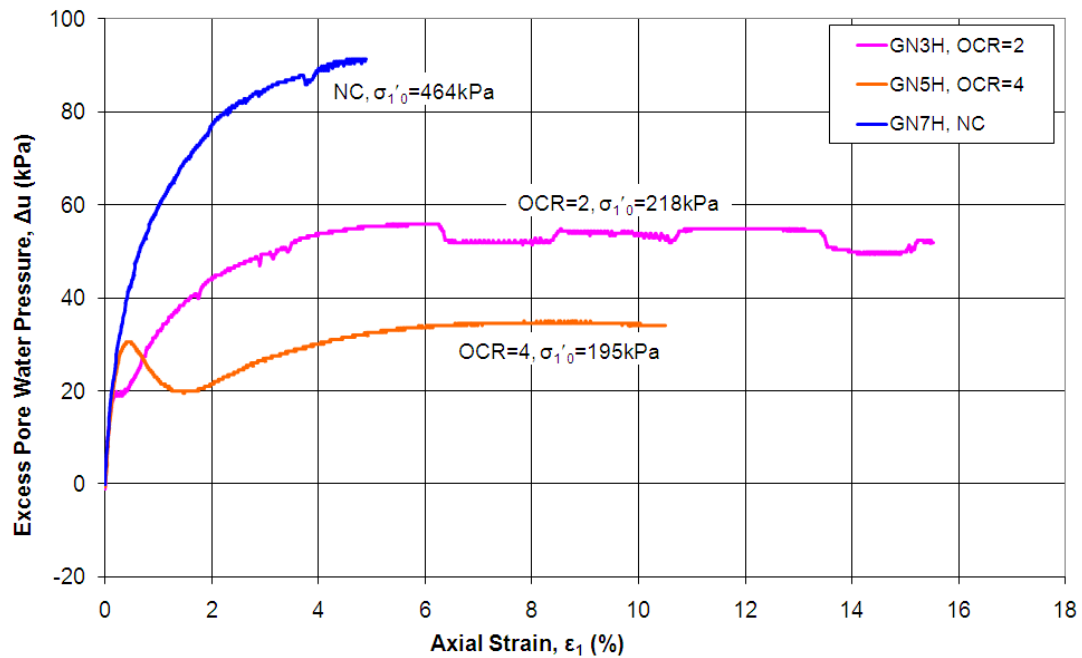


Figure 5.5 (e)

Figure 5.5 Undrained Behaviour of Horizontally Cut Intact Residual Soil from CK_0UC Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) Variations of q/p' with Axial Strain; (d) Normalized Stress-Strain Curves; (e) Variations of Excess Pore Water Pressure with Axial Strain

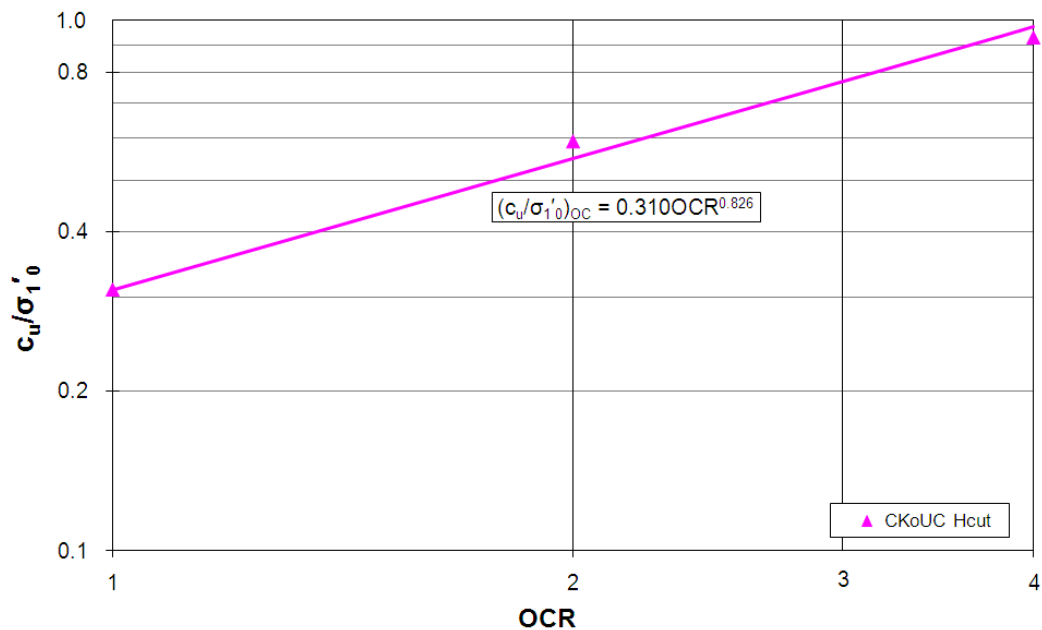


Figure 5.6 c_u/σ'_{10} vs. OCR Relationship Obtained from CK_0UC Tests on Horizontally Cut Intact Residual Soil Specimens

The test results of CK_0UC tests on horizontally cut intact residual soil specimens are also normalized against a coupled stress, T_0 (Yao & Zhu, 2009). The critical state line determined from tests on NC specimen is shown by dotted line in Figure 5.7. The slope M is 1.35, which is very close to the value of 1.34 for vertically cut specimens.

The Skempton's pore pressure parameter A increased as axial strain increased for NC specimen, and it approached a constant value for OC specimen (as shown in Figure 5.8(a)). The A value at failure (defined as the peak deviator stress) is also calculated for each test. The A_f values was observed to decrease with the OCR values as presented in Figure 5.8(b). Similar observation was made by on the CIU testing results by Simons (1960). The difference between the presented data and Simons' data could be resulted from different consolidation methods.

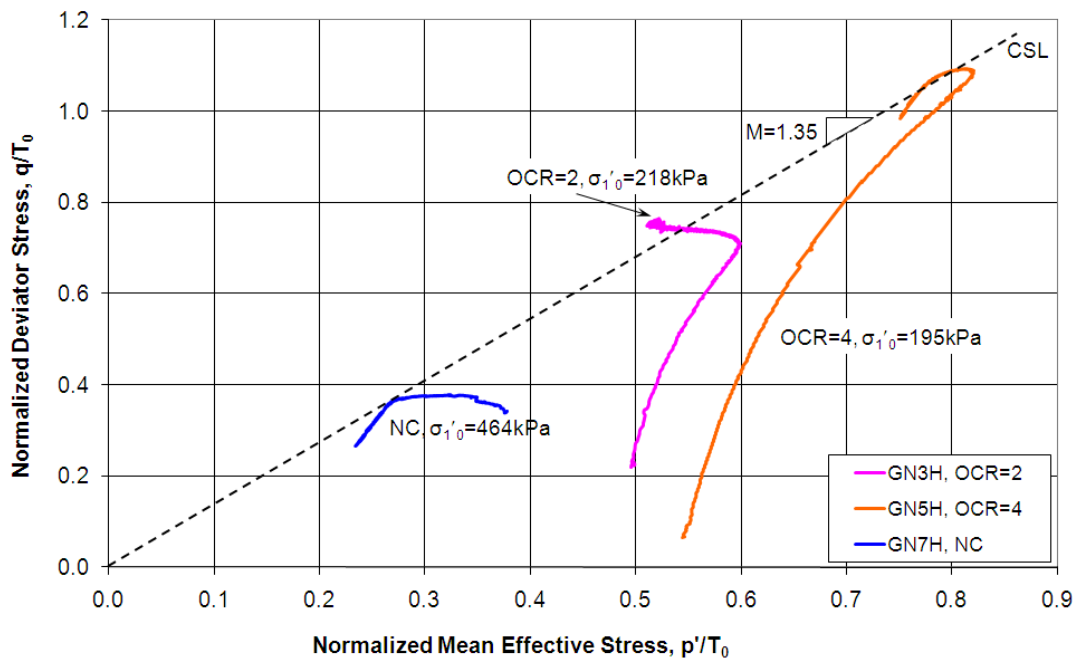


Figure 5.7 Normalized Stress Paths of Horizontally Cut Intact Residual Soil from CK_0UC Tests

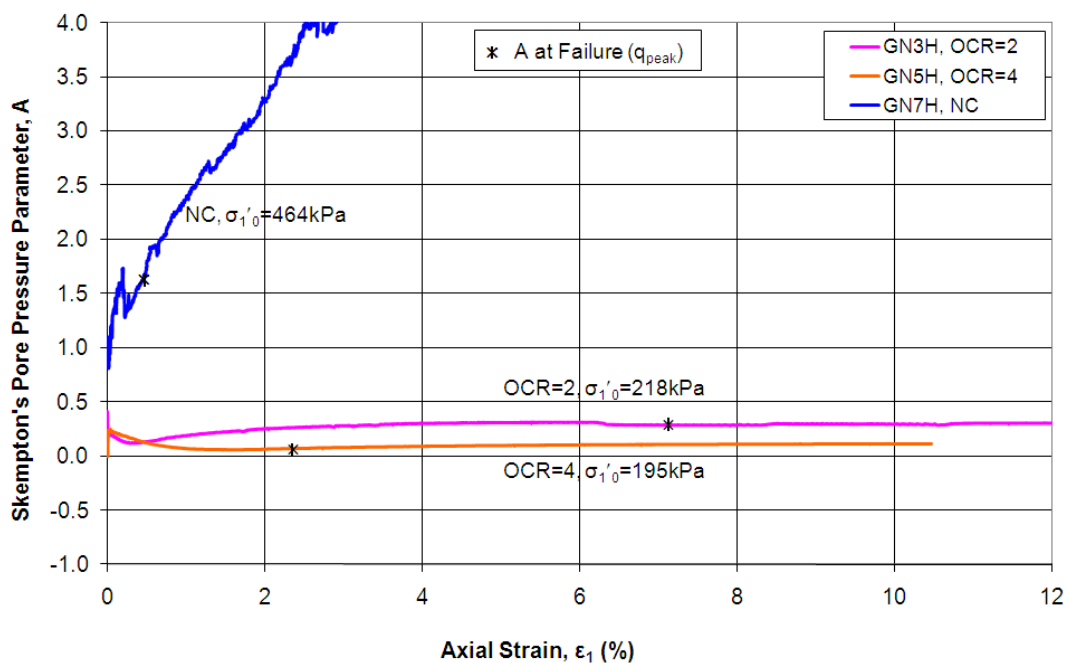


Figure 5.8 (a)

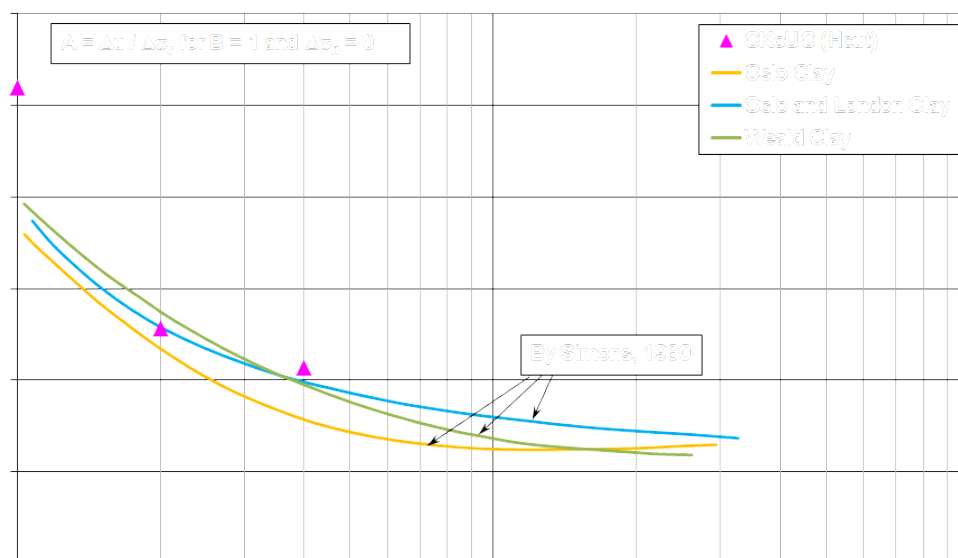


Figure 5.8 (b)

Figure 5.8 Variations of Skempton's Pore Pressure Parameter, A in CK_0UC Tests:
(a) A vs. Axial Strain Curves; (b) A_f vs. OCR Curves

5.2.3 Comparison

The CK_0UC tests conducted on vertically cut and horizontally cut intact residual soil specimens are compared in this section. The major findings are discussed.

Figure 5.9 compares the normalized stress-strain behaviour between vertically cut and horizontally cut specimens. For specimens in both orientations, the normalized deviator stress increased as OCR increased. However, no significant difference was observed in the normalized stress-strain behaviour between vertically cut and horizontally cut specimen. It indicated that the anisotropy effect could be eliminated by consolidating the specimens beyond the pre-consolidation stress, which consistent with the experimental study conducted by Parry & Nadarajah (1974). It also shows that testing soil specimens in different cutting orientation is not a good way to study strength anisotropy. The strength anisotropy behaviour could be understood better by conducting tests at different major principal stress orientations (such as compression, extension and direct simple shear tests) (O'Rourke et al., 1976).

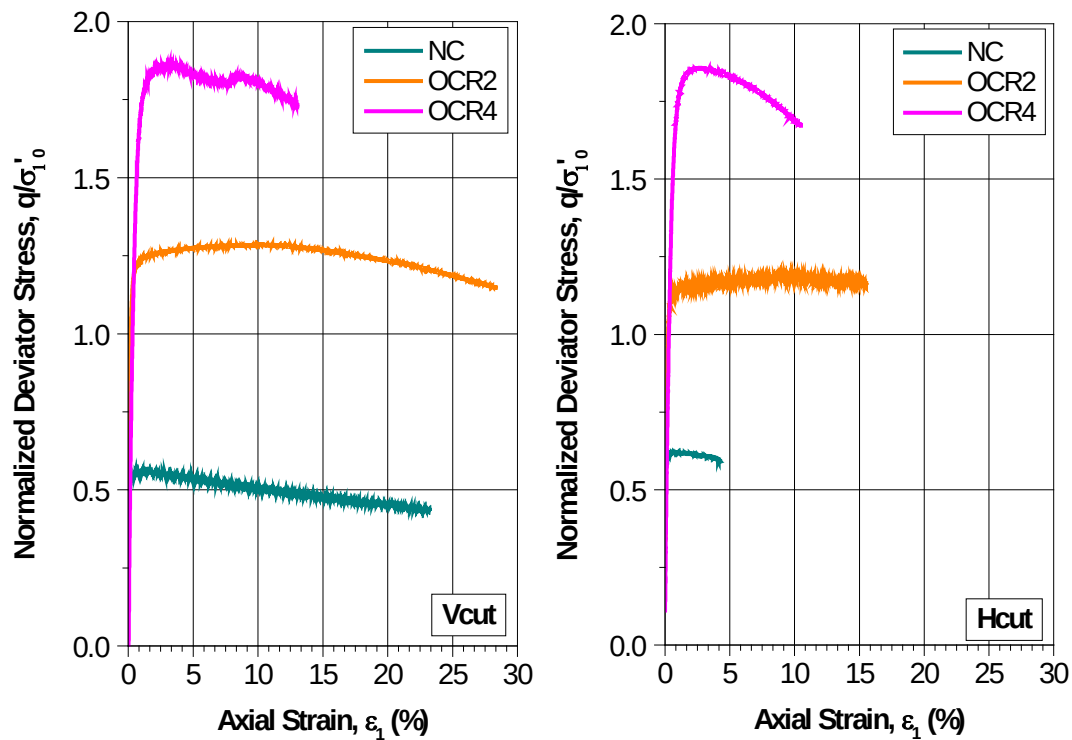


Figure 5.9 Comparison of Stress-Strain Behaviour during CK_0UC Tests

The failure envelopes of vertically cut and horizontally cut intact residual soil are compared in Figure 5.10. The failure envelope was determined from the points of peak effective stress ratio, $(q/p')_{\text{peak}}$. As shown in Figure 5.10(a), both vertically cut and horizontally cut intact residual soil specimens approached to a unique failure envelope. The solid lines in black color represent the failure envelope within the tested stress range, which has a slope of 1.2. The dash lines are the assumed failure envelopes passing through the origin, since the true cohesion of the intact residual soil was negligible as explained in Chapter 3. The corresponding q and p' values at $(q/p')_{\text{peak}}$ was plotted in Figure 5.10(b) for comparison also. It is clearly observed that the failure points determined for vertically cut and horizontally cut specimens are highly consistent. It is indicative that the effective failure envelopes are not affected by specimen orientation.

Figure 5.11 presents the comparison of $c_u/\sigma_1'{}_0$ versus OCR relationships. The data for both vertically cut and horizontally cut intact residual soil specimens fell into a consistent relationship. No observation of stress anisotropy was made for the intact residual soil. This indicated that the $c_u/\sigma_1'{}_0$ versus OCR relationships was independent of the specimen orientation. An average $(c_u/\sigma_1'{}_0)_{\text{NC}}$ value of 0.297 was taken from the data of normally consolidated vertically cut and horizontally cut specimens.

The variations of A_f with OCR for vertically cut and horizontally cut intact residual soil specimens were compared in Figure 5.12. At NC state, the A_f value in the vertical direction was much higher than the one in the horizontally direction. However, it reached a unique constant value as OCR increased, regardless of specimen orientation. It shows that the effect of specimen orientation was little on highly over-consolidated soil, since the soil bonds was broke down completely during consolidation. In Figure 5.12, it also presents the relationships determined from previous studies on Oslo clay, Oslo and London clay and Weald clay (Simons, 1960). It should be point out that the relationships established by Simons (1960) were from isotropically consolidated undrained (CIU) tests. As can be seen from Figure 5.12, the A_f value of the intact residual soil at NC state, especially the one

determined from vertically cut specimens, was much higher than the value for the other soils. This may be caused by the higher permeability of the intact residual soil, comparing to the clay studied by Simons. It is also possible due to the different consolidation methods adopted.

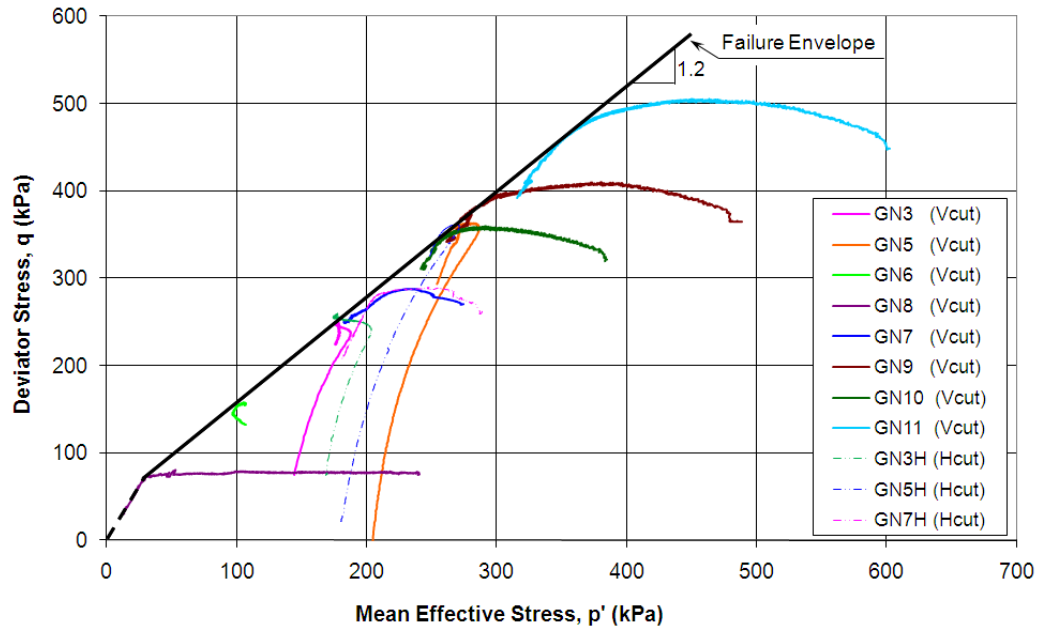


Figure 5.10 (a)

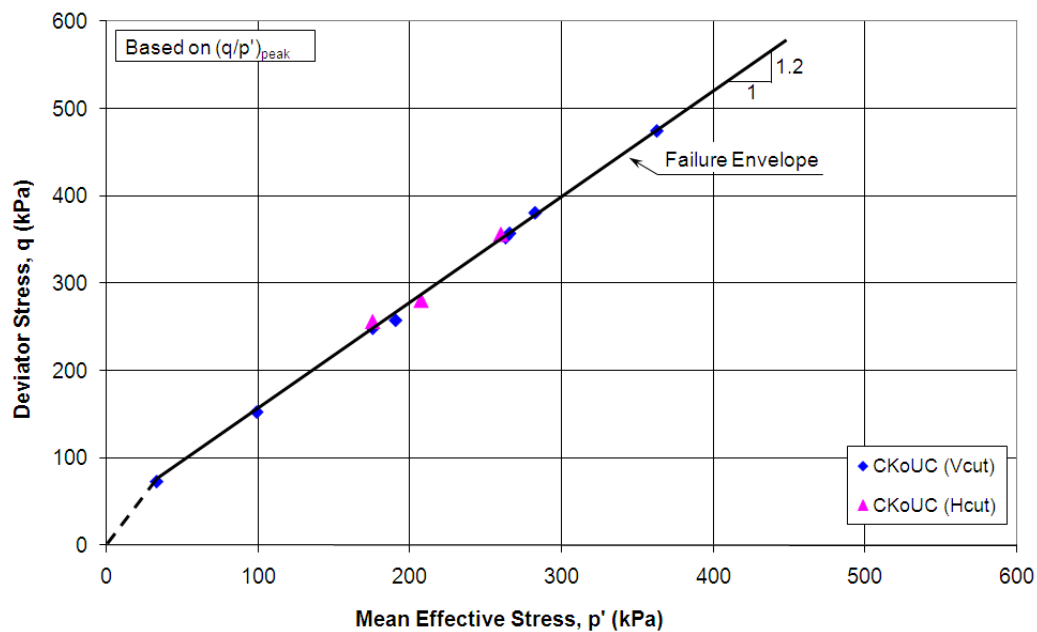


Figure 5.10 (b)

Figure 5.10 Comparison of Failure Envelopes Determined from CK_0UC Tests: (a)

From Stress Paths; (b) From $(q/p')_{peak}$

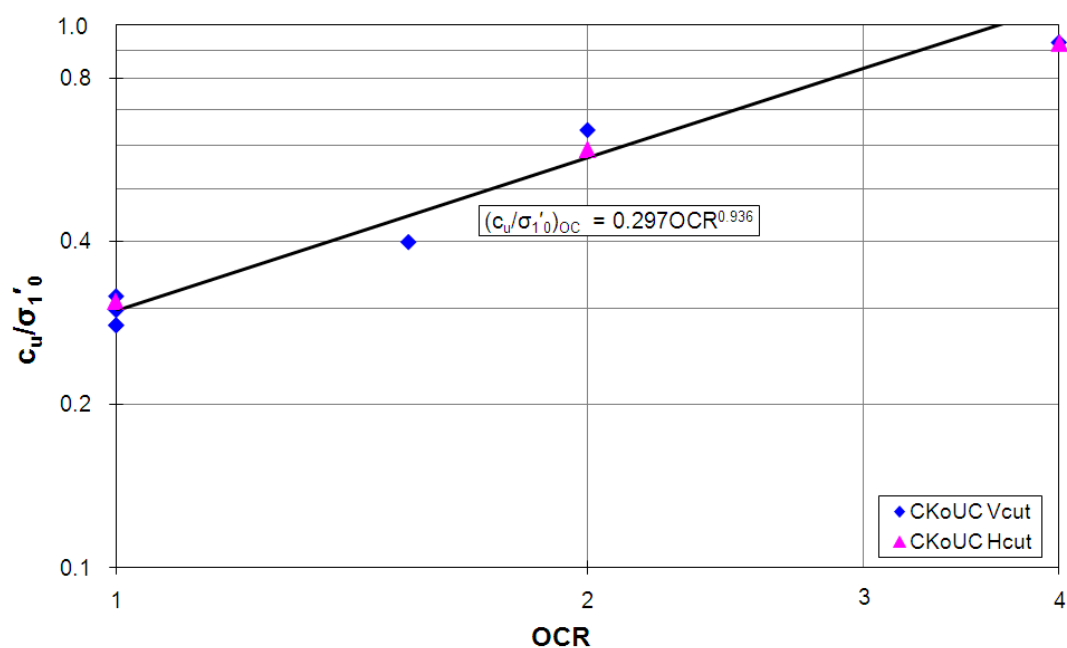


Figure 5.11 Comparison of c_u/σ'_{10} vs. OCR Relationships Obtained from CK₀UC Tests

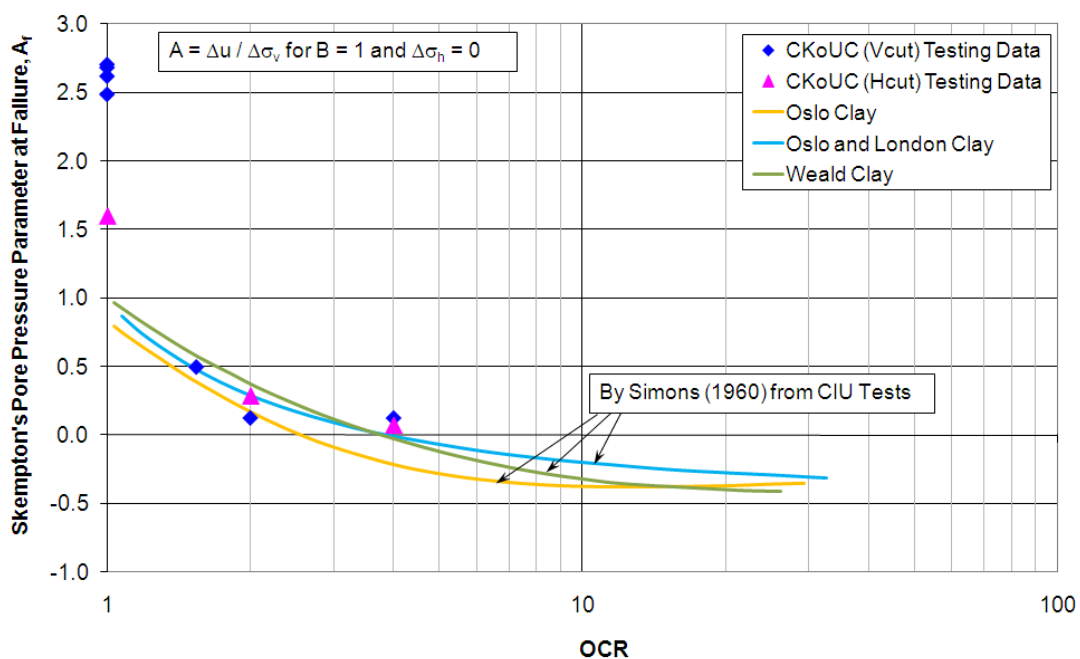


Figure 5.12 Comparison of A_f vs. OCR Relationships Obtained from CK₀UC Tests on the Intact Residual Soil and Previous Studies by Simons (1960)

5.3 K_0 Consolidated Undrained Triaxial Extension (CK_0UE) Tests

5.3.1 Vertically Cut Specimens

The CK_0UE tests conducted on vertically cut intact residual soil specimens are summarized in Table 5.3. The stress behaviour resulted from four CK_0UE tests is shown from Figure 5.13(a) to (e).

Table 5.3 Summary of CK_0UE Tests on Vertically Cut Intact Residual Soil Specimens

Test No.		EGN1	EGN2	EGN3	EGN4
Pressure for Saturation (kPa)	Cell	860	860	1010	1010
	Back	850	850	1000	1000
OCR		2	1.53	4	1
Shearing Rate (mm/min)		0.005	0.007	0.007	0.007
Weight (g)	Pre-Test	1024.76	1043.28	1046.92	1054.43
	Post-Test	1027.44	1056.22	1023.84	1038.04
Water Content (Pre-Test) (%)	Average	30.5	30.65	30.89	31.84
Water Content (Post-Test) (%)	Average	31.59	32.77	28.74	30.43
	Top	31.01	31.09	28.56	30.35
	Middle	31.39	31.64	28.58	30.39
	Bottom	31.18	32.45	28.96	30.38
	Shear Band	32.79	34.97	28.88	30.66
σ_1' (kPa)		195.15	195.81	195.45	452.22
q_0 (kPa)		90.65	132.31	28.95	265.72
p'_0 (kPa)		134.32	107.60	176.65	275.57
q_{peak} (kPa)		-119.65	-92.2	-258.91	-208.6
ε_1 at q_{peak} (%)		-4.31	-2.86	-7.01	-6.01
$(q/p')_{peak}$		-1.22	-1.29	-1.20	-1.22
$R_{max} = (\sigma_1'/\sigma_3')_{max}$		7.31	10.33	6.92	7.26
ε_1 at $(q/p')_{peak}$ (%)		-4.06	-2.60	-6.98	-5.33
Secant Friction Angle, ϕ' (°)		30.40	32.00	29.90	30.51

The NC specimen was anisotropically (K_0) consolidated to 452 kPa of σ_1' value, and the rest were K_0 consolidated based on the in-situ vertical stress of the intact

residual soil (195 kPa). An OCR value of 2 (EGN1) was imposed by consolidating the specimen to σ_1' value of 390kPa, then unloading to 195kPa. The same procedure was adopted for the specimen with OCR value of 4 (EGN3), except the specimen was consolidated to 780kPa before unloading. It is important to mention that the specimen with OCR value of 1.53 (EGN2) was anisotropically consolidated to a σ_1' value of 195kPa directly without unloading, which was below the pre-consolidation stress of vertically cut intact residual soil (300kPa).

The effective stress paths resulted from CK_0UE tests are presented in Figure 5.13(a). The failure envelope was determined based on the peak effective stress ratio $((q/p')_{peak})$. For vertically cut intact residual soil specimens, the slope of the failure envelope is 1.23 and with a friction angle of 30.70° .

Figure 5.13(b) presents the stress-strain curves obtained from the same tests. The deviator stress reached the peak value at relatively large strain (3-7%). For OC specimens, which were consolidated to the same stress level, the higher the OCR value, the higher the peak deviator stress, q_{peak} .

Figure 5.13(c) shows that variation of stress ratio with axial strain were quite consistent for tests EGN1, EGN3 and EGN4. The change of peak stress ratio, $(q/p')_{peak}$ with OCR is not observed. It is also observed the specimen EGN2 behaved differently from the other specimens. It reached the peak stress ratio much earlier than the rest. This may be because the specimen EGN2 was consolidated to the lowest stress level, which was below the pre-consolidation stress. The soil bonds were not destroyed completely during consolidation as the others did. The existence of soil bonds may affect the variation of stress ratio significantly, as observed in CK_0UC tests.

The stress-strain behaviour of vertically cut intact residual soil is normalized against the initial effective vertical stress ($\sigma_1'{}_0$) (as shown in Figure 5.13(d)). It is observed that the peak normalized deviator stress increased as OCR increased. It is also noted that a significant amount of strain softening was observed for OC specimens. The

corresponding c_u/σ'_{10} versus OCR relationship is presented in Figure 5.14. The NC specimen reached a c_u/σ'_{10} value of 0.234. A power relationship was plotted and the equation was shown.

The excess pore water pressure generated during undrained shearing is shown in Figure 5.13(e). It decreased initially until peak deviator stress was reached, and increased subsequently with further shearing. Such behaviour was typical for OC clay in undrained extension tests. For NC specimen (EGN4), the excess pore water pressure decreased during undrained shearing. As can be seen from Figure 5.13(e), the excess pore water pressures in test EGN1, EGN2 and EGN4 were increased slightly at the beginning of shearing. This may be due to a sudden change from K_0 consolidation to undrained shearing. Hence, a proper creep should be conducted after consolidation.

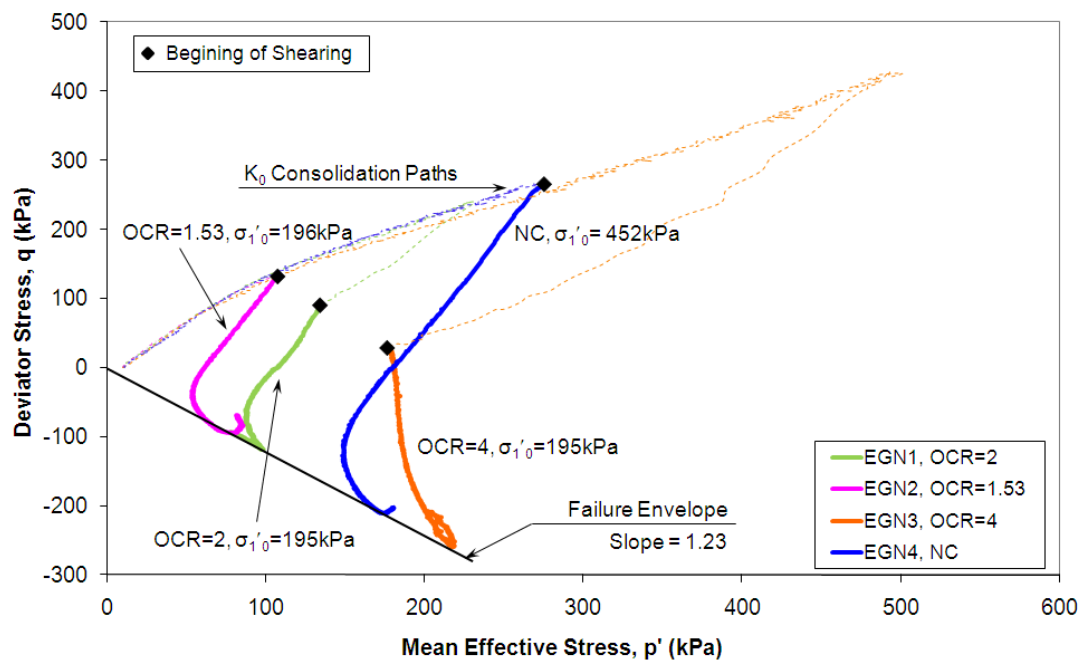


Figure 5.13 (a)

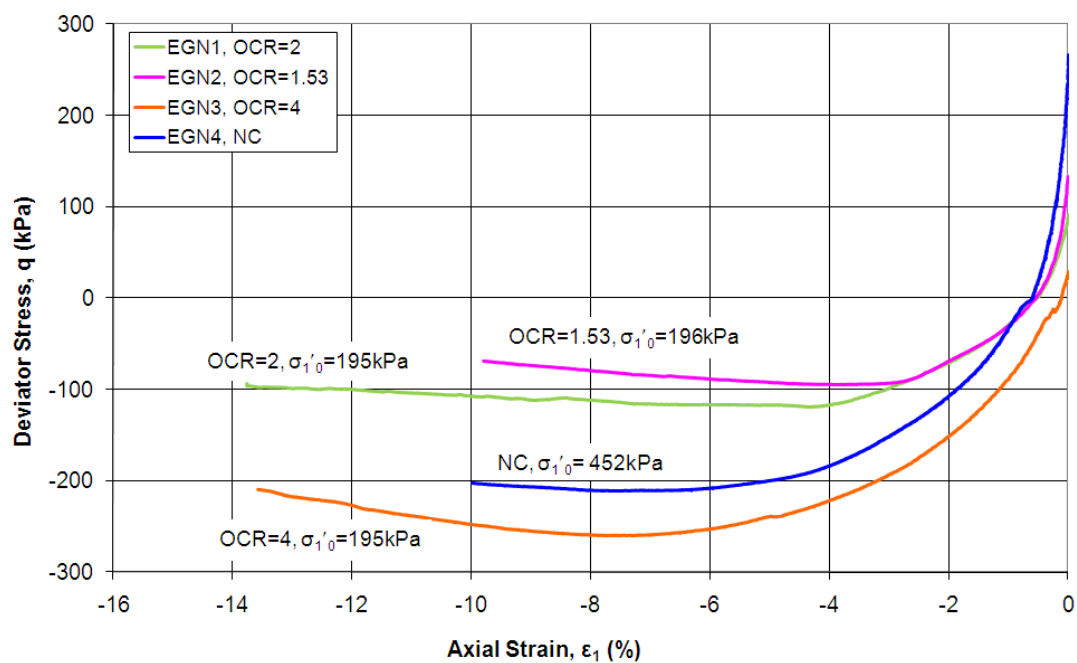


Figure 5.13 (b)

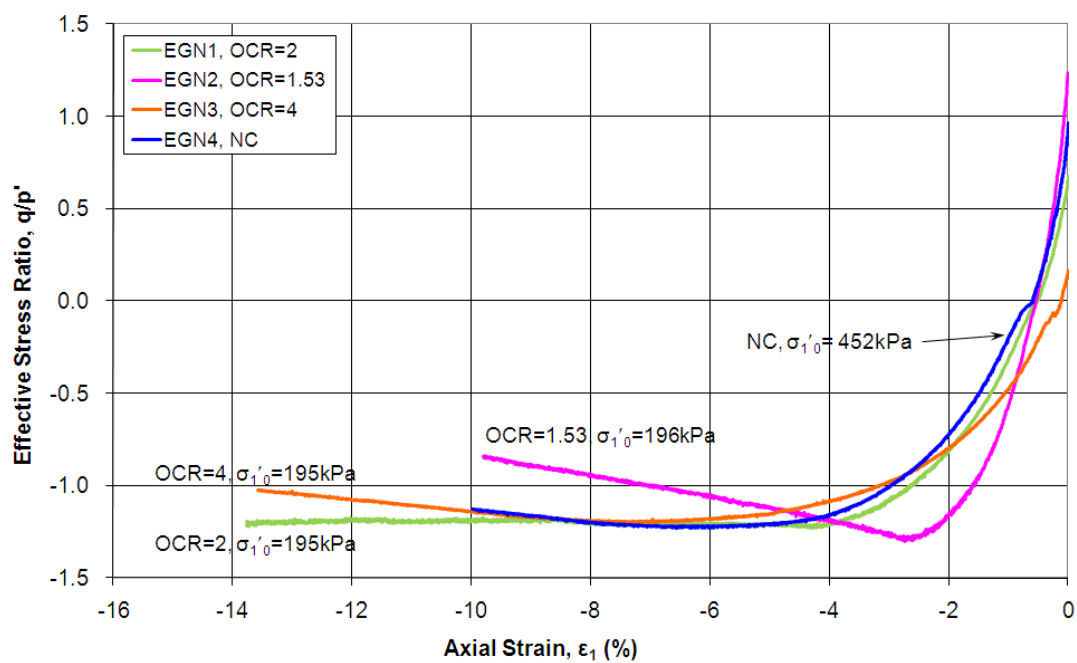


Figure 5.13 (c)

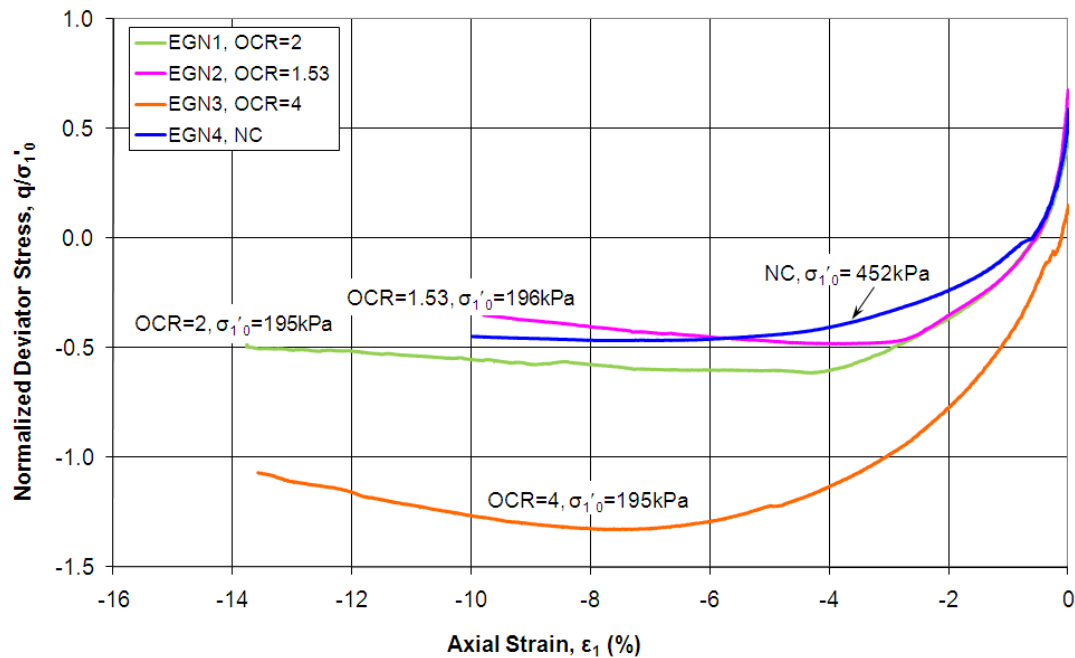


Figure 5.13 (d)

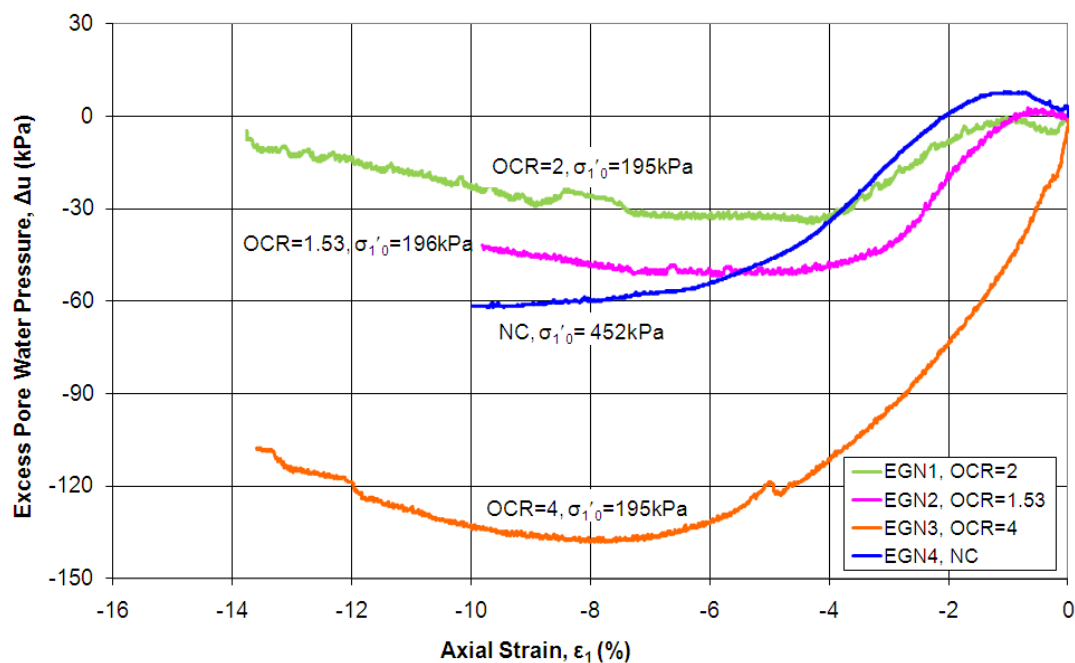


Figure 5.13(e)

Figure 5.13 Undrained Behaviour of Vertically Cut Intact Residual Soil from CK_0 UE Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) q/p' vs. $\epsilon_1\%$ Curves; (d) Normalized Stress-Strain Curves; (e) Δu vs. $\epsilon_1\%$ Curves

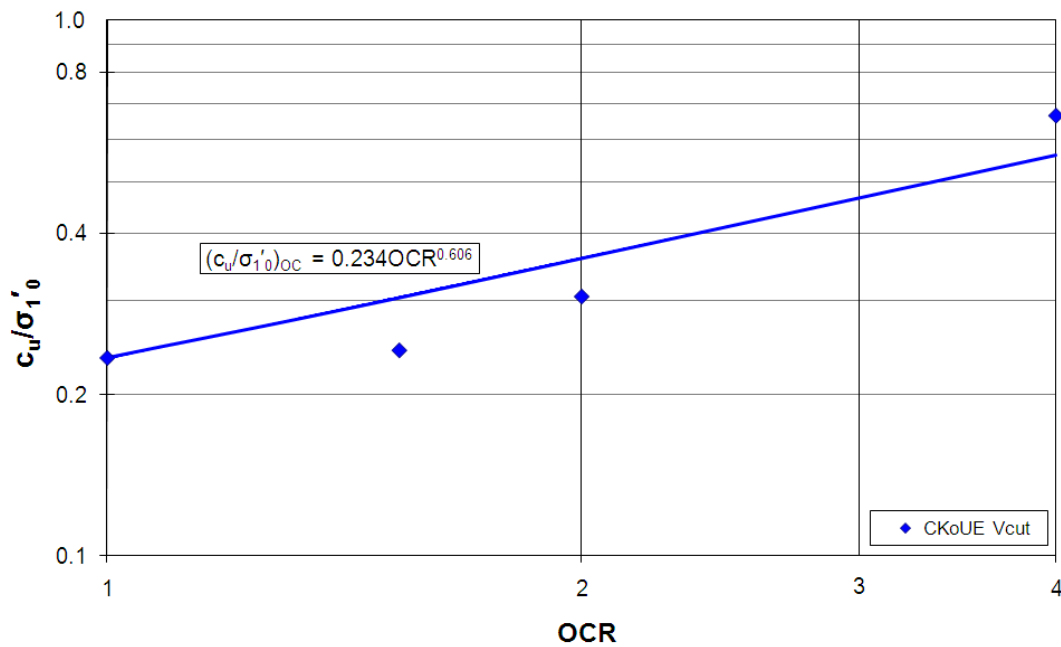


Figure 5.14 c_u/σ'_{10} vs. OCR Relationship Obtained from CK₀UE Tests on Vertically Cut Intact Residual Soil Specimens

Same as the CK₀UC test results, the stress paths obtained from CK₀UE tests were normalized against a coupled stress, T_0 (Yao & Zhu, 2009). As shown by dotted line in Figure 5.15, the critical state line as determined from CK₀UE tests on vertically cut intact residual soil has a slope of $M=1.22$.

As shown by Law & Holtz (1978), with rotation of principal stresses, the Skempton's pore pressure parameter A under triaxial extension condition should be calculated as follow:

$$A = 1 - \frac{\Delta u}{\Delta \sigma_v} \text{ for } B = 1 \text{ and } \Delta \sigma_h = 0 \quad (5-4)$$

where Δu is the change in excess water pressure, $\Delta \sigma_v$ is the change in vertical stress and $\Delta \sigma_h$ is the change in horizontal stress. The variation of A with axial strain determined from CK₀UE tests under constant cell pressure is presented in Figure 5.16(a). For OC specimens, the A value reduced rapidly at beginning. This is because the response of pore water pressure was much slower than the change in deviator stress at the beginning stage of shearing. After reached the minimum value,

the Skempton's pore pressure parameter increased back and reached a constant value. In contrast, the NC specimen behaved exactly opposite.

As described by Das (1998), the Skempton's pore pressure parameter at failure (A_f) was determined at the point when peak deviator stress occurs. The curves determined from CIU tests on different soils (Simons, 1960) were also shown for reference. As can be seen from Figure 5.16(b), the A_f value decreased with OCR. Similar observation was made on CK_0UC tests. The variations of A_f with OCR in CK_0UE tests will be compared between vertically cut and horizontally cut intact residual soil later in this chapter.

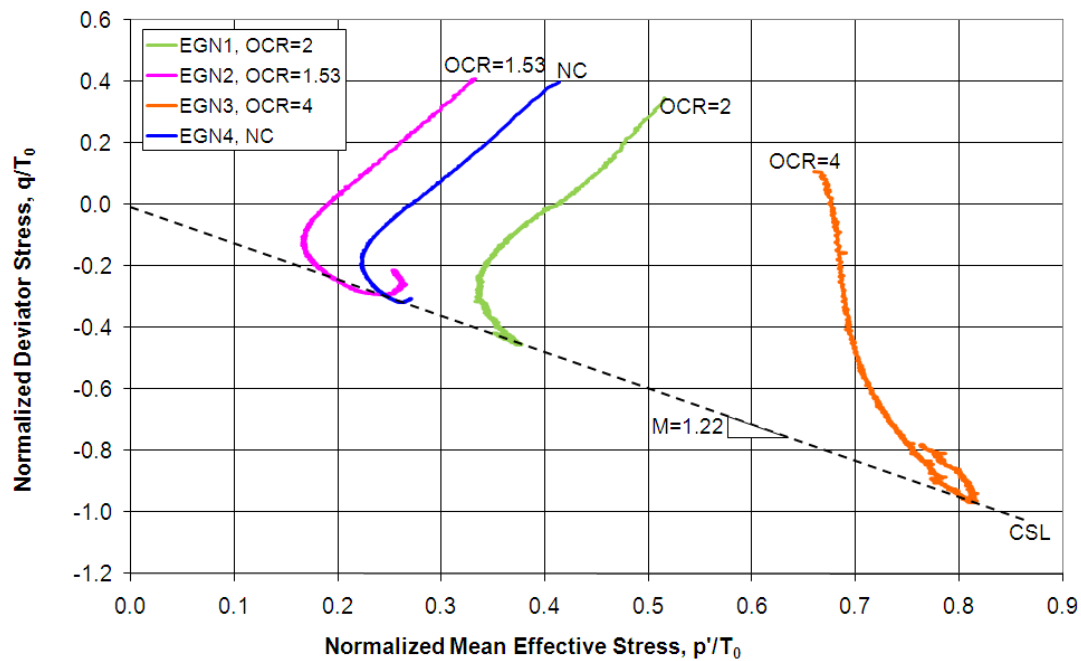


Figure 5.15 Normalized Stress Paths of Vertically Cut Intact Residual Soil from CK_0UE Tests

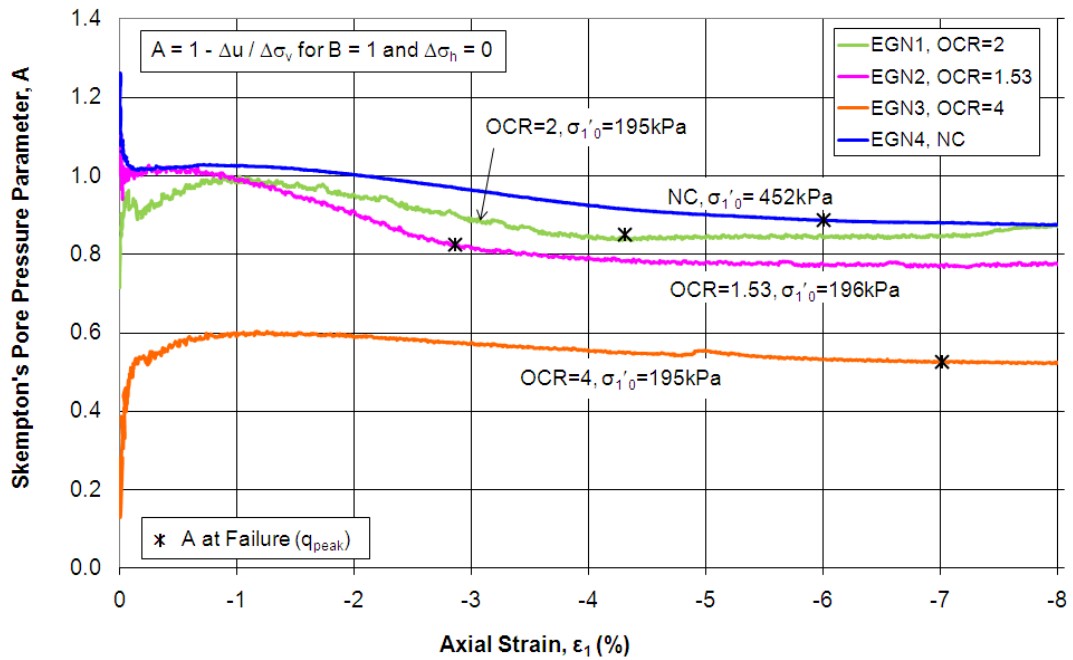


Figure 5.16 (a)

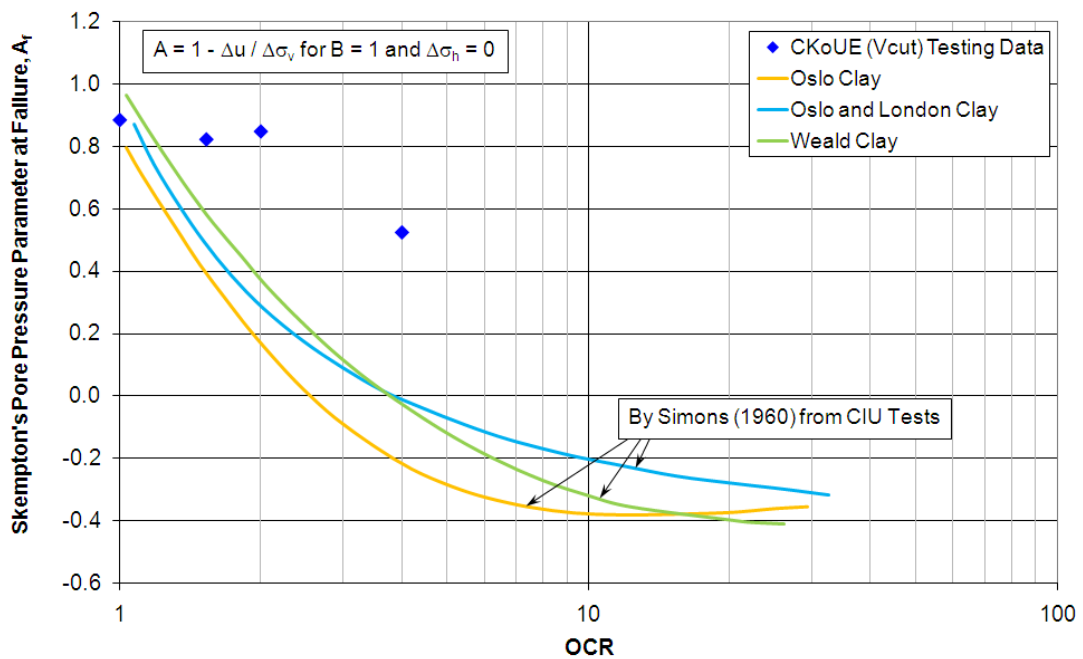


Figure 5.16 (b)

Figure 5.16 Variations of Skempton's Pore Pressure Parameter, A in CK_0UE Tests:
(a) A vs. Axial Strain Curves; (b) A_f vs. OCR Curves

5.3.2 Horizontally Cut Specimens

Horizontally cut intact residual soil was also used for CK₀UE tests. The tests conducted are listed in Table 5.4. The test results are presented in Figure 5.17.

Table 5.4 Summary of CK₀UE Tests on Horizontally Cut Intact Residual Soil Specimens

Test No.		EGN1H	EGN2H	EGN3H	EGN4H
Pressure for Saturation (kPa)	Cell	860	860	1010	1010
	Back	850	850	1000	1000
OCR		2	1.54	4	1
Shearing Rate (mm/min)		0.005	0.007	0.007	0.007
Weight (g)	Pre-Test	1028.97	1028.25	1035.04	1024.07
	Post-Test	1030.73	1059.28	1040.12	-
Water Content (Pre-Test) (%)	Average	32.77	31.56	31.91	31.72
Water Content (Post-Test) (%)	Average	32.72	35.85	32.38	-
	Top	32.42	39.40	32.50	-
	Middle	33.68	34.02	31.71	-
	Bottom	32.81	34.15	32.94	-
	Shear Band	31.97	39.40	at top	-
σ'_{10} (kPa)		197.01	194.89	199.74	447.17
q_0 (kPa)		79.01	133.39	9.74	248.67
p'_0 (kPa)		144.34	108.46	193.25	281.39
q_{peak} (kPa)		-195.11	-136.70	-262.20	-213.91
ε_1 at q_{peak} (%)		-5.50	-3.69	-4.68	-4.15
$(q/p')_{\text{peak}}$		-1.18	-1.33	-1.06	-1.10
$R_{\text{max}} = (\sigma'_1/\sigma'_3)_{\text{max}}$		6.52	12.71	4.58	5.14
ε_1 at $(q/p')_{\text{peak}}$ (%)		-2.68	-2.05	-4.28	-3.43
Secant Friction Angle, ϕ' (°)		29.79	32.98	26.70	27.97

Five CK₀UE tests were conducted on horizontally cut intact residual soil specimens. Same consolidation method as described in Section 5.3.1 was adopted to create different OCR values. The stress paths for both consolidation and shearing are

shown in Figure 5.17(a). The failure envelope is assumed to pass through the origin as CK_0UC tests. It is shown by line BC within the tested stress level.

The stress-strain curves and the variation of effective stress ratio during CK_0UE tests are presented in Figure 5.17(b) and (c). For horizontally cut intact residual soil, strain softening was observed for both NC and OC specimen (as shown in Figure 5.17(b)). The peak effective stress ratio occurred before the deviator stress reached its peak value. As seen from Figure 5.17(c), the effective stress ratio determined from test EGN2H was very different from the others. It reached a higher peak value of q/p' at a smaller axial strain. This could be due to the effect of soil bonds, since specimen EGN2H was consolidated to a lowest stress level, which was below the pre-consolidation stress of horizontally cut intact residual soil.

The stress-strain curves were normalized by initial effective vertical stress in Figure 5.17(d). The normalized deviator stress increased with OCR. The corresponding c_u/σ'_{10} versus OCR relationship obtained from CK_0UE tests on horizontally cut intact residual soil specimens are presented in Figure 5.18. The NC specimen reached a c_u/σ'_{10} of 0.239. A power relationship was plotted and the equation was shown.

Figure 5.17(e) presents the generation of excess pore water pressure during the CK_0UE tests. It is noted that the excess pore water pressure increased after the peak effective stress ratio was achieved. This may be caused by the stress concentration or the generation of shear band after failure.

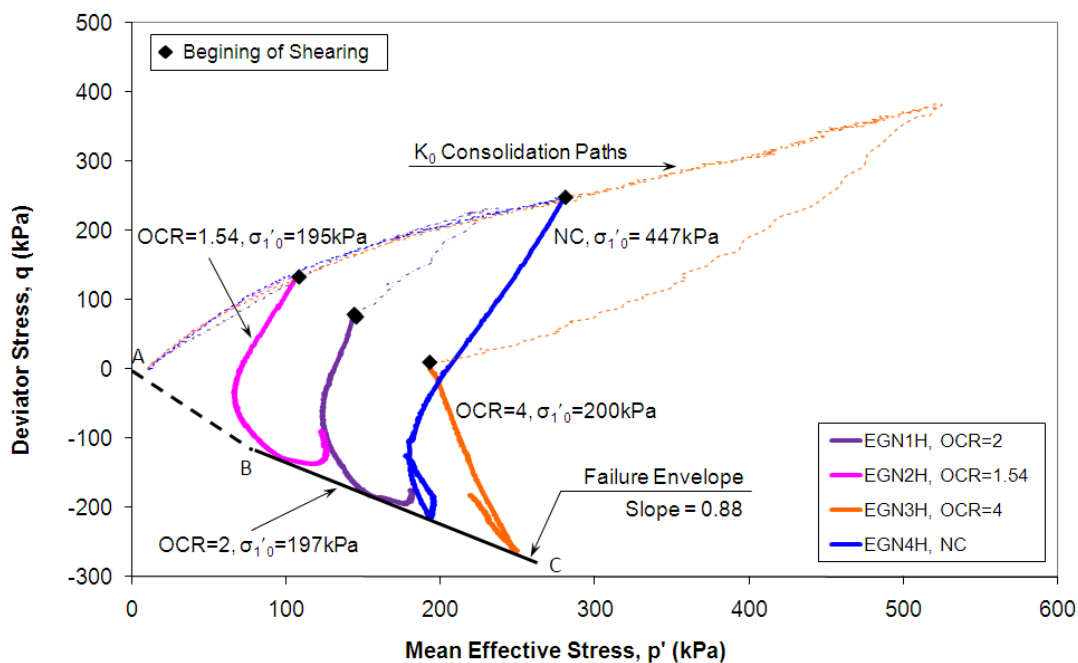


Figure 5.17 (a)

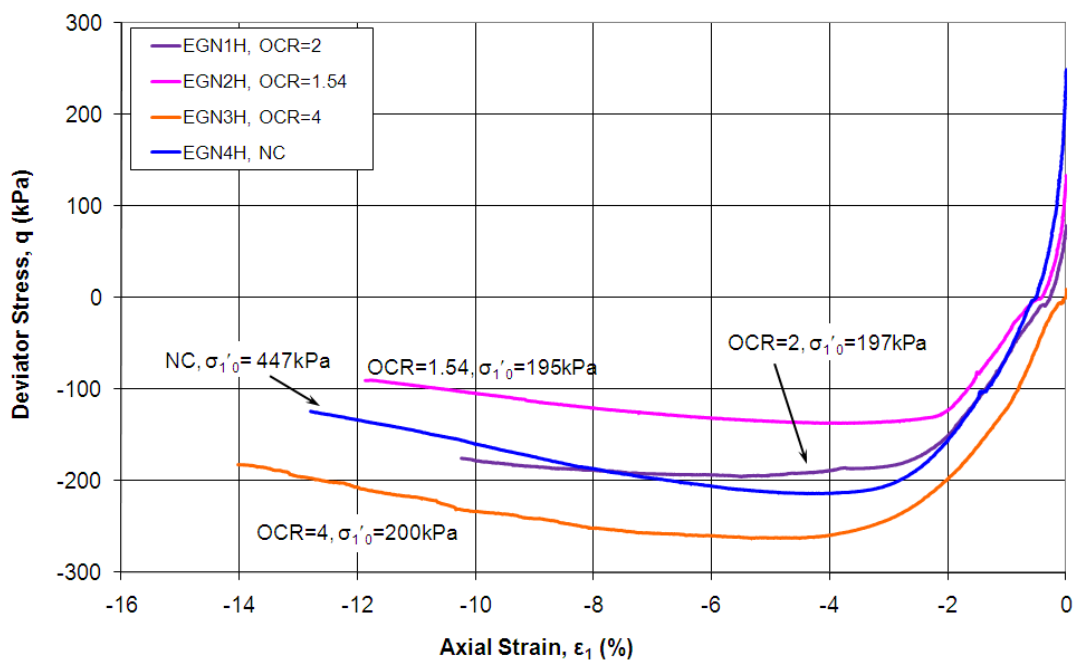


Figure 5.17 (b)

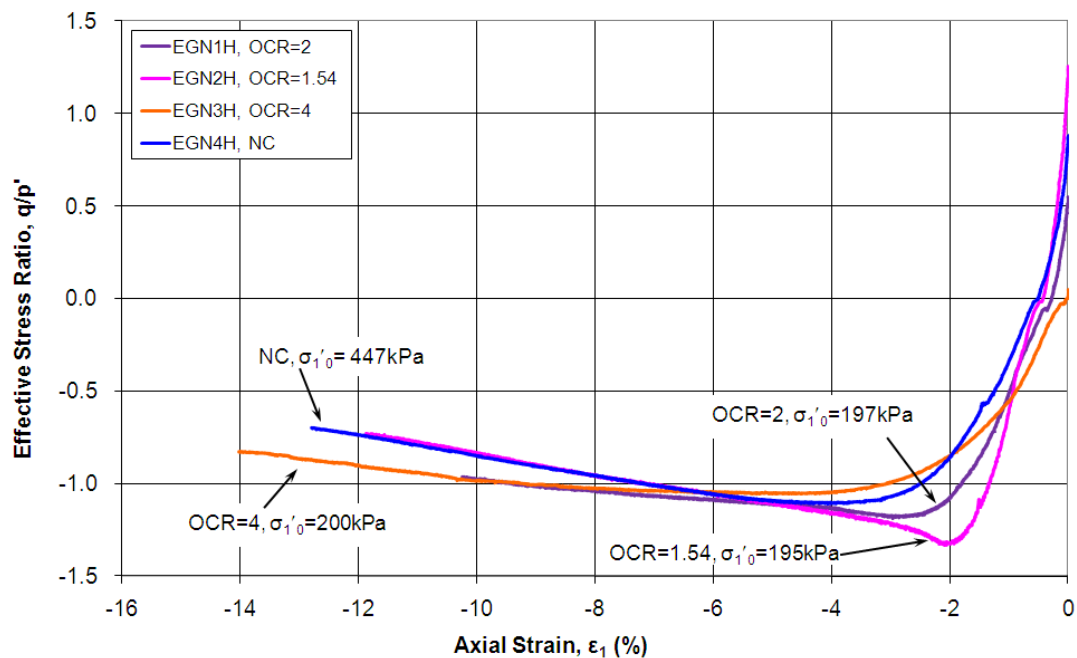


Figure 5.17 (c)

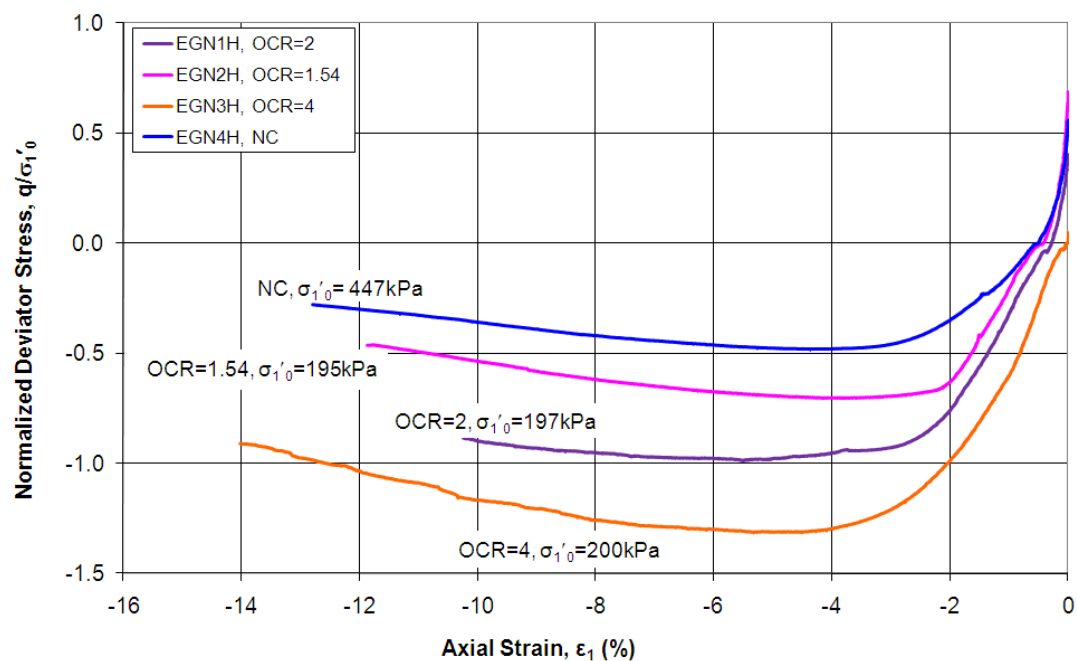


Figure 5.17 (d)

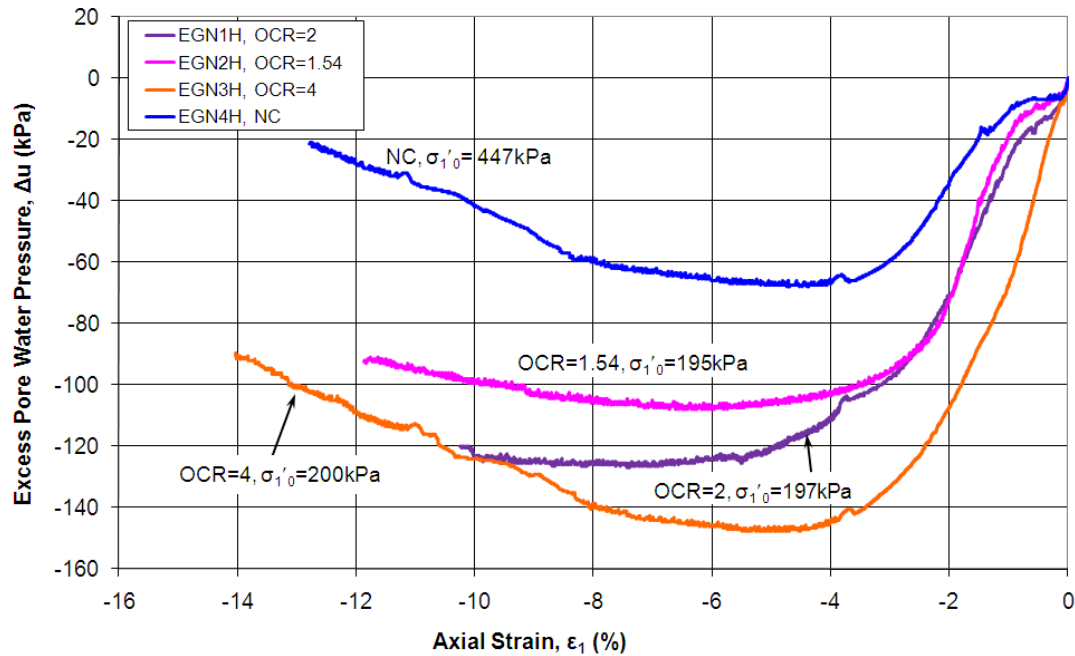


Figure 5.17 (e)

Figure 5.17 Undrained Behaviour of Horizontally Cut Intact Residual Soil from CK_0UE Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) Variations of q/p' with Axial Strain; (d) Normalized Stress-Strain Curves; (e) Variations of Excess Pore Water Pressure with Axial Strain

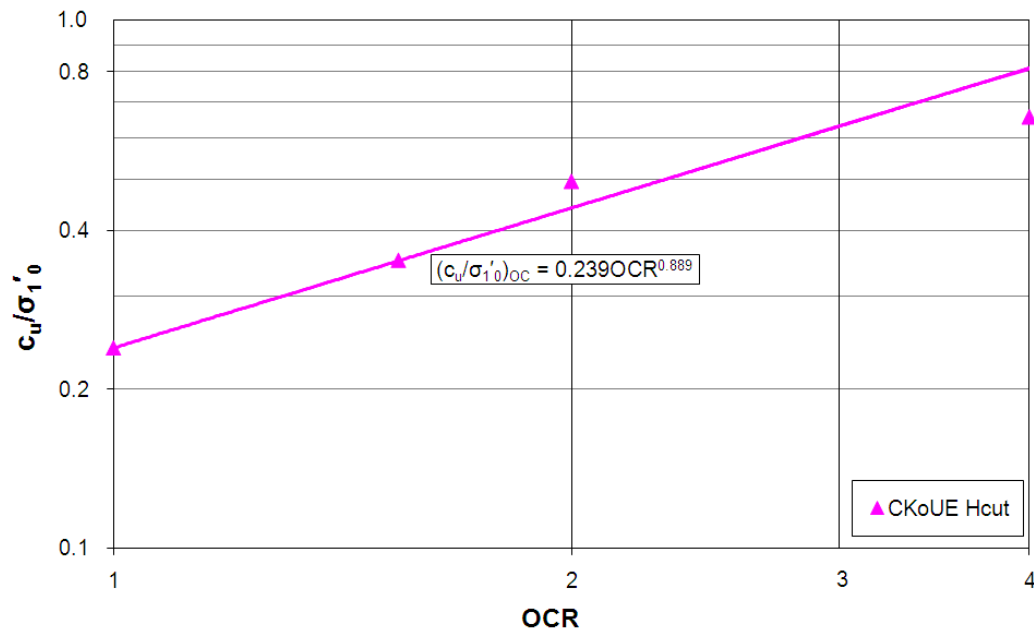


Figure 5.18 c_u/σ'_{10} vs. OCR Relationship Obtained from CK_0UE Tests on Horizontally Cut Intact Residual Soil Specimens

Figure 5.20 presents the normalized effective stress paths of horizontally cut intact residual soil. The critical state line determined from the test on NC specimen is shown as a dotted line. It has a slope of -1.1.

The variation of Skempton's pore pressure parameter, A with axial strain during CK_0UE testing conditions is shown in Figure 5.20(a). The A_f was determined at the point when q_{peak} was reached, which was observed to decrease with OCR. Similar observation was made during CK_0UC tests for both vertically cut and horizontally cut specimens. It should point out that the data reported by Simons' as shown in Figure 5.20(b) were determined from CIU tests on different types of clay. The consolidation method could significantly affect the A_f values, especially under NC state, as reported earlier in this chapter. It is also important to mention that different equations were proposed to calculate A value at compression and extension conditions. Therefore, difference between presented data and Simons' data were expected.

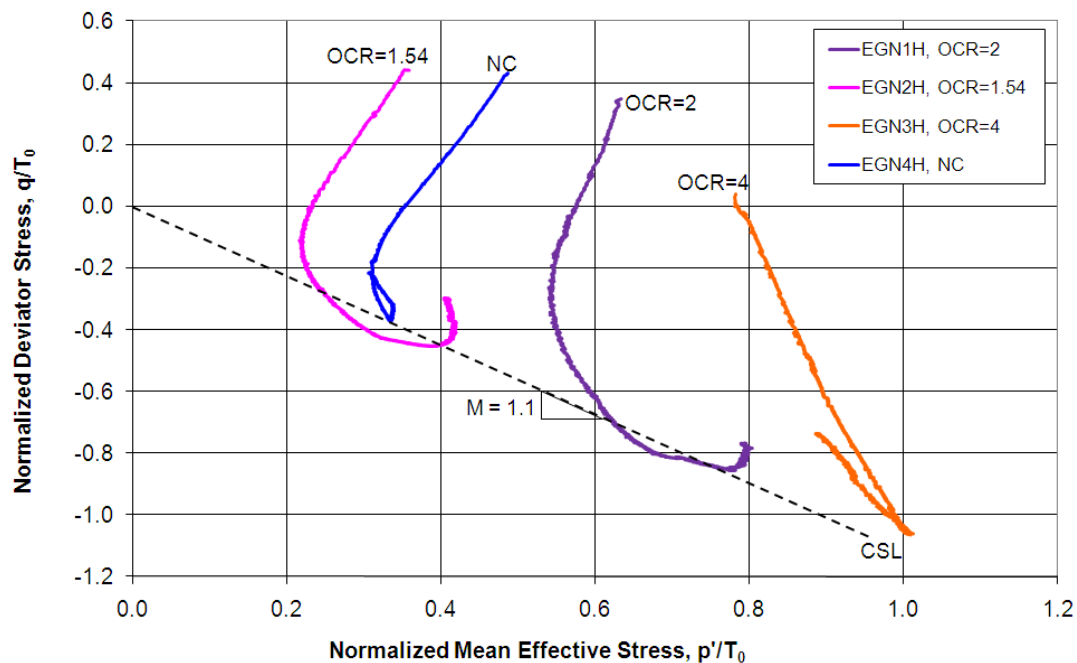


Figure 5.19 Normalized Stress Paths of Horizontally Cut Intact Residual Soil from CK_0UE Tests

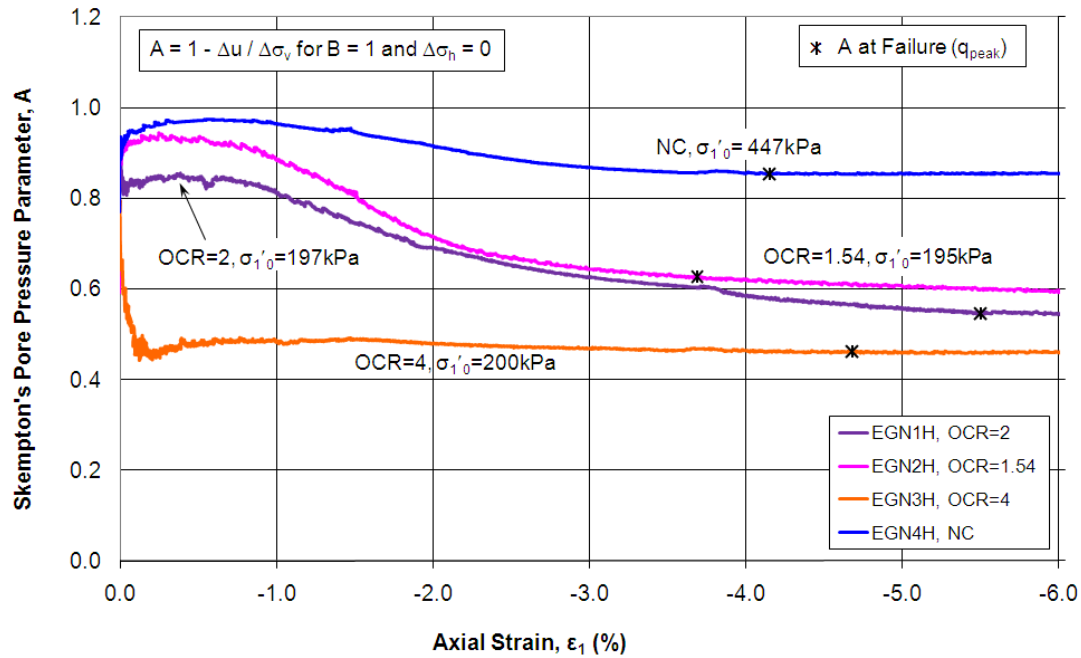


Figure 5.20 (a)

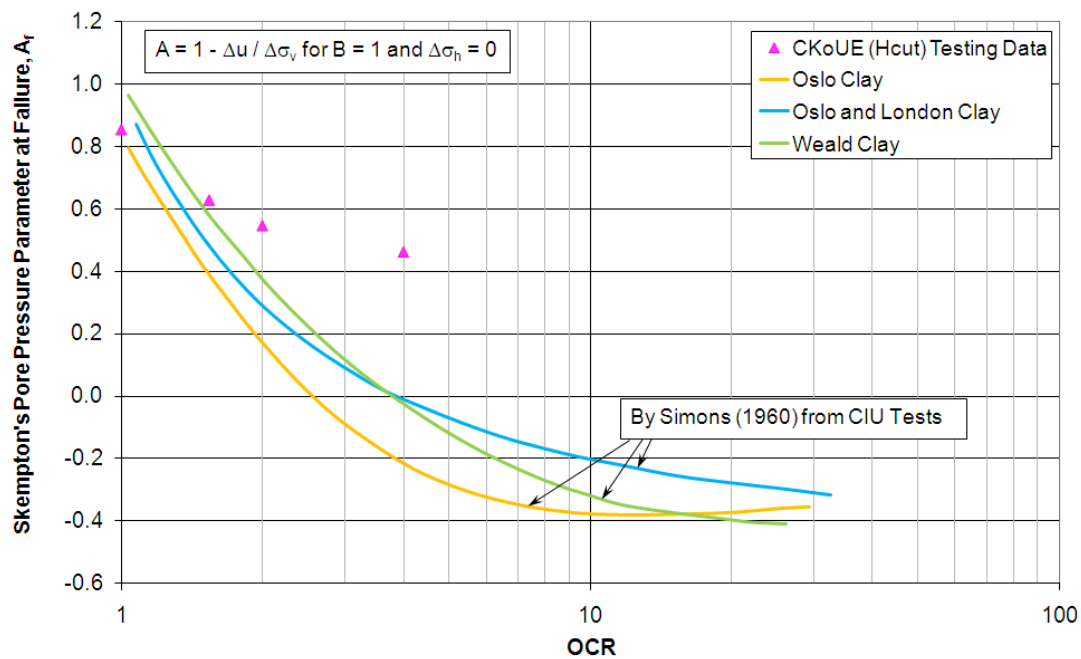


Figure 5.20 (b)

Figure 5.20 Variations of Skempton's Pore Pressure Parameter, A in CK_0UE Tests:

(a) A vs. Axial Strain Curves; (b) A_f vs. OCR Curves

5.3.3 Comparison

The normalized stress-strain curves determined from CK_0UE tests are compared between vertically cut and horizontally cut intact residual soil specimens. As seen from Figure 5.21, at the same OCR, horizontally cut specimens reached a higher peak than vertically cut specimens.

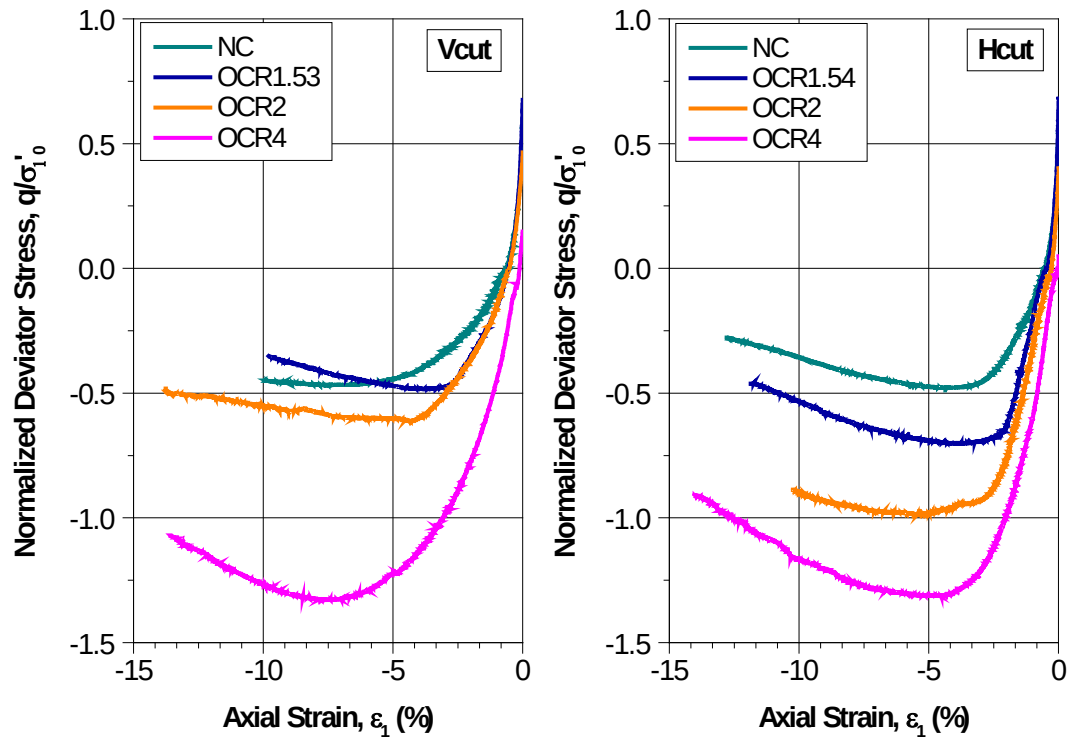


Figure 5.21 Comparison of Normalized Stress-Strain Behaviour Determined from CK_0UE Tests

The failure envelopes of vertically cut and horizontally cut intact residual soil specimens are compared in Figure 5.22. The failure envelope was determined from the point of peak (q/p'). Within the testing stress range, both vertically cut and horizontally cut specimens approached to the same failure envelope with a slope of one (as shown in Figure 5.22(a)). By plotting the corresponding failure points in Figure 5.22(b), it is noted that all the points fell into a straight line. Similar observation has been made from CK_0UC tests (as presented in Section 5.2). It is an indication that the failure envelope is not greatly affected by the specimen orientation.

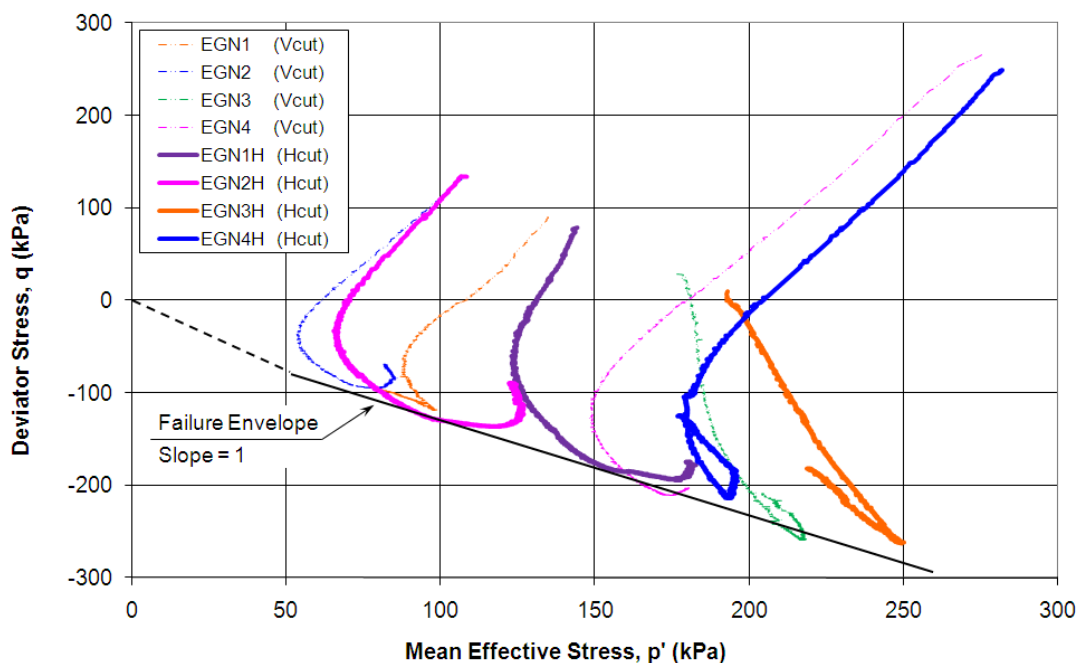


Figure 5.22 (a)

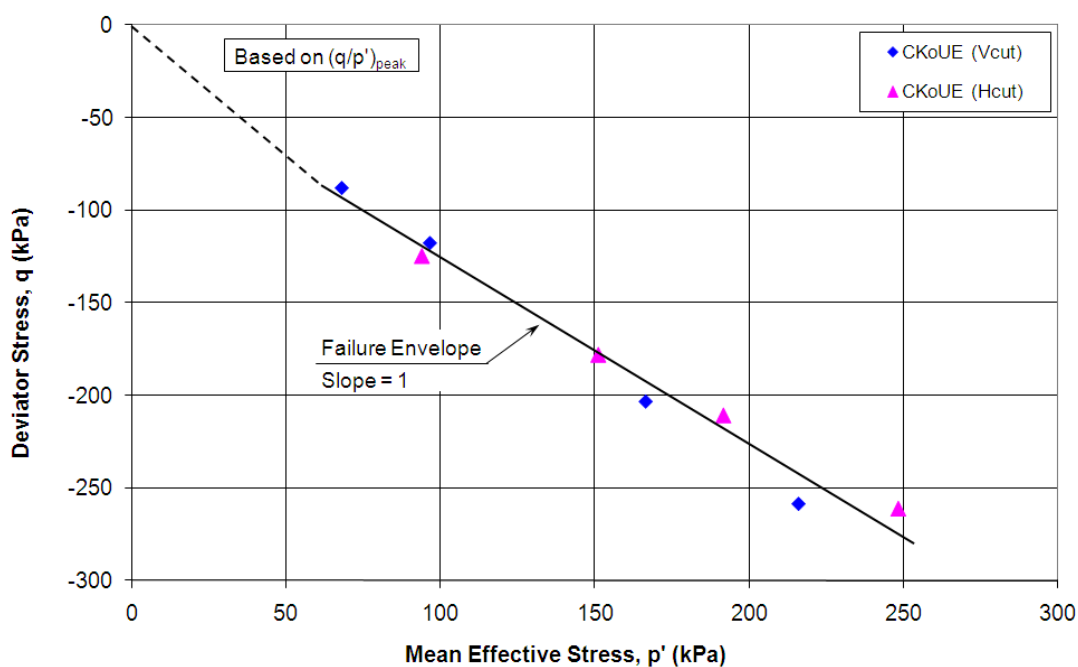


Figure 5.22 (b)

Figure 5.22 Comparison of Failure Envelopes Determined from CK_0UE Tests: (a) From Stress Paths; (b) From $(q/p)_{peak}$

Figure 5.23 presents the comparison of c_u/σ'_{10} versus OCR relationships. There are some differences. However, as the data at ORC =1 and ORC =4 are fairly close, we may assume they follow the same relationship as shown in Figure 5.23. This is consistent with the results obtained from CK₀UC tests. The reasons for the small differences in the undrained shear strength behaviour between vertically cut and horizontally cut specimens are possibly: (1) the use of vertically and horizontally cut specimens is not a good testing method to study the anisotropic strength behaviour, (2) the samples at in-situ were probably consolidated under a stress condition close to the isotropic state as implied by the oedometer tests on vertically and horizontally cut specimens. Therefore, the inherent anisotropy is not pronounced.

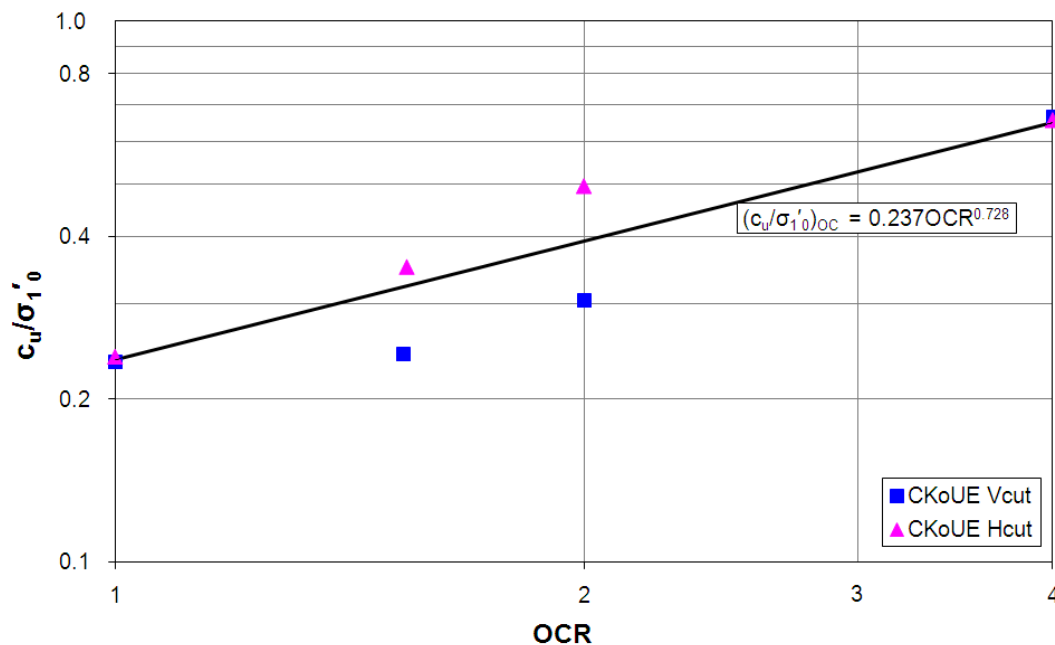


Figure 5.23 Comparison of c_u/σ'_{10} vs. OCR Relationships Obtained from CK₀UE Tests

The A_f vs. OCR relationships obtained from CK₀UE tests are compared in Figure 5.24. It is noted that for vertically cut intact residual soil, the A_f value just reduced a little bit from NC to OCR2 state. It dropped significantly afterwards. In contrast, the A_f value obtained from tests on horizontally cut specimens decreased gradually with

OCR, which was consistent with the observations from CK_0UC tests on the intact residual soil specimens. Simons' data from CIU tests on different clays are plotted for reference. It is important to point out that the calculation for A_f are different for compression and extension tests.

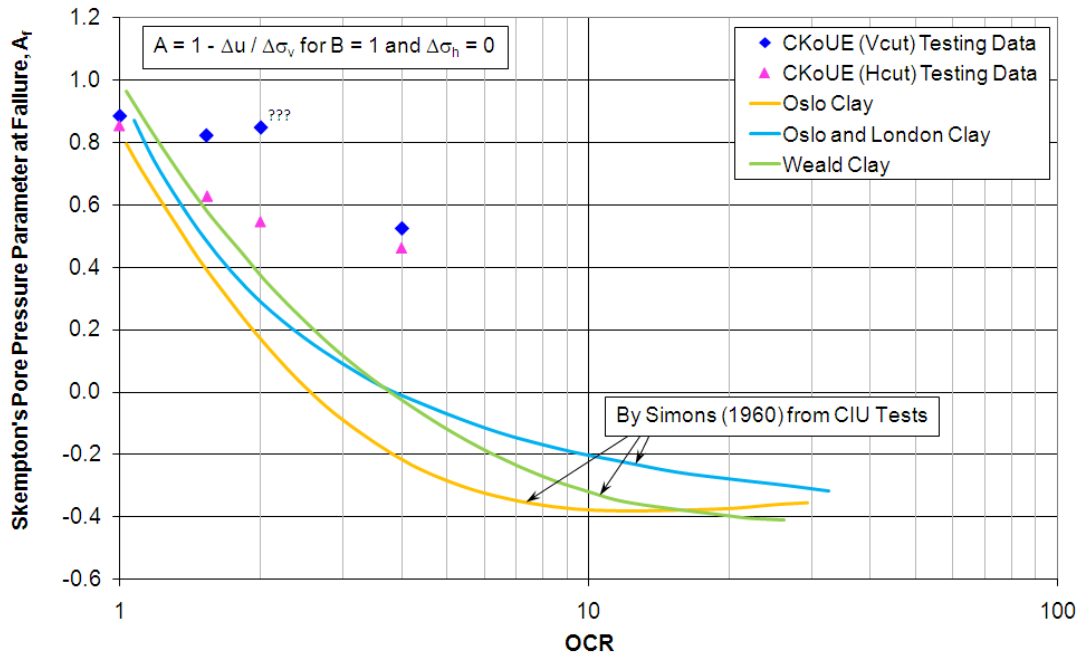


Figure 5.24 Comparison of A_f vs. OCR Relationships Obtained from CK_0UE Tests

5.4 K_0 Consolidated Undrained Direct Simple Shear (CK_0UDSS) Tests

5.4.1 Vertically Cut Specimens

Six CK_0UDSS tests were conducted on vertically cut intact residual soil specimens (as summarized in Table 5.5). The test results are presented in Figure 5.25.

The normally consolidated specimens (V-NC1, V-NC2 and V-NC3) were K_0 consolidated to effective normal stresses of 452kPa, 300kPa and 550kPa respectively. All the over-consolidated (OC) intact residual soil specimens were started to shear under the same effective normal stress of 195 kPa. The shear stress-strain behaviour is compared between NC and OC specimens in Figure 5.25(a). It is clearly shown that the shear resistant increased as the consolidation stress increased

for NC specimen, and increased with OCR for OC specimen. The shear stresses reached the peak at relatively small strain (3%), then followed by a large amount of strain softening (as shown in Figure 5.25(a)).

Table 5.5 Summary of CK_0 UDSS Tests on Vertically Cut Intact Residual Soil Specimens

Test No.	V-NC1	V-NC2	V-NC3	V-OCR1.3	V-OCR2	V-OCR4
OCR	1	1	1	1.54	2	4
Shearing Rate (mm/min)	0.03	0.054	0.054	0.021	0.021	0.021
σ_{n0} (kPa)	452.05	300.61	549.80	194.80	195.32	195.58
τ_{max} (kPa)	102.12	75.87	131.22	51.45	80.03	145.51
Shear Strain at τ_{max} (%)	3.74	3.73	4.17	3.17	3.11	3.33
σ_n at τ_{max} (kPa)	225.73	146.78	277.49	102.82	195.32	303.68
$(\tau/\sigma_n)_{max}$	0.54	0.65	0.60	0.61	0.55	0.51
Secant Friction Angle, ϕ' (°)	28.37	33.02	30.96	31.38	28.81	27.20

Figure 5.25(b) shows the response of normal stress during undrained shearing. According to Chu et al. (1999) and Chu et al. (2003), the change in the normal stress can be considered as the change in pore water pressure. It is noted that the normal stress for OC specimens (V-OCR2 and V-OCR4) increased first then followed by a reduction, which consistent with the dilation and compression behaviour of OC clay. For NC specimens (V-NC1, V-NC2 and V-NC3), the normal stress reduced due to the compression during undrained shearing.

The failure envelope determined from CK_0 UDSS tests on vertically cut intact residual soil is curved within the tested stress range (BC in Figure 5.25(c)). Curved envelopes have been observed for many other types of clay (Lo and Morin, 1972; Burland et al., 1996; Georgiannou & Burland, 2001; Asghari et al., 2003; Barker, 2004; Lo & Hinchberger, 2007; Tiwari, 2007) although mainly for triaxial tests. The failure envelope in the low stress range is assumed to be straight and passing

through the origin, as shown by dotted curve AB in Figure 5.25(c), i.e. it is assumed the true cohesion of the intact residual soil is small enough to be ignored.

The variations of stress ratio during CK_0UDSS tests are presented in Figure 5.25(d). For OC specimens, the stress ratio increased as OCR decreased.

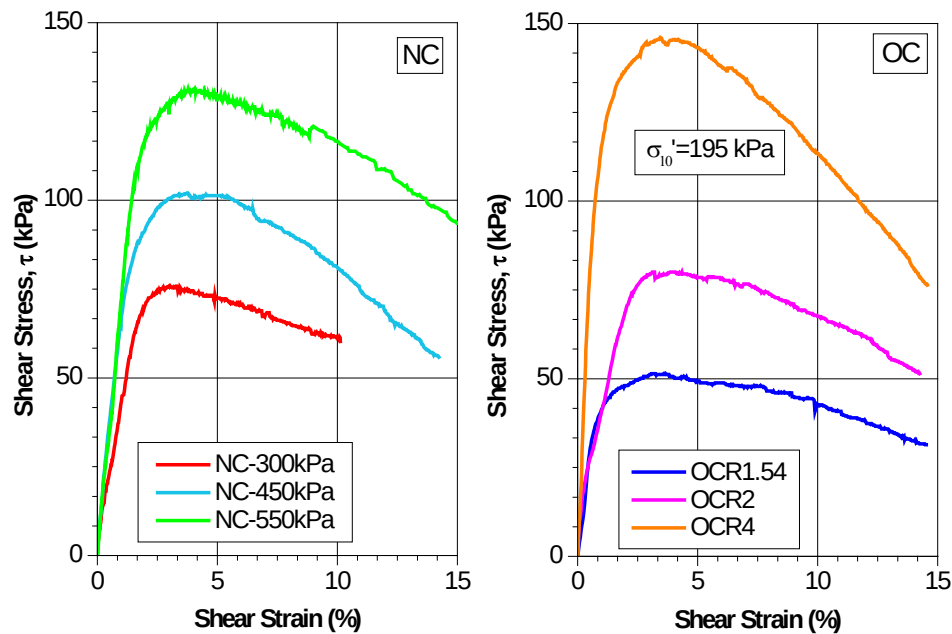


Figure 5.25 (a)

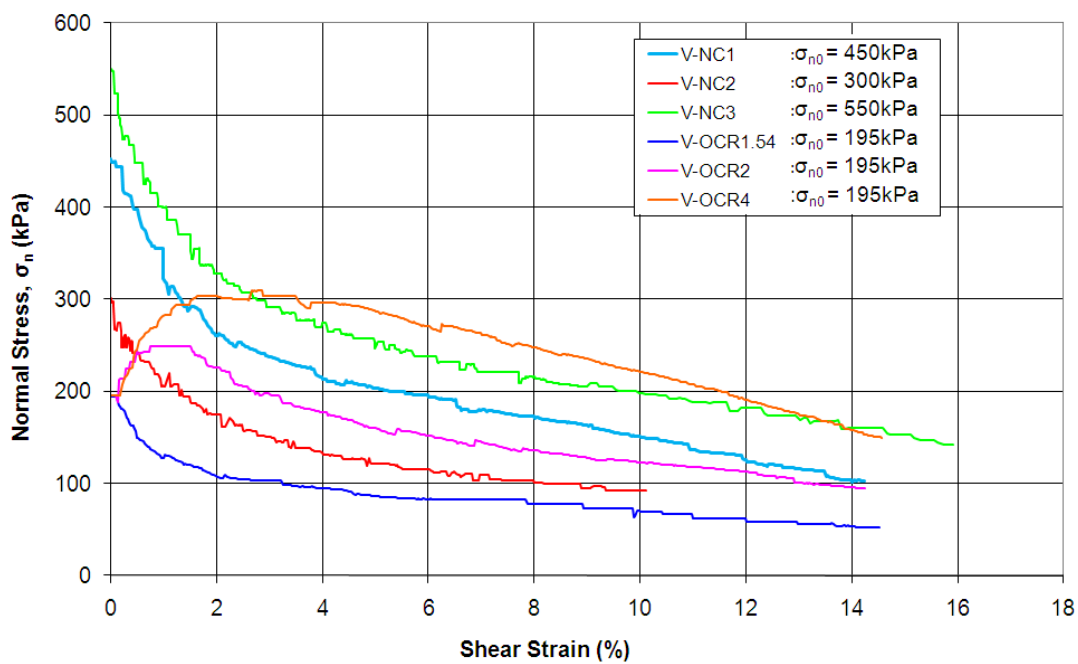


Figure 5.25 (b)

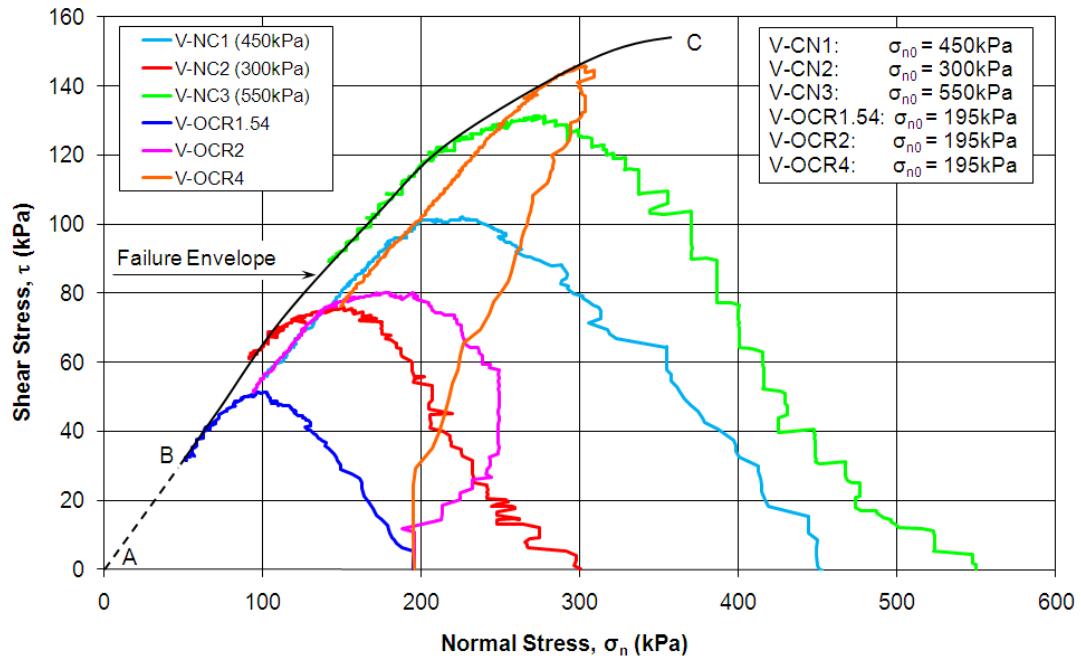


Figure 5.25 (c)

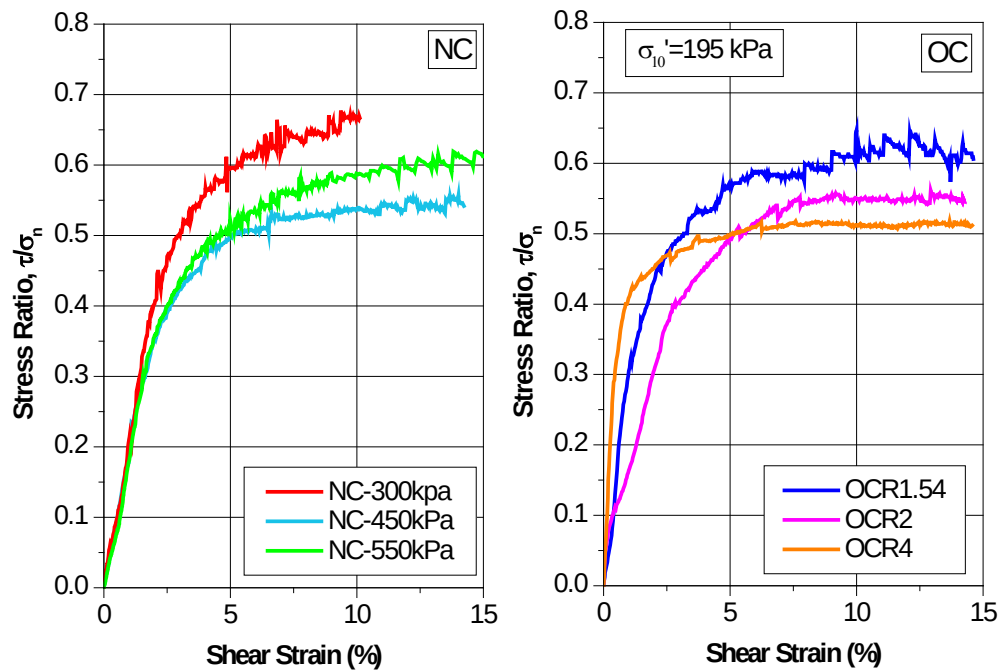


Figure 5.25 (d)

Figure 5.25 Undrained Behaviour of Vertically Cut Intact Residual Soil from CK_0UDSS Tests: (a) Shear Stress-Strain Curves; (b) Normal Stress vs. Shear Strain Curves; (c) Shear Stress vs. Normal Stress Curves; (d) Variations of Stress Ratio with Shear Strain

To compare the tests conducted under different consolidation stresses, the CK_0UDSS test results are normalized against the initial normal stress (σ_{n0}) in Figure 5.26. From Figure 5.26(a), it is observed that the normalized shear strength increased as OCR increased. It is also noted that all the NC specimens tended to reach the same peak value of normalized shear stress, i.e. a constant τ/σ_{n0} ratio does exist for NC intact residual soil specimen. A critical state line (CSL) with a slope of 0.6 was determined based on the tests conducted on NC specimens, as shown in Figure 5.26(b).

The τ/σ_{n0} vs. OCR relationship determined from CK_0UDSS tests on vertically cut intact residual soil specimens is presented in Figure 5.27. The test results fitted into the relationship precisely. An average value of 0.239 is used for $(\tau/\sigma_{n0})_{NC}$. The equation determined from the test results was shown in the plot.

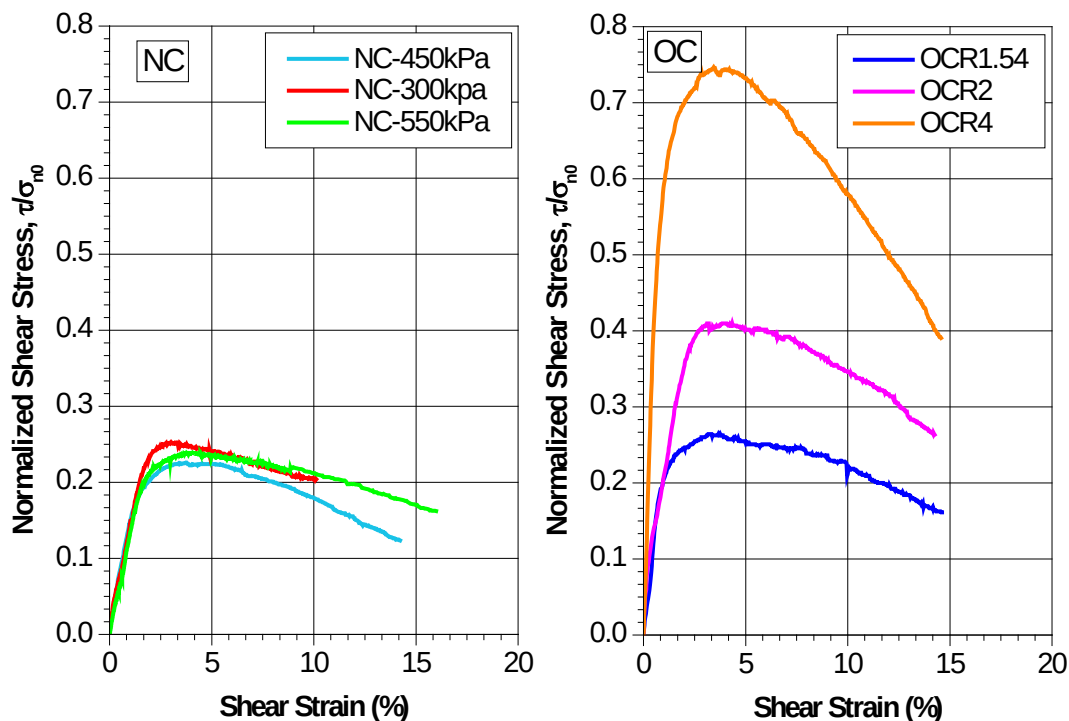


Figure 5.26 (a)

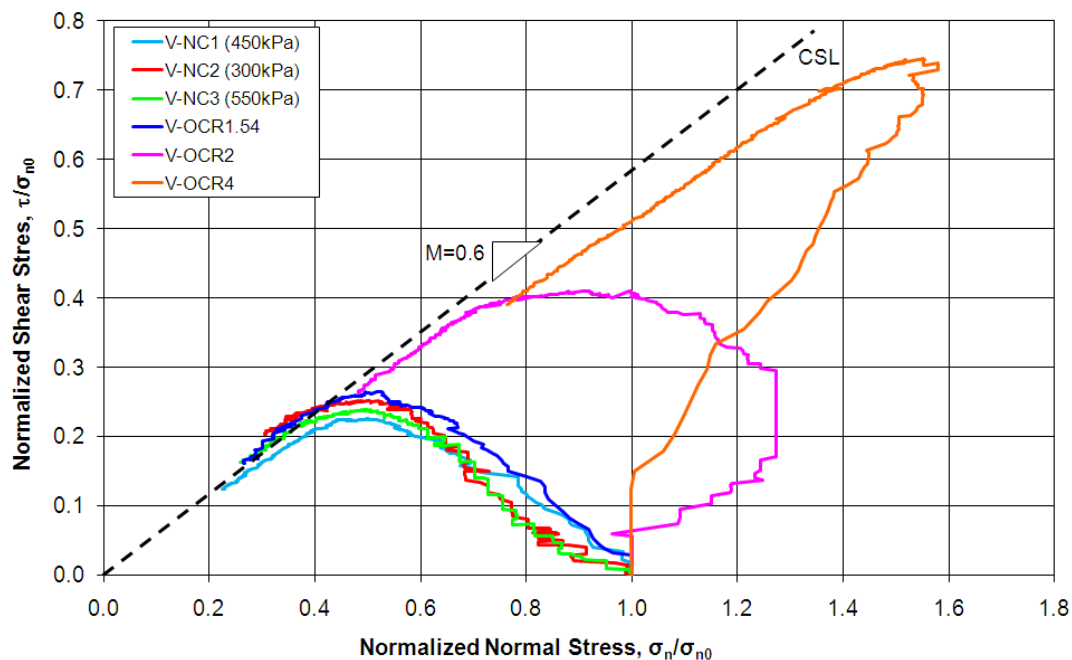


Figure 5.26 (b)

Figure 5.26 Normalized Undrained Behaviour of Vertically Cut Intact Residual Soil from CK_0UDSS Tests: (a) Normalized Stress-Strain Curves; (b) Normalized Shear Stress vs. Normal Stress

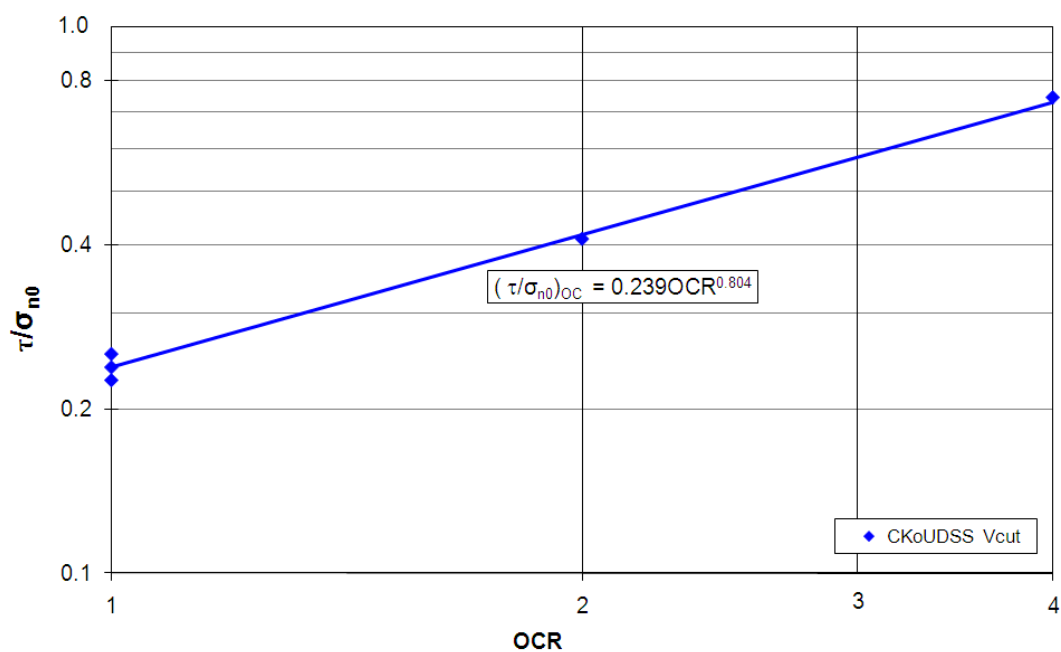


Figure 5.27 τ/σ_{n0} vs. OCR Relationship Obtained from CK_0UDSS Tests on Vertically Cut Intact Residual Soil Specimens

5.4.2 Horizontally Cut Specimens

Table 5.6 summarized the CK_0 UDSS tests conducted on horizontally cut intact residual soil specimens. The undrained behaviour obtained from those tests was presented in Figure 5.28.

Table 5.6 Summary of CK_0 UDSS Tests on Horizontally Cut Intact Residual Soil Specimens

Test No.	H-NC	H-OCR2	H-OCR4	H-195
OCR	1	2	4	1.54
Shearing Rate (mm/min)	0.03	0.03	0.03	0.03
σ_{n0} (kPa)	389.69	194.80	194.80	195.84
$\tau_{\max}$ (kPa)	83.15	84.19	157.21	52.23
Shear Strain at $\tau_{\max}$ (%)	2.29	3.70	3.50	3.06
σ_n at $\tau_{\max}$ (kPa)	212.73	170.90	317.45	100.48
$(\tau/\sigma_n)_{\max}$	0.50	0.58	0.56	0.58
Secant Friction Angle, ϕ' (°)	26.10	30.11	29.10	30.11

Four CK_0 UDSS tests were conducted on horizontally cut intact residual soil specimens, which were anisotropically (K_0) consolidated to different OCRs. The same consolidation method as mentioned before was adopted.

The shear stress-strain behaviour during undrained shearing is shown in Figure 5.28(a). The shear stress reached the peak at about 3% of shear strain, followed by a notable amount of strain softening. The stress ratio approached the peak after the peak shear stress, at about a shear strain of 8% (as shown in Figure 5.28(d)). The response of normal stress during undrained shearing is plotted in Figure 5.28(b). A typical OC behaviour was observed for specimen H-OCR2 and H-OCR4, which was similar to the observation made for the vertically cut intact residual soil specimens. The stress paths reached a linear failure envelope (AC) as shown in Figure 5.28(c). The friction angle determined from the failure envelope is 29.2. The failure envelope was determined from the peak stress ratio, $(\tau/\sigma_n)_{\text{peak}}$.

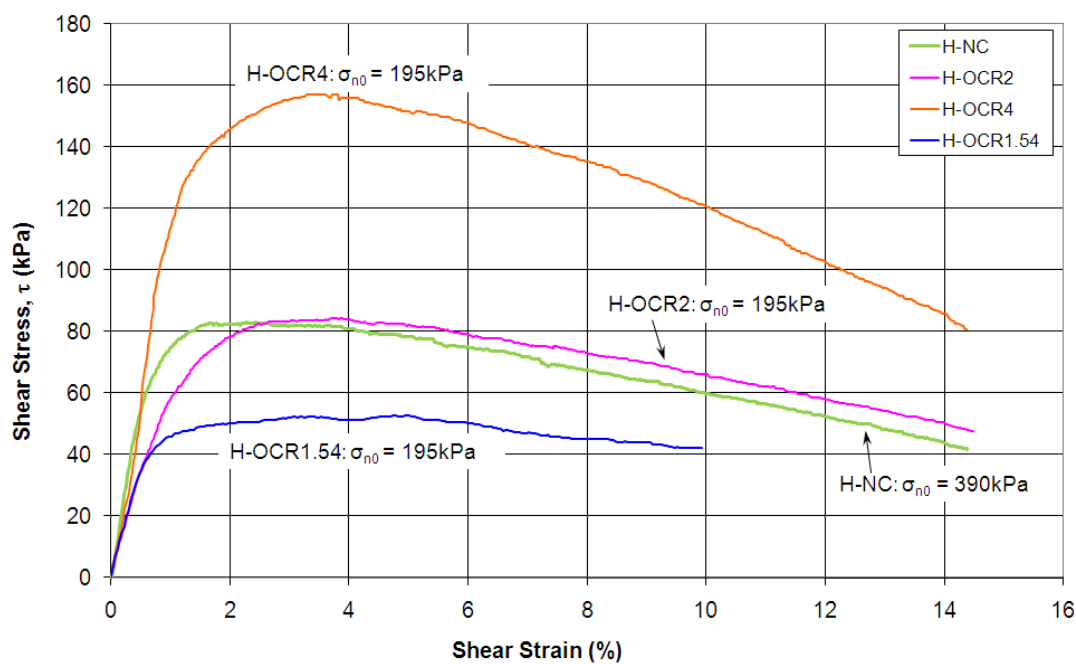


Figure 5.28 (a)

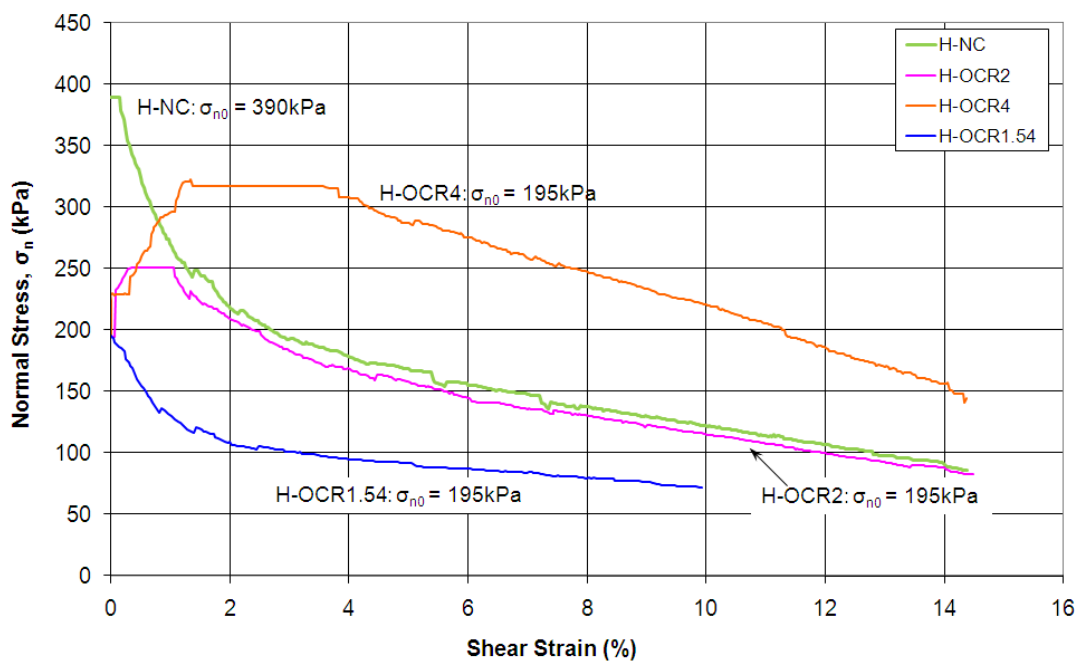


Figure 5.28 (b)

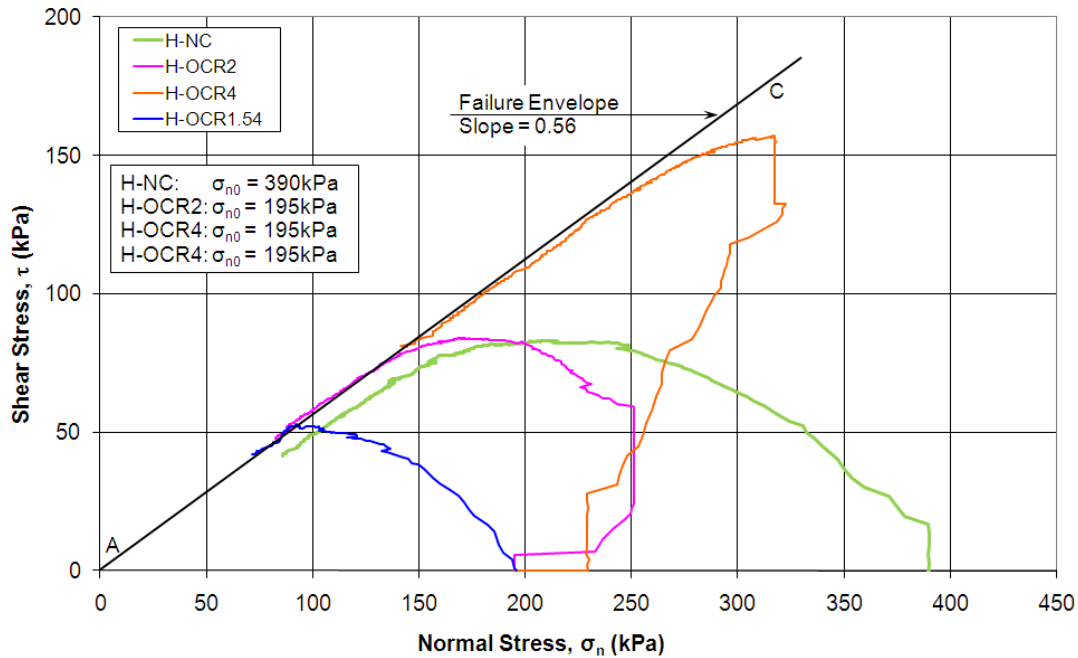


Figure 5.28 (c)

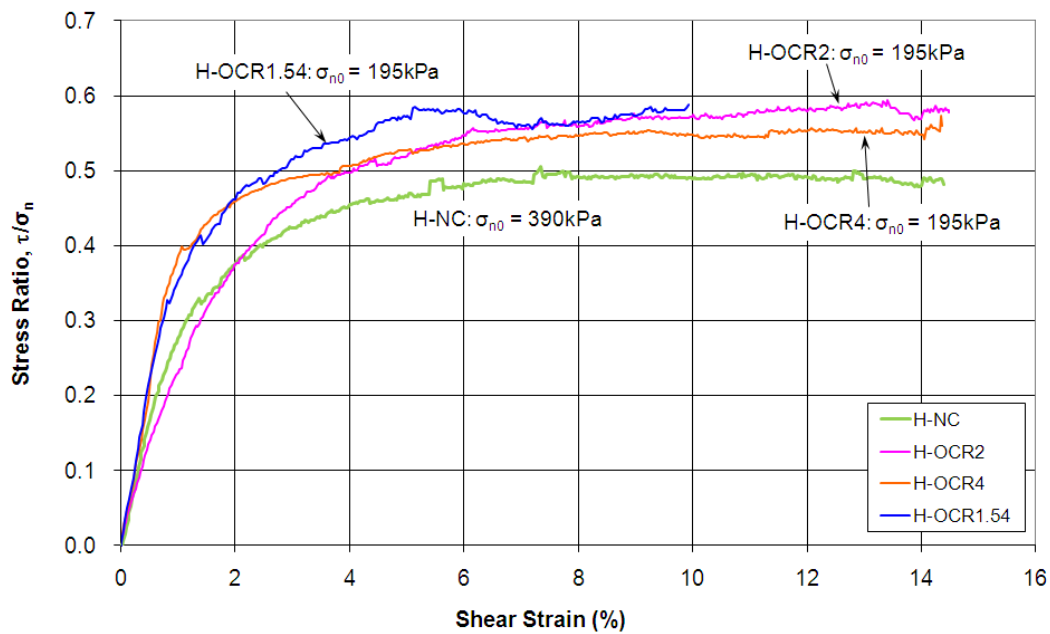


Figure 5.28 (d)

Figure 5.28 Undrained Behaviour of Horizontally Cut Intact Residual Soil from CK_0UDSS Tests: (a) Shear Stress vs. Normal Stress Curves; (b) Shear Stress-Strain Curves; (c) Normal Stress vs. Shear Strain Curves; (d) Variations of Stress Ratio with Shear Strain

A set of normalized undrained test results are presented in Figure 5.29. Same as the results of vertically cut intact residual soil specimens, the initial normal stress (σ_{n0}) was chosen as a normalized parameter. A critical state line (CSL) with a slope of 0.5 was determined, as presented in Figure 5.29(a). It is observed from Figure 5.29(b) that the normalized shear strength increased as OCR increased, which was consistent with the results of vertically cut specimens.

The τ/σ_{n0} vs. OCR relationship determined from CK_0UDSS tests on vertically cut intact residual soil specimens is presented in Figure 5.30. The test results were highly consistent with the power relationship. A value of 0.213 was taken as $(\tau/\sigma_{n0})_{NC}$. The equation determined from the test results was shown in the plot.

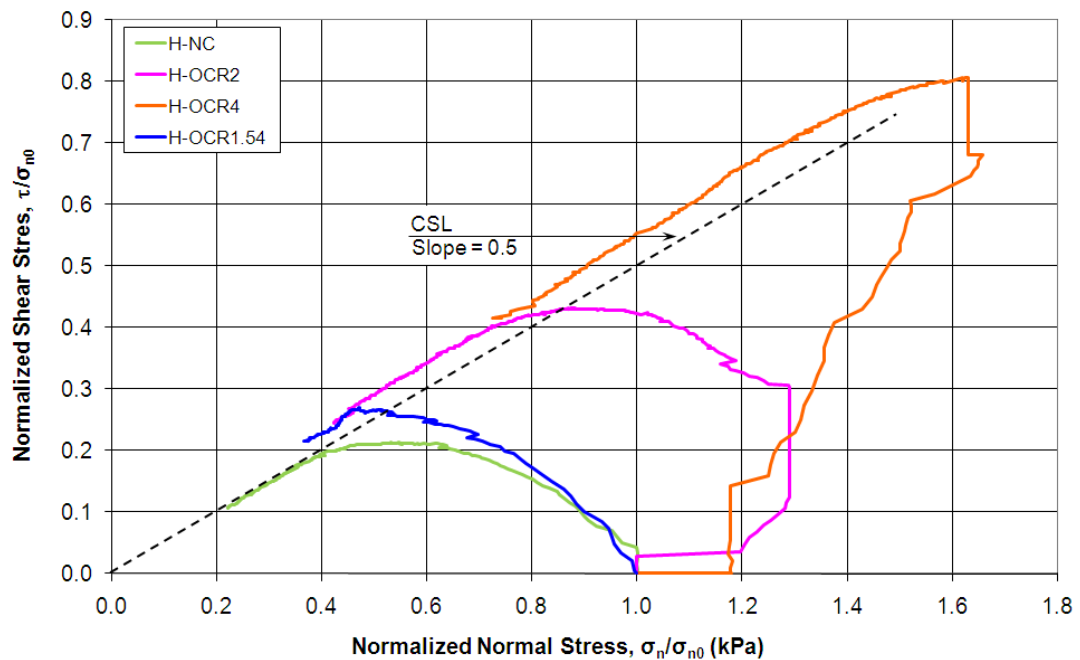


Figure 5.29 (a)

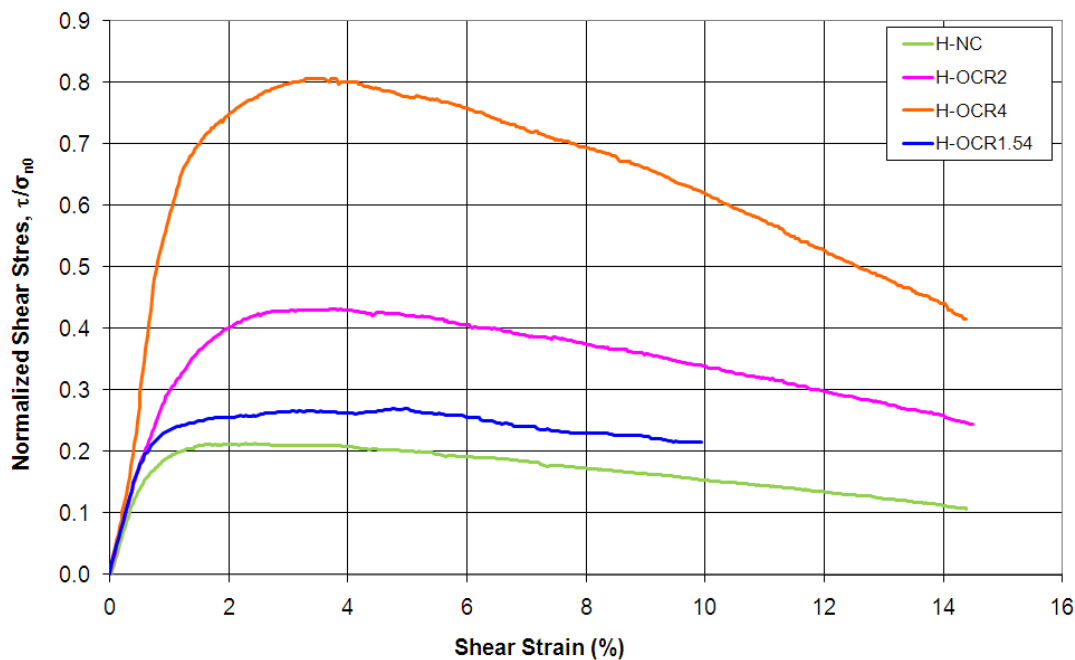


Figure 5.29 (b)

Figure 5.29 Normalized Undrained Behaviour of Horizontally Cut Intact Residual Soil from CK_0 UDSS Tests: (a) Normalized Shear Stress versus Normal Stress; (b) Normalized Stress-Strain Curves

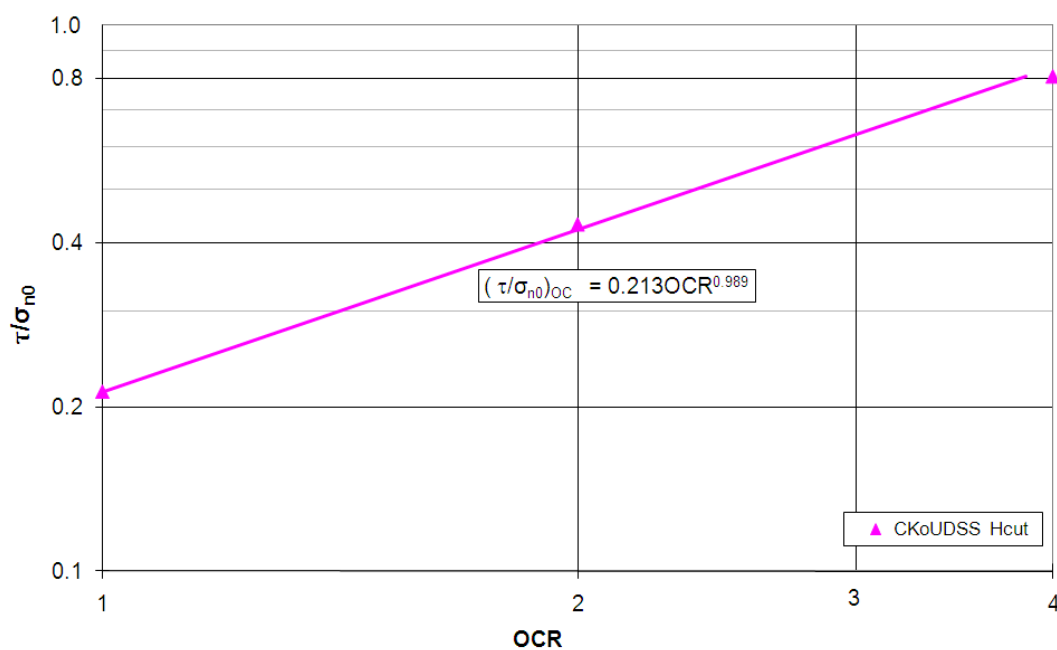


Figure 5.30 τ/σ_{n0} vs. OCR Relationship Obtained from CK_0 UDSS Tests on Horizontally Cut Intact Residual Soil Specimens

5.4.3 Comparison

The normalized shear stress-strain behaviour is compared in Figure 5.31. It is noted that the horizontally cut intact residual soil reached a higher peak than vertically cut ones, especially for highly over-consolidated specimens. However, the difference is not as large as observed in CK_0UE tests.

By comparing the effective stress paths of vertically cut and horizontally cut specimens in Figure 5.32(a), a unique failure envelope with a slope of 0.57 was determined under direct simple shear conditions. The observation can be seen with ease from the locations of failure points (as shown in Figure 5.32(b)). It is evident that the failure envelope of the intact residual soil was independent of the specimen orientation. Similar observations were made from CK_0UC and CK_0UE tests.

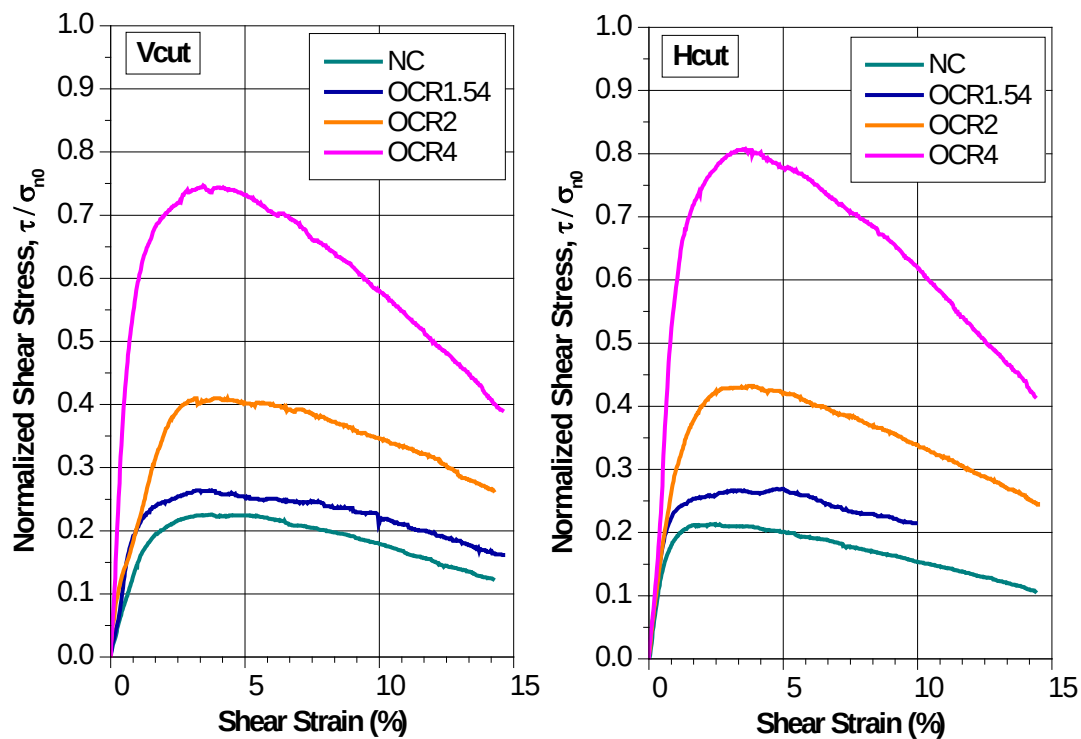


Figure 5.31 Comparison of Normalized Shear Stress-Strain Behaviour Determined from CK_0UDSS Tests

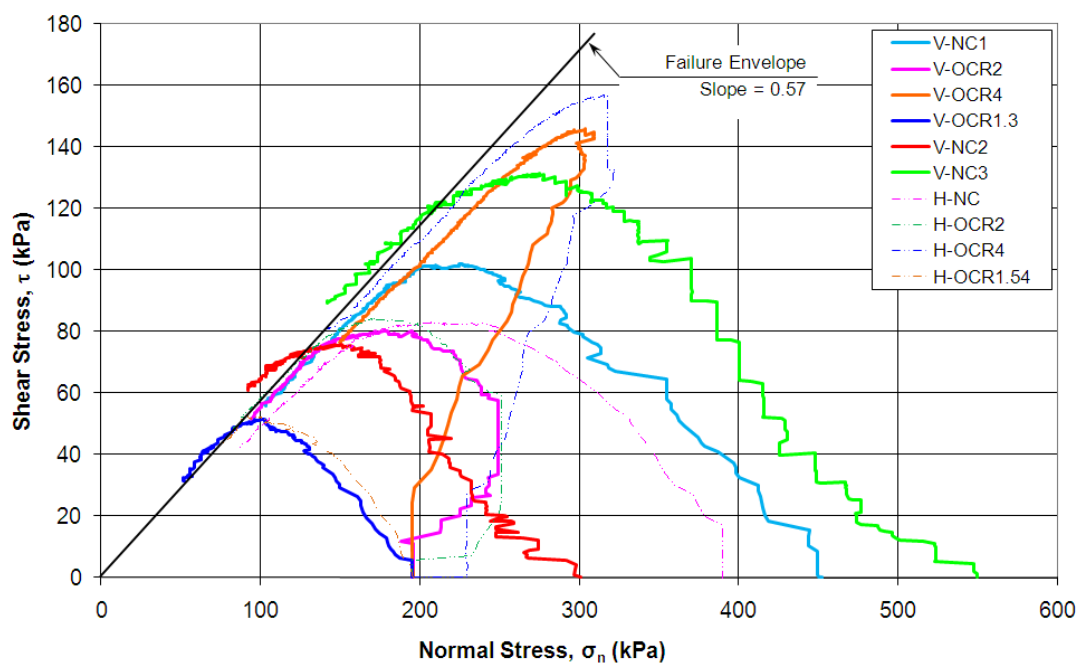


Figure 5.32 (a)

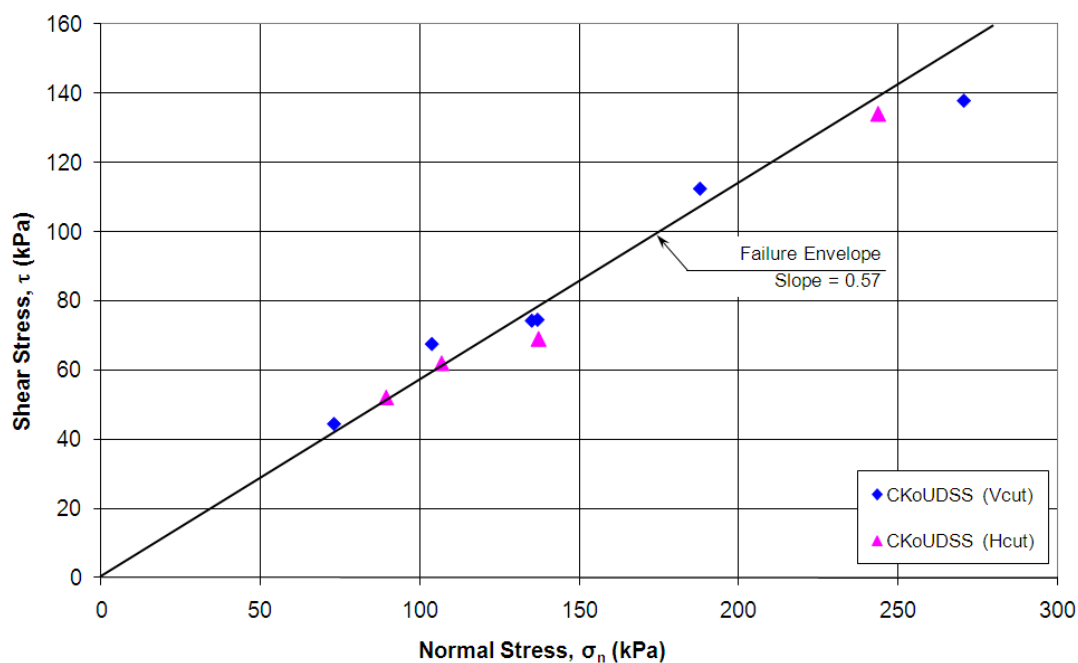


Figure 5.32 (b)

Figure 5.32 Comparison of Failure Envelopes Determined from CK₀UDSS Tests: (a) from Stress Paths; (b) from Failure Points

Figure 5.33 presents the τ/σ'_{n0} versus OCR relationships determined from CK₀UDSS tests. The data obtained from tests on vertically cut and horizontally cut intact residual soil specimens were highly consistent. A unique power relationship was drawn in Figure 5.33. An average $(\tau/\sigma'_{n0})_{NC}$ value of 0.233 was adopted.

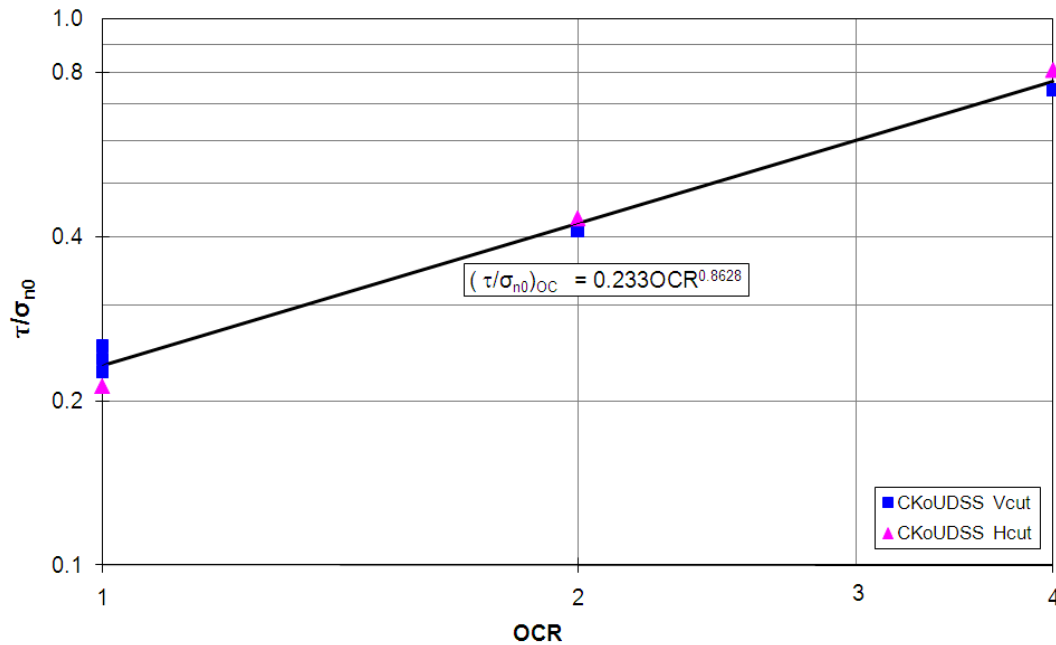


Figure 5.33 Comparison of τ/σ'_{n0} vs. OCR Relationships Obtained from CK₀UDSS Tests

5.5 K₀ Consolidated Undrained Plane-Strain (CK₀UPS) Tests

A summary of five K_0 consolidated undrained plane-strain (CK₀UPS) tests conducted on the intact residual soil specimens is given in Table 5.7, and the test results are presented in Figure 5.34(a) to (d). Due to the quantity and quality of the intact residual soil sample, only vertically cut specimens were tested using plane-strain apparatus.

Figure 5.34(a) presents the effective stress paths obtained from the above five tests. It can be seen that all the effective stress paths reached the same straight failure envelope (solid line BC in Figure 5.34(a)). The failure envelope in the low effective confining stress range is assumed to be straight and passing through the origin, as shown by dotted line AB in Figure 5.34(a).

Table 5.7 Summary of CK_0 UPS Tests Conducted on Vertically Cut Intact Residual Soil Specimens

Test No.		PSGN3	PSGN4	PSGN5	PSGN6	PSGN7
Pressure for Saturation (kPa)	Cell	860	860	860	860	860
	Back	850	850	850	850	850
OCR		3.1	1	1	2	1.46
Shearing Rate (mm/min)		0.01	0.01	0.01	0.01	0.01
Weight (g)	Pre-Test	792.50	802.74	770.76	801.47	813.64
	Post-Test	805.23	794.67	764.52	795.44	830.35
Water Content (Pre-Test) (%)	Average	31.32	31.14	30.12	30.28	29.47
Water Content (Post-Test) (%)	Average	29.52	30.54	31.35	31.63	32.36
	Top	28.82	30.00	31.47	31.22	32.70
	Middle	29.42	30.92	31.48	32.37	32.16
	Bottom	30.34	30.70	31.11	31.17	32.31
σ'_{10} (kPa)		280.22	452.61	449.13	261.89	205.06
q_0 (kPa)		4.73	250.60	238.40	60.80	129.81
p'_0 (kPa)		262.25	286.05	290.24	221.54	118.53
$q_{\max}$ (kPa)		408.01	269.31	256.49	282.76	159.96
ε_1 at q_{peak} (%)		1.95	0.86	0.89	0.78	1.15
$(q/p')_{\text{peak}}$		1.26	1.29	1.23	1.20	1.42
$R_{\max} = (\sigma'_1/\sigma'_3)_{\max}$		4.15	4.57	4.40	4.20	5.03
ε_1 at $(q/p')_{\text{peak}}$ (%)		4.17	5.96	4.71	3.98	2.00
Secant Friction Angle, ϕ' (°)		37.71	39.86	39.02	37.98	41.90

Similar to the triaxial tests, the peak deviator stresses of CK_0 UPS tests were attained at small axial strains (about 1% in Figure 5.34(b)). As shown in Figure 5.34(c), the value of q/p' reached its maximum value well after the peak deviator stress. There was no notable drop in q/p' after the failure of specimens. The b-values reached the peak at about the same time as q/p' (Figure 5.34(d)). The peak b-values were within the range of 0.2 to 0.3, and they dropped significantly right after the peaks.

The stress-strain curves determined from CK_0 UPS tests were normalized by the initial effective vertical stress, σ'_{10} in Figure 5.34(e). The normalized deviator stress reached the peak value at about 1% of axial strain. It increased as OCR

increased. The normalized stress-strain curves for NC specimens were close to each other. The consequent c_u/σ'_{10} versus OCR relationship obtained from CK₀UPS tests is presented in Figure 5.35. All the points fitted into a consistent power relationship. An average value of 0.292 was adopted as $(c_u/\sigma'_{10})_{NC}$ value.

Figure 5.34(f) presents the generation of excess pore water pressure during undrained shearing. For OC specimens (PSGN3, PSGN6 and PSGN7), the excess pore water pressure increased drastically at first and then decreased a little bit. It reached a constant value towards the end of the tests. This is consistent with the compression and dilation behaviour of OC clay.

In addition to above observations, it is also noted that the q/p' , b value and Δu measured in test PSGN7 were very different from the other tests. This is may be because the specimen PSGN7 was anisotropically consolidated to the lowest stress level comparing to others. The natural structure of the intact residual soil was not broke down completely. The undrained behaviour of specimen PSGN7 was greatly affected by the soil bonds. Similar behaviour was observed as in both CK₀UC and CK₀UE tests.

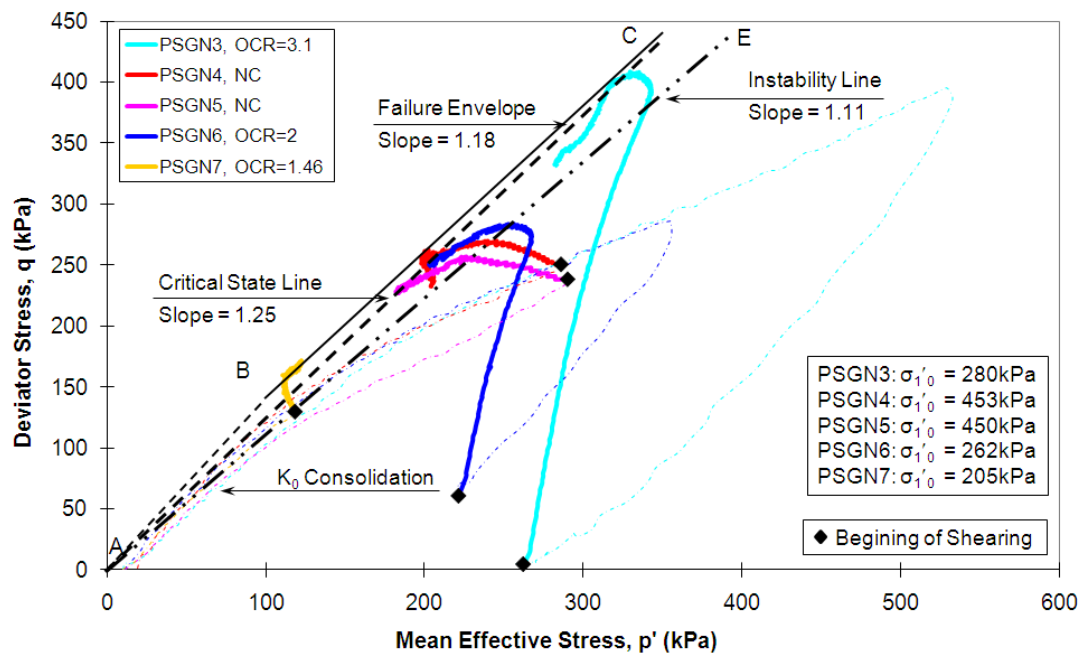


Figure 5.34 (a)

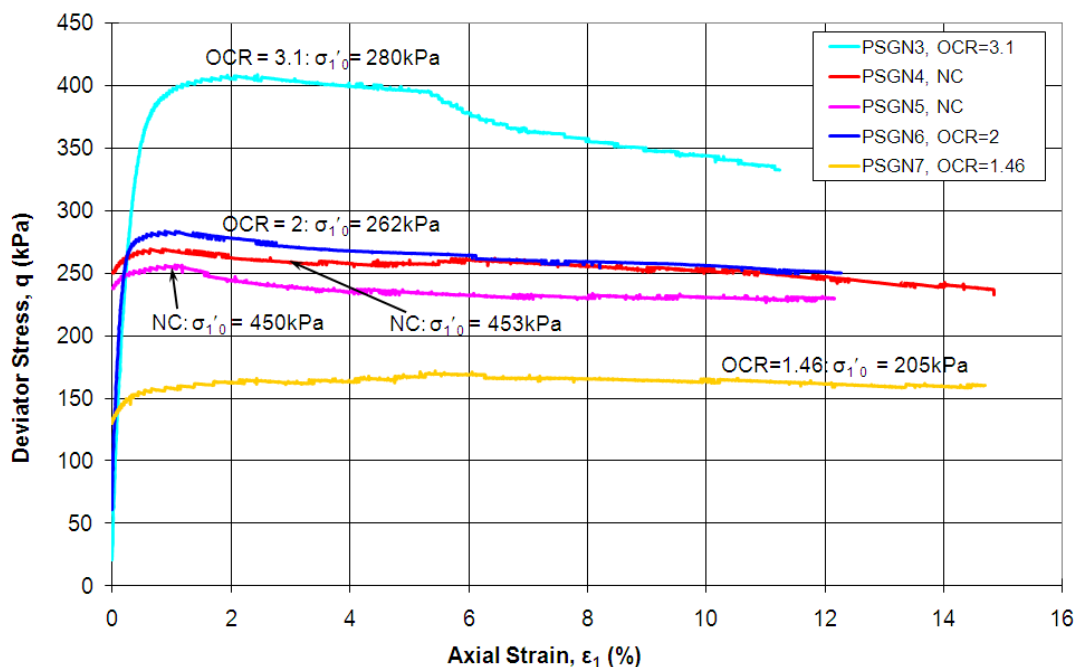


Figure 5.34 (b)

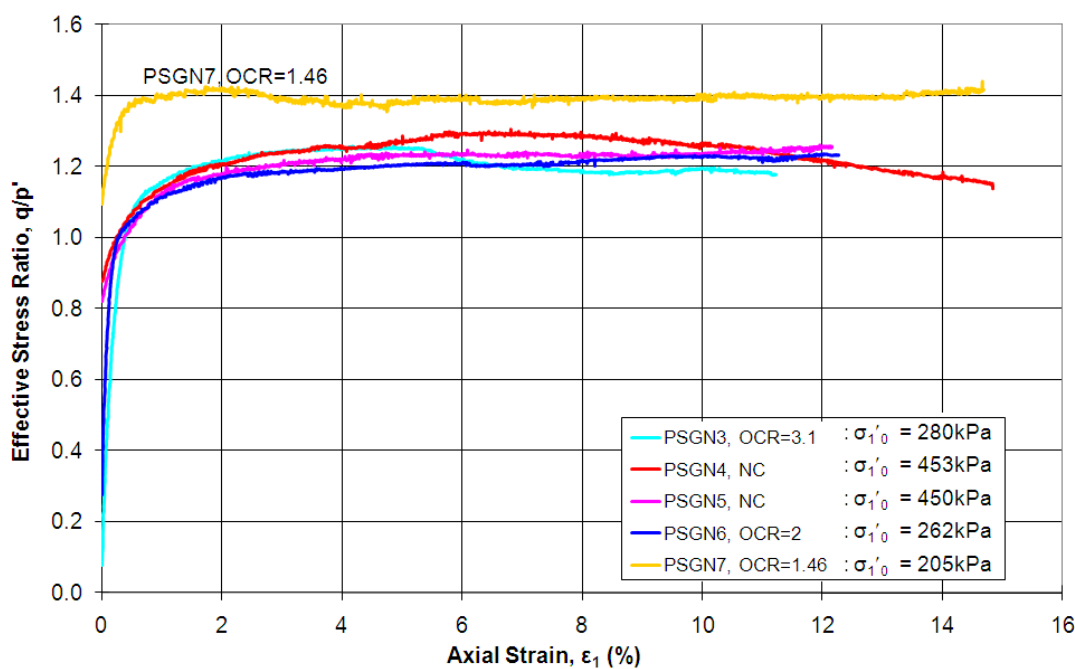


Figure 5.34 (c)

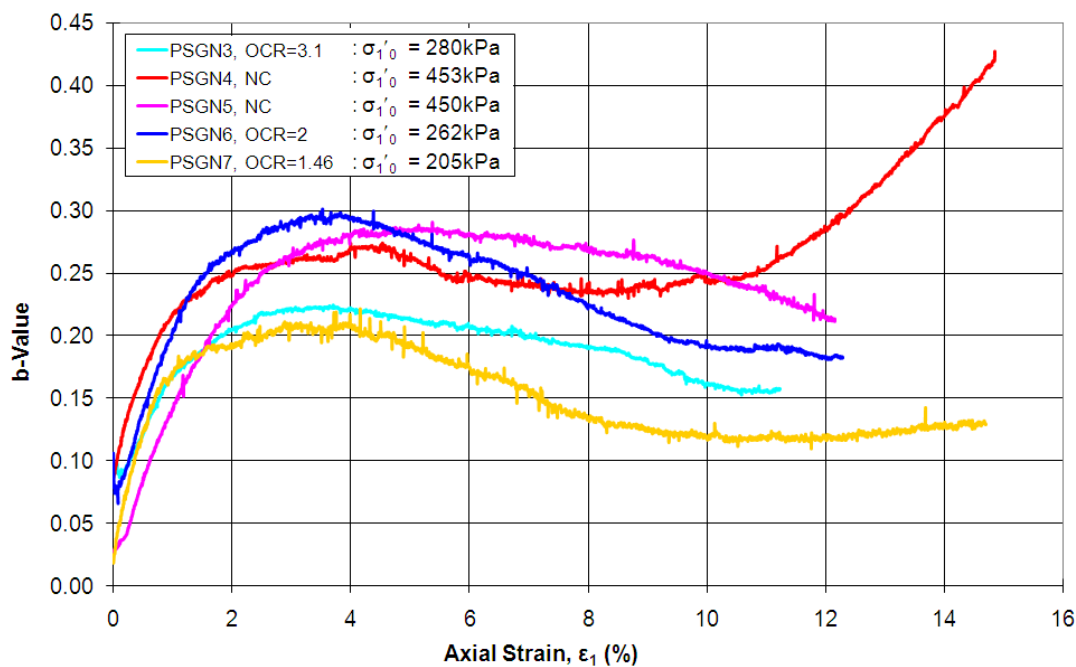


Figure 5.32 (d)

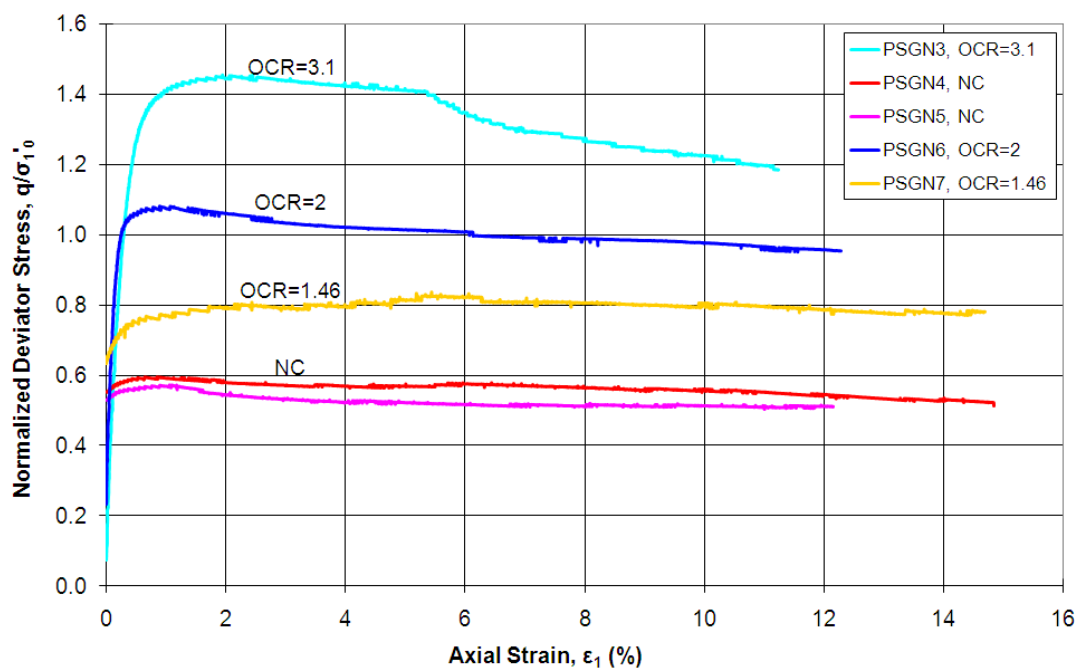


Figure 5.34 (e)

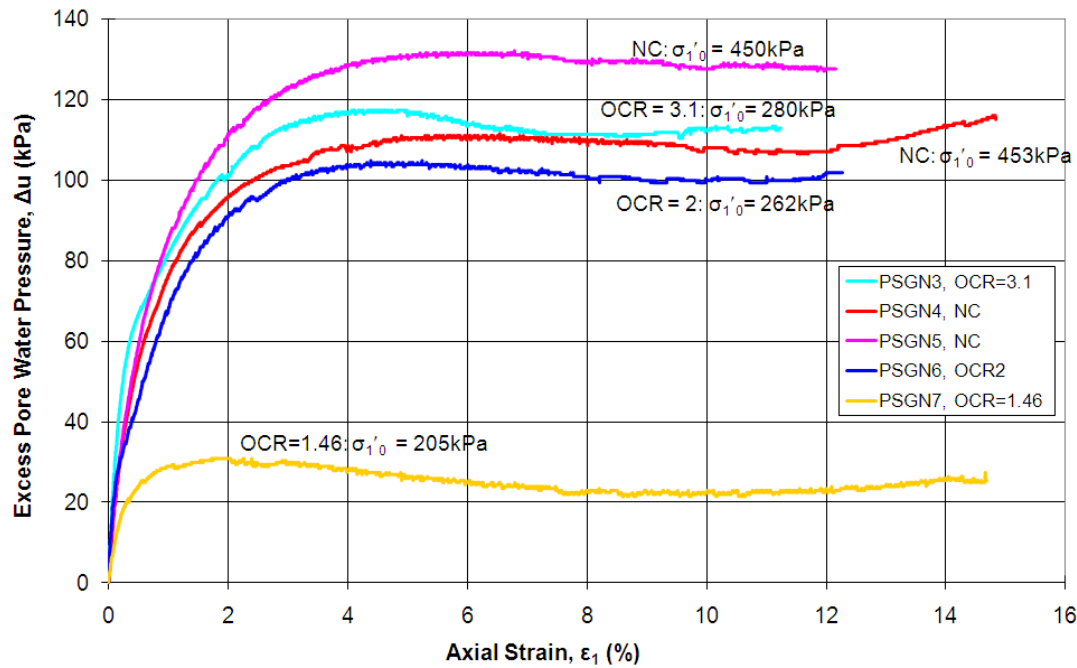


Figure 5.34 (f)

Figure 5.34 Undrained Behaviour of Vertically Cut Intact Residual Soil from CK₀UPS Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) Variations of q/p' ; (d) Variations of b -Value; (e) Normalized Stress-Strain Curves; (f) Variations of Excess Pore Water Pressure

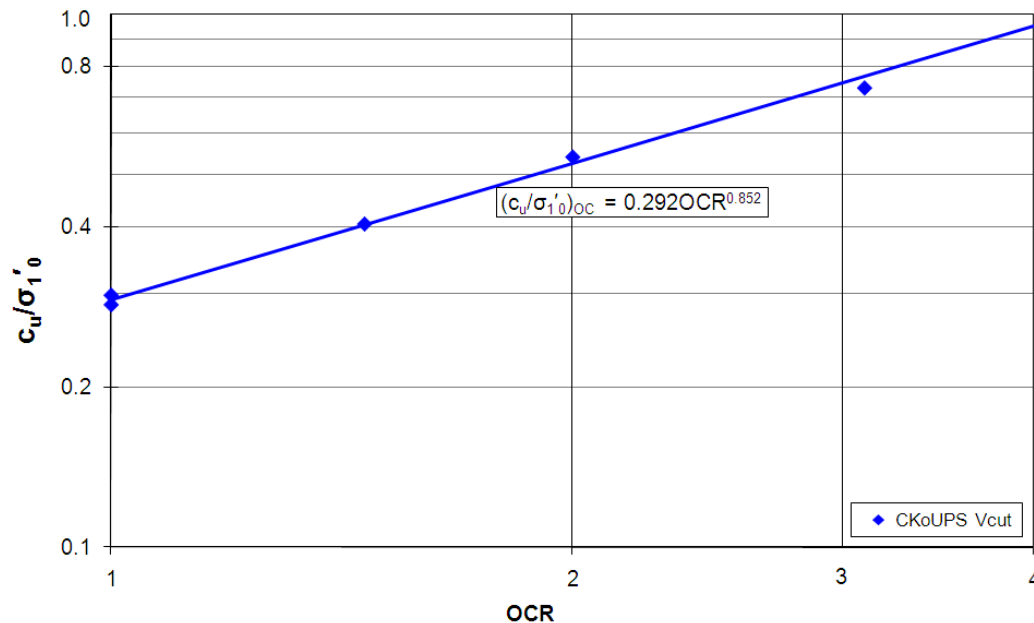


Figure 5.35 c_u/σ'_{10} vs. OCR Relationship Obtained from CK₀UPS Tests on Vertically Cut Intact Residual Soil Specimens

Since the specimens for tests PSGN7, PSGN6 and PSGN3 were over-consolidated along the K_0 line to different effective confining stresses, the effective stress paths presented in Figure 5.37(a) were normalized by $T_0 = M^2 p'_0 + q_0^2 / p'_0$ as used before. M is the slope of critical state line determined from tests on NC specimens. The critical state line is shown in Figure 5.37(a), and $M=1.25$.

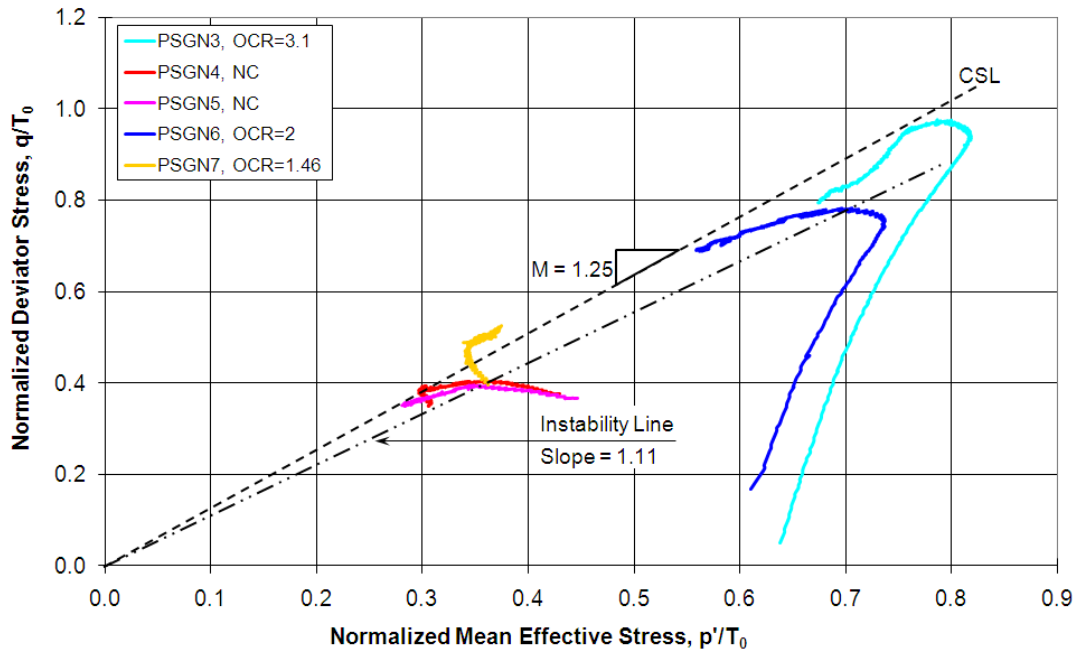


Figure 5.36 Normalized Stress Paths of Vertically Cut Intact Residual Soil from CK_0 UPS Tests

The calculations of Skempton's pore pressure parameter, A , proposed by Skempton (1954) and Law & Holtz (1978) are both under the assumption of axisymmetric conditions, which was not applicable for plane-strain testing conditions. Later on, Henkel (1960) proposed a more general pore pressure equation to take into account the effect of the intermediate principal stress, which was defined as Henkel pore pressure parameter, a . It can be calculated as follow:

$$a = \frac{\Delta u - \Delta \sigma_{oct}}{\Delta \tau_{oct}} \text{ for } B = 1 \quad (5-5)$$

where $\sigma_{oct} = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$

$$\tau_{oct} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$$

Figure 5.37(a) presents the variation of Henkel pore pressure parameter, a , during undrained shearing under plane-strain conditions. For NC specimens, it increased drastically once shearing started. For OC clay, the “ a ” value was very consistent. It tended to reach a constant value towards the failure. The Henkel pore pressure parameter at failure (a_f) was also calculated based on the peak deviator stress for each tests. As seen from Figure 5.37(b), the a_f value decreased drastically with OCR. It approached to a constant value once OCR was beyond 2. Similar observations have been made from CK_0UC and CK_0UE tests. A comparison of pore pressure parameters was made between axisymmetric and plane-strain conditions in the following section.

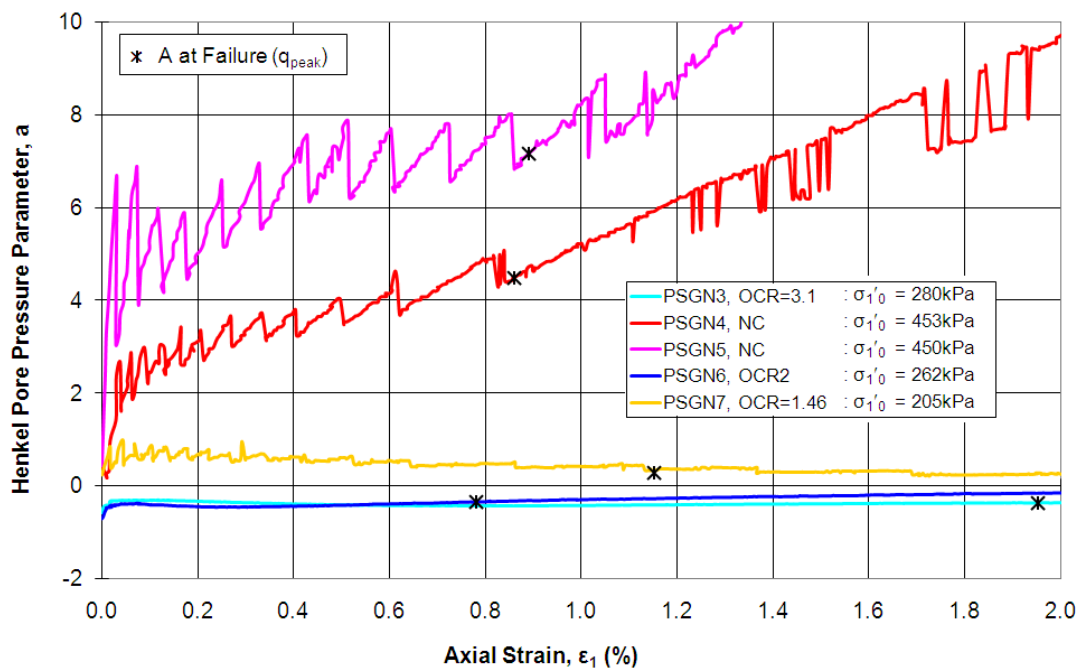


Figure 5.37 (a)

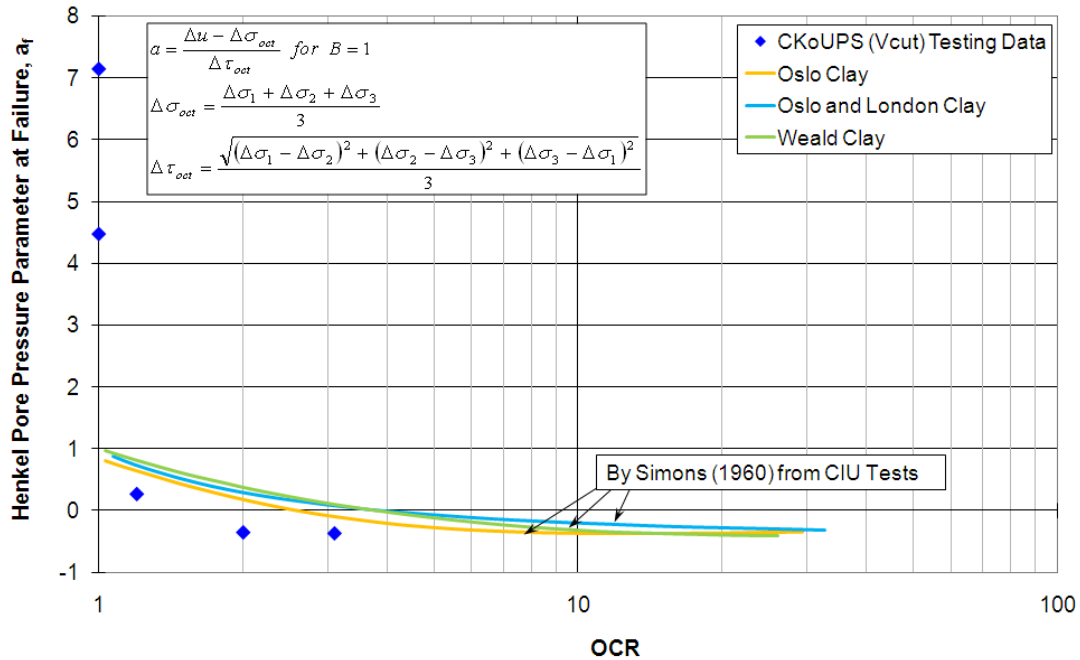


Figure 5.37 (b)

Figure 5.37 Variations of Henkel Pore Pressure Parameter, a in CK_0 UPS Tests: (a) a vs. Axial Strain Curves; (b) a_f vs. OCR Curves

5.6 Discussions

To study the undrained shear strength and stress-strain behaviour of the intact residual soil of Bukit Timah granite, a series of test were conducted under axisymmetric and plane-strain conditions, which included K_0 consolidated undrained triaxial compression (CK_0 UC), extension (CK_0 UE) and direct simple shear (CK_0 UDSS) tests, and plane-strain (CK_0 UPS) tests. The results are compared between different types of tests, and also between the same tests but conducted on the specimens with different cutting orientations. The specimen orientation was introduced in Chapter 3.

As observed earlier in this chapter, the failure envelope was not greatly affected by the specimen orientation. Hence, the comparison of failure envelopes was just between different types of tests, as shown in Figure 5.38. The solid lines represented the failure envelopes within the tested stress range. And the dash lines were assumed failure envelope at low stress level, which passing through the origin.

The failure envelope obtained from CK₀UDSS tests was converted into q vs. p' plot by using the following relationship:

$$\tan \alpha' = \frac{q}{p'} = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (5-6)$$

where ϕ' is the angle of internal friction in terms of effective stress. The derivation of the relationship is given in Appendix I.

The failure envelope determined from CK₀UE tests was plotted in positive scale for comparison purpose. As seen from Figure 5.38, the CK₀UPS tests gave the highest failure envelope, i.e., CK₀UPS tests gave the highest undrained shear strength (c_u) at the same stress level. This is consistent with many other previous studies (Ladd & Lambe, 1963; Ladd et al., 1980; Wanatowski & Chu, 2008; Mayne et al., 2009). The failure envelopes obtained from CK₀UC and CK₀UDSS tests are almost identical. It is indicated that CK₀UDSS test could be a great substitution for CK₀UC test to determine the internal friction angle and failure envelope of the intact residual soil, since it is cheaper, faster and earlier to conduct. It is also observed that the failure envelope resulted from CK₀UE test was the lowest among the four types of tests conducted.

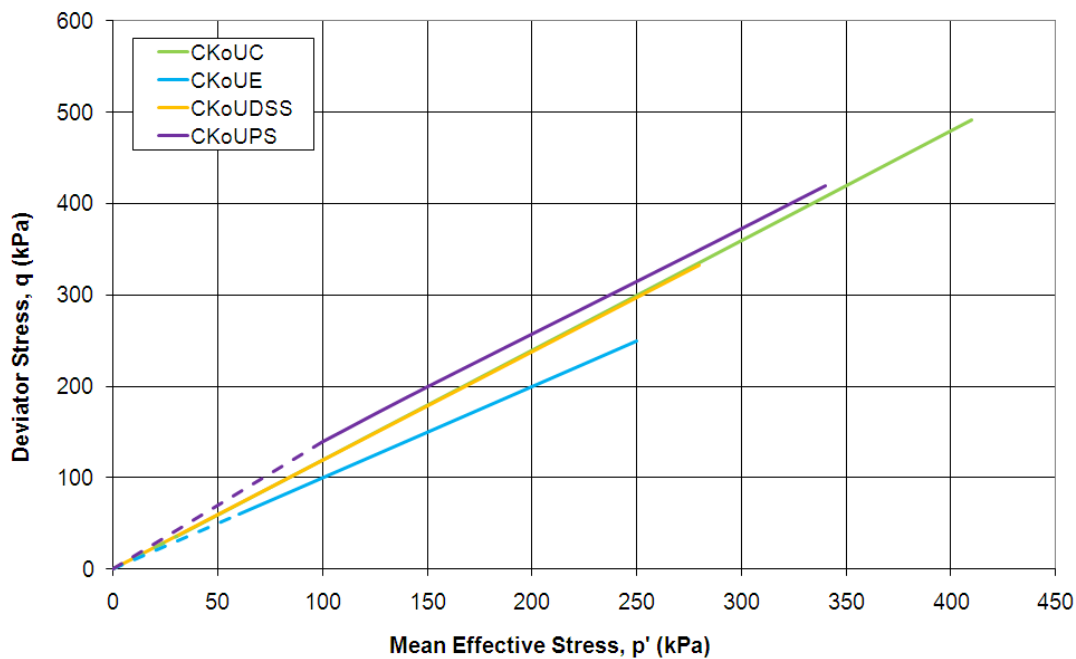


Figure 5.38 Failure Envelopes Determined from Different Types of Tests

The c_u/σ'_{v0} versus OCR relationships obtained from CK₀UC, CK₀UE, CK₀UDSS and CK₀UPS tests are compared in Figure 5.39. As presented earlier in this chapter, the c_u/σ'_{v0} versus OCR relationships were also independent of the specimen orientations. As shown in Figure 5.39, at a given OCR, the c_u/σ'_{v0} ratio obtained from the CK₀UC tests was the highest, followed by that from the CK₀UPS tests. The c_u/σ'_{v0} ratio obtained from the CK₀UE tests was the lowest. The one determined from CK₀UDSS was somewhere in between. This is consistent with the observations made from other soils (Ladd, 1986; Chu et al. 1999).

It is also noted from Figure 5.39 that the value of c_u under extension is much lower than that under compression. Therefore, c_u can be overestimated by CK₀UC or CK₀UPS tests. By comparing the UU test result with the others in Figure 5.39, it is important to point out that the c_u value determined from UU test was even much higher than the one determined from CK₀UC or CK₀UPS tests. It is evident that UU test could significantly overestimate the c_u value of the intact residual soil. The detail of UU test is available in Appendix II.

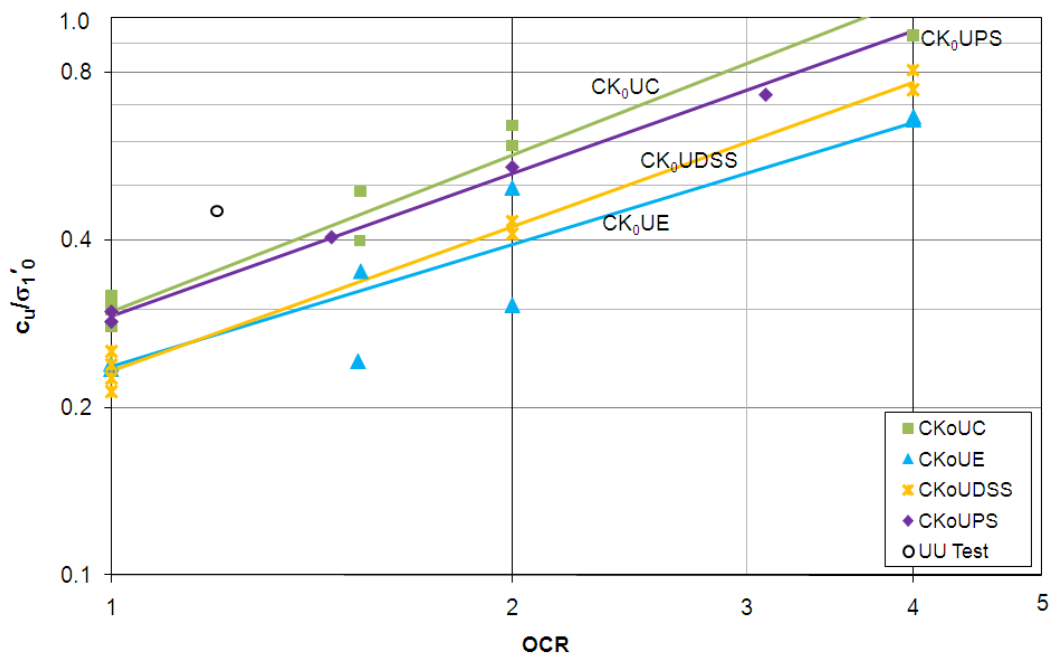


Figure 5.39 c_u/σ'_{v0} vs. OCR Relationships Determined from Different Types of Tests

The c_u of soil is also affected by the rotation of principal stress direction. The effect of principal stress rotation can be evaluated by plotting the ratio of $c_{u/\alpha}/c_{u/CK_0UC}$ versus the angle of major principal stress rotation to the vertical, α . $\alpha = 0^\circ$ represents the normal compression testing condition, with the major principal stress at failure, σ_{1f}' acting vertically. $\alpha = 90^\circ$ represents the major principal stress direction to be horizontal, as in CK_0UE condition. $\alpha = 45^\circ$ is assumed for the principal stress direction in a direct simple shear testing condition for illustration purpose. The results of the three tests, after being normalized with the undrained shear strength for $\alpha = 0^\circ$ are presented in Figure 5.40(a) and (b) for vertically cut and horizontally cut intact residual soil specimens, respectively. It can be seen that the undrained shear strength is significantly affected by rotation of major principal stress, regardless of specimen cutting orientations. This is consistent with previous studies carried out by O'Rourke et al. (1976). It is indicated that the strength anisotropy could be investigated better by conducting tests with different major principal stress directions.

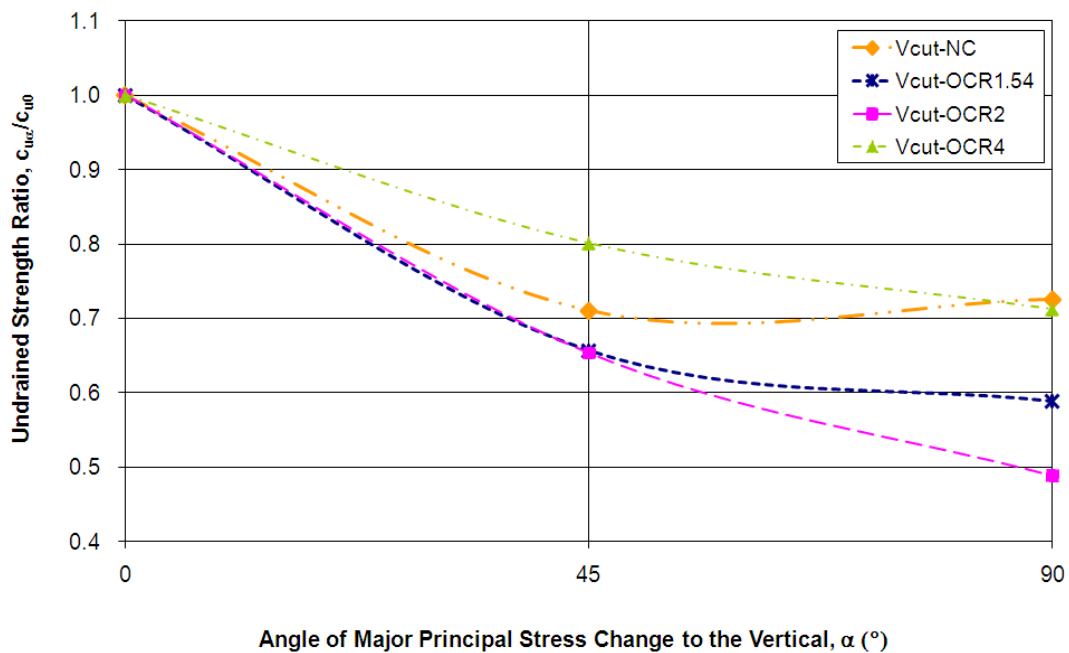


Figure 5.40 (a)

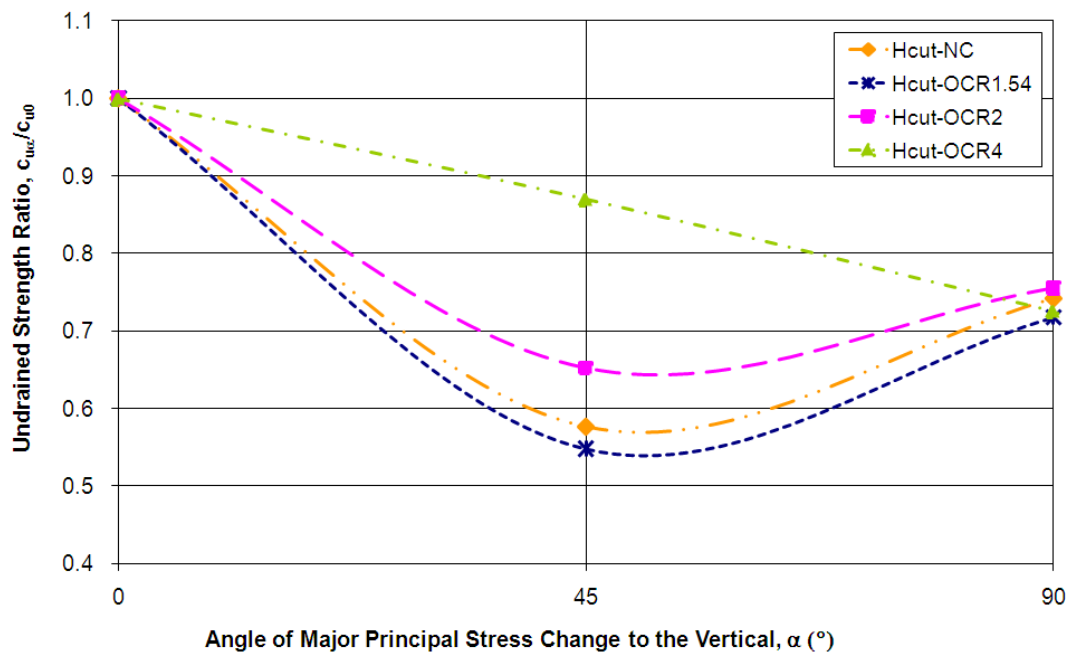


Figure 5.40 (b)

Figure 5.40 Variation of Undrained Strength Ratio with the Angle of Stress Reorientation: (a) Vertically Cut Specimens; (b) Horizontally Cut Specimens

The pore pressure parameters during undrained shearing were calculated for each type of tests. The pore pressure parameters at failure (A_f for CK_0UC & CK_0UE tests, and a_f for CK_0UPS tests) are compared in Figure 5.41 based on the specimen orientation. It is important to mention that the calculation equations for pore pressure parameters are different for CK_0UC , CK_0UE and CK_0UPS testing conditions. The corresponding equations were shown in Equation 5-3, 5-4 and 5-5. According to Das (1998), the failure is defined as the point at which the deviator stress reached its peak. Simons (1960) conducted a series of CIU tests on different types of clay, the data were also plotted in Figure 5.41 for comparison. For vertically cut intact residual soil specimens at NC state, CK_0UPS tests gave a_f values much higher than the others, followed by CK_0UC and CK_0UE tests (as shown in Figure 5.41(a)). The A_f (or a_f) values of the vertically cut intact residual soil were much higher than the ones determined by Simons. This may be because the high permeability of residual soil. It is also possible due to the different consolidation methods applied. In general, the pore pressure parameter at failure

decreased as OCR increased. It approached a constant value at high OCR. The reduction of a_f value in CK₀UPS tests was most drastic. The effect of OCR in CK₀UE tests was mild compare to others. The A_f (or a_f) value may be affected significantly by the testing conditions, and it also varied a lot from soil to soil. Similar observations were made for horizontally cut intact residual soil specimens. The A_f value reduced with OCR. The reduction of A_f value with OCR in CK₀UC tests was more significant than the one in CK₀UE tests.

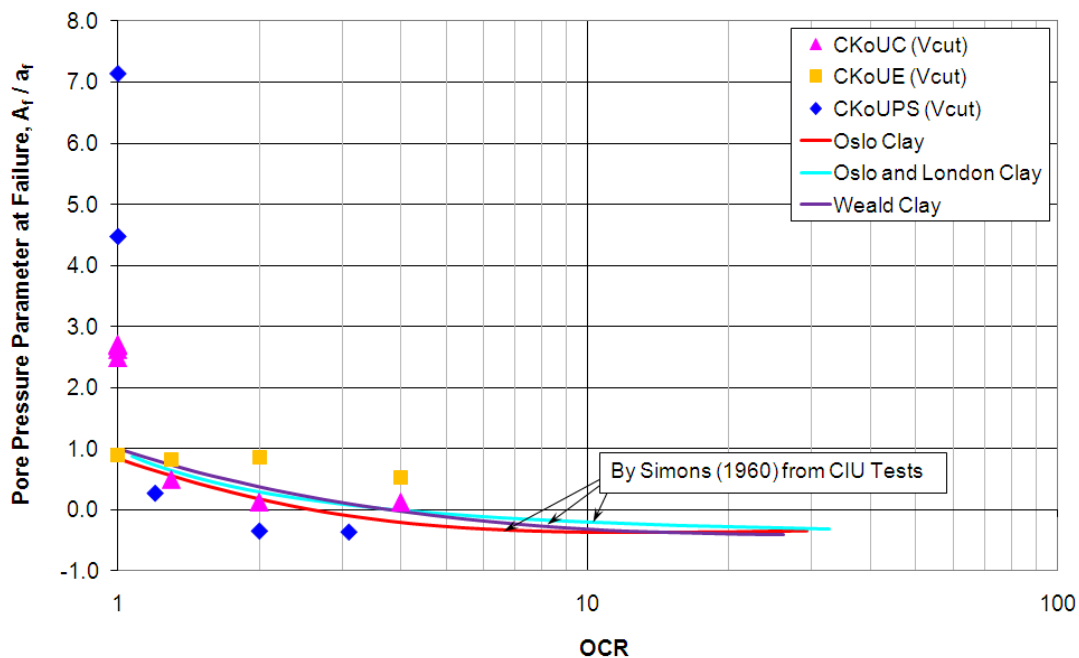


Figure 5.41 (a)

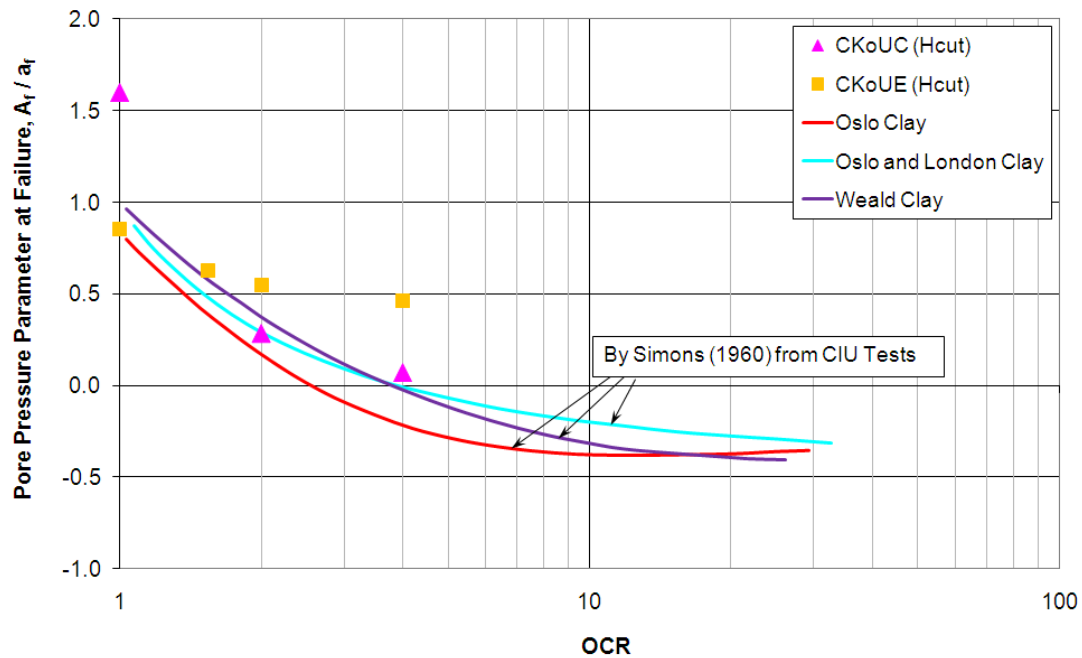


Figure 5.41 (b)

Figure 5.41 Variation of Pore Water Pressure Parameters at Failure with OCR Determined from Different Types of Tests: (a) on Vertically Cut Intact Residual Soil Specimens; (b) on Horizontally Cut Intact Residual Soil Specimens

A comparison of the secant friction angle obtained from different type of tests is shown in Figure 5.42. Here secant friction angle, ϕ_s' , is defined as:

$$\sin \phi_s' = (\sigma_1' - \sigma_3') / (\sigma_1' + \sigma_3') \quad (5-7)$$

irrespective of whether the effective cohesion is zero or not. For all types of tests, the secant friction angles obtained from tests conducted on the specimen, which consolidated to the in-situ stress was the highest. It may be because all NC, OCR2, OCR3 and OCR4 specimens had been consolidated to a stress level much more higher than the pre-consolidation stress and the structure of the residual soil specimens might have been destroyed during consolidation process as pointed out by Smith et al. (1992).

For vertically cut intact residual soil specimens, CK₀UPS tests gave the highest ϕ'_s , followed by CK₀UC tests. The lowest ϕ'_s was calculated from CK₀UDSS tests (as shown in Figure 5.42(a)).

Since no CK₀UPS tests were conducted on horizontally cut intact residual soil specimens, the CK₀UC tests gave the highest ϕ'_s (see Figure 5.42(b)). Different from the CK₀UDSS test results on vertically cut specimen, no significant reduction in the ϕ'_s value was observed for horizontally cut specimens. Hence, at high OC state (OCR > 2), value of ϕ'_s determined from CK₀UDSS tests was higher than the one obtained from CK₀UE tests.

By comparing Figure 5.42(a) and (b), it should also point out that the ϕ'_s values of horizontally cut intact residual soil were higher than the values of vertically cut specimen. The ϕ'_s values were affected by specimen orientation.

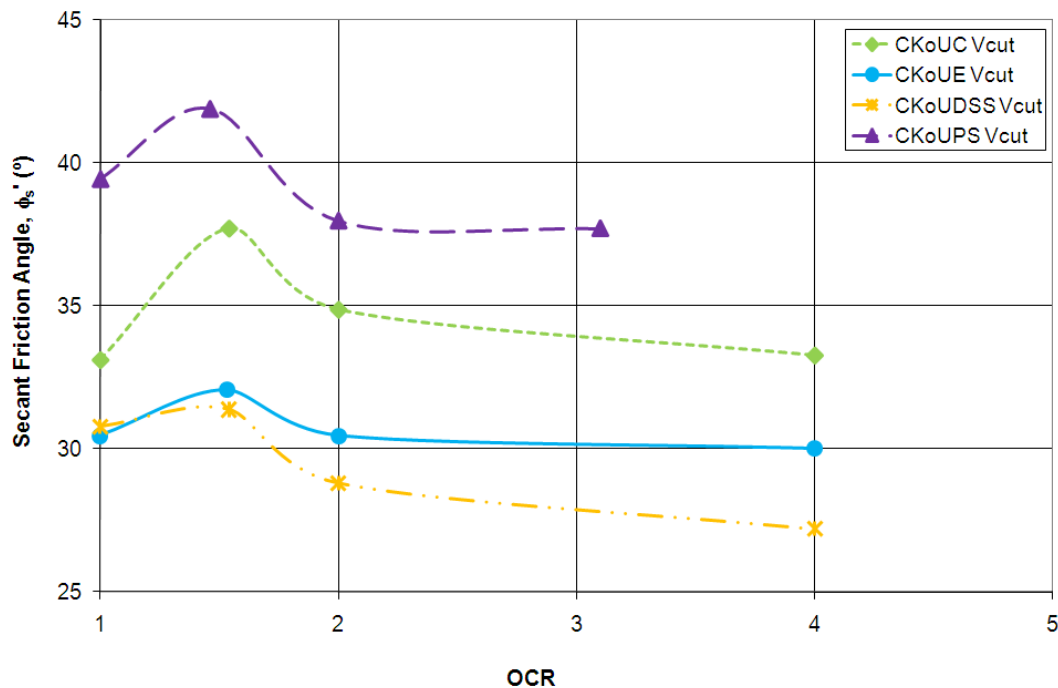


Figure 5.42 (a)

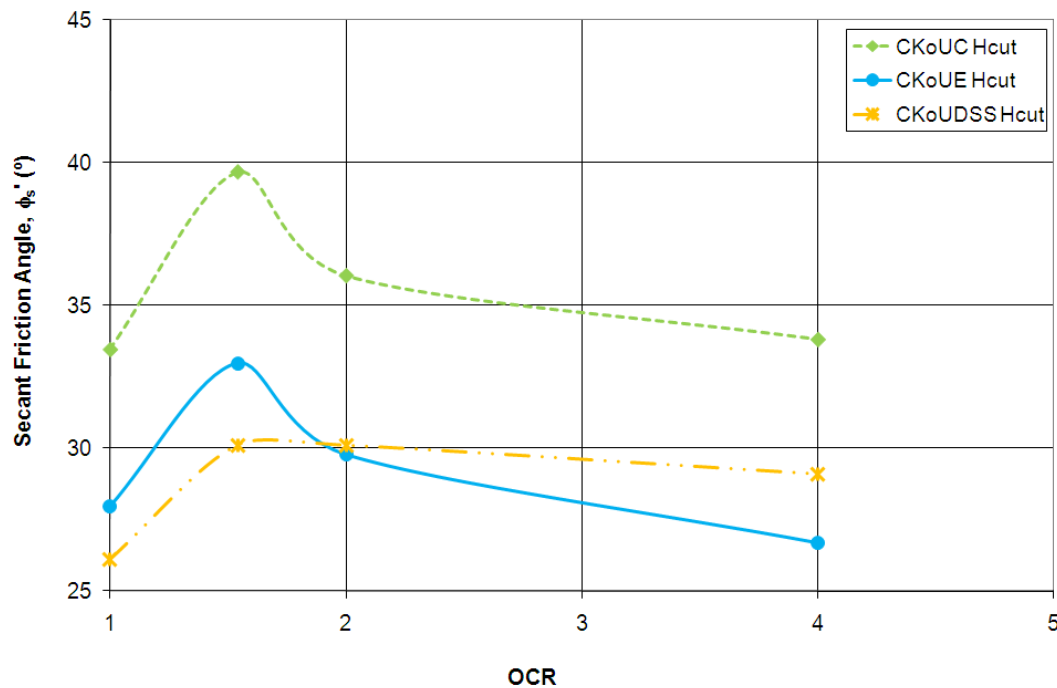


Figure 5.42 (b)

Figure 5.42 Secant Friction Angles Determined from Different Types of Tests: (a) on Vertically Cut Intact Residual Soil Specimens; (b) on Horizontally Cut Intact Residual Soil Specimens

5.7 Conclusions

A series of K_0 consolidated undrained triaxial compression (CK_0UC), extension (CK_0UE) and direct simple shear (CK_0UDSS) tests, and plane-strain (CK_0UPS) tests were conducted on both vertically cut and horizontally cut intact residual soil specimens. Results from the tests were presented and compared in this chapter. Following conclusions can be made from these results:

1. As pointed out by many researchers (Ladd, 1986; Kulhawy & Mayne, 1990; Chu et al., 1999), the undrained shear strength, c_u , is not a constant, but varies with many factors, including effective overburden stress, $\sigma_v'{}_0$, OCR, consolidation path (isotropic or K_0 consolidation), stress states and stress anisotropy. From the test results presented earlier, it is found that the undrained behaviour of the intact residual soil was affected greatly by the

consolidation stress. When the soil specimens were consolidated to a stress level just below the pre-consolidation stress, the variation of q/p' and the generation of excess pore water pressure (Δu) during undrained shearing were observed to be very distinctive different from that consolidated just above the preconsolidation stress (for example, Figure 5.34(c) and Figure 5.34(f)). It is also observed that the b-value obtained in CK_0 UPS tests were dependent on the consolidation stress (see Figure 5.34(d)), as q/p' and Δu did. The above differences may be caused by whether bonds in the intact residual could be preserved or not which is related to whether the stress level is above or below the pre-consolidation stress. Consequently, the soil behaviour was partially controlled by the soil bonds left behind. The secant friction angle is also affected by bonding. This explained partially why an excessively high friction angle can be obtained at low stress level.

2. The failure envelope of the intact residual soil was not greatly affected by the cutting orientations of soil specimen. This is observed in different types of tests including CK_0 UC, CK_0 UE and CK_0 UDSS tests conducted on specimens cut with different orientations.
3. The failure envelope was also compared between different type of tests with principal stress rotation, e.g., in CK_0 UC, CK_0 UE and CK_0 UDSS tests (see Figure 5.38). It is observed that the CK_0 UPS test gives the highest failure envelope, which is consistent with previous studies, such as Ladd & Lambe (1963), Ladd et al., (1980), Wanatowski & Chu (2008) and Mayne et al. (2009).
4. Normalized stress behaviour, that is, the independence of $c_u/\sigma_v'0$ of $\sigma_v'0$, was observed in general for the normally consolidated (NC) intact residual soil specimens. In anisotropically consolidated shearing tests, both q and p' at the end of consolidation are different. To overcome this problem, a stress parameter, $T = M^2 p' + q^2/p'$ was used to normalize the behaviour obtained from CK_0 UC, CK_0 UDSS and CK_0 UPS tests. The normalized stress paths

were consistent in different type of tests. A constant $(q/\sigma_1')_{NC}$ value was obtained for each type of tests. The results demonstrated the applicability of the well-known power relationship between c_u/σ_v' and OCR to the intact residual soil used in this study.

5. The c_u/σ_v' versus OCR relationships were established for each type of tests. The tests conducted on vertically and horizontally cut specimens did not show a strong strength anisotropy in terms of c_u/σ_v' ratio. However, pronounced strength anisotropy both in terms of effective friction angle and undrained shear strength was observed from the tests with major principal stress rotation and plane-strain tests. It is observed from the testing results that, in general, under a given OCR the c_u/σ_v' obtained from the CK₀UPS tests is the highest and this is followed by the CK₀UC tests. The CK₀UE tests give the lowest one, whereas the value obtained from CK₀UDSS tests is somewhere in between. The strength anisotropy could be induced by different degree of bond breakdown due to different volumetric responses with major principal stress rotation. This is consistent with the observations of Malandraki and Toll (2000).
6. The pore pressure parameter at failure was calculated for each individual test (A_f for CK₀UC & CK₀UE tests, and a_f for CK₀UPS test). At NC state, A_f is comparable to a_f . However, with increasing in OCR, the A_f/a_f value decreases significantly as shown in Figure 5.41. The OCR effect was most significant in CK₀UPS test.

It should be pointed out that the testing data could have been affected by the natural variation and nonhomogeneity in the block samples used. Therefore, some of the conclusions may not be generalized without further verification. For this reason, tests on compacted residual soil and an artificially cemented soil were found necessary.

CHAPTER 6

UNDRAINED BEHAVIOUR OF ARTIFICIALLY CEMENTED SOIL

6.1 Introduction

As mentioned in Chapter 3, the residual soil of weathered Bukit Timah granite had a true cohesion that is small enough to be ignored. Thus, it leads to the question on whether the shear strength and stress-strain behaviour determined for residual soils applicable to cemented soil. In order to study the cementation effects, an artificially cemented soil was tested to compare with the results of intact residual soil. A series of K_0 consolidated undrained triaxial compression (CK_0UC) and direct simple shear (CK_0UDSS) tests were conducted on the artificially cemented soil specimens. The soil behaviour under axisymmetric and plane-strain conditions is compared. Few extension tests were also conducted for illustration purpose. The results obtained from these tests are presented in this chapter. Major findings from the tests are highlighted. The strength and stress-strain behaviour were compared between intact residual soil and artificially cemented soil.

6.2 K_0 Consolidated Undrained Triaxial Compression (CK_0UC) Tests

A series of K_0 consolidated undrained triaxial compression (CK_0UC) tests were conducted on the artificially cemented soil specimens. A summary of the tests is given in Table 6.1. The undrained behaviour is presented in Figure 6.1. It should mention that in order to check the effect of consolidation methods on the undrained

Table 6.1 Summary of CK_0UC Tests on Artificially Cemented Soil Specimens

Test No.		AGN3	AGN4	AGN6	AGN7	AGN13	AGN18
Pressure for Saturation (kPa)	Cell	860	860	860	860	860	860
	Back	850	850	850	850	850	850
OCR		1	1	2.85	1.81	1	5
Shearing Rate (mm/min)		0.005	0.005	0.005	0.005	0.005	0.005
Weight (g)	Pre-Test	271.91	274.68	289.62	274.59	275.51	277.86
	Post-Test	357.18	356.89	377.39	366.39	353.27	371.62
Water Content (Post-Test) (%)	Average	30.80	29.54	30.10	32.46	Specimen used for post-test volume measurement	33.75
	Top	30.69	28.80	30.02	32.49		33.96
	Middle	30.76	29.25	31.11	32.60		33.75
	Bottom	30.96	30.58	27.64	31.91		33.53
	Shear Band	N.A.	N.A.	31.64	32.83		N.A.
σ'_{10} (kPa)		1150.06	1012.42	350.58	552.63	1139.642	199.84
q_0 (kPa)		680.06	618.42	306.58	445.63	672.642	177.845
p'_0 (kPa)		696.35	600.137	146.192	255.542	691.214	81.675
q_{peak} (kPa)		701.2	637.88	518.93	480.8	729.572	326.164
ε_1 at q_{peak} (%)		0.276	0.256	0.851	0.246	0.185	0.68
$(q/p')_{\text{peak}}$		1.422	1.509	2.372	1.886	1.493	2.546
$R_{\text{max}} = (\sigma'_1/\sigma'_3)_{\text{max}}$		3.695	4.017	12.041	6.044	3.972	17.813
ε_1 at $(q/p')_{\text{peak}}$ (%)		5.511	7.916	0.233	0.37	7.321	0.163
Secant Friction Angle, ϕ' (°)		36.838	36.967	57.848	45.731	36.709	62.67

behaviour of the cemented soil, specimen AGN13 was consolidated to the same stress state as specimen AGN3 along a constant stress ratio path. The constant stress ratio chosen for AGN13 is $R = 2.45$, which is the same as the one for AGN3 at end of K_0 consolidation.

The effective stress paths during K_0 consolidation and undrained shearing are shown in Figure 6.1(a). It is observed that the failure envelope of the artificially cemented soil is greatly dependent of OCR. When $OCR = 3$, a failure envelope with a slope as high as 2.52 was determined. It can be seen more clearly from the normalized stress paths presented in Figure 6.2. This could be affected by soil structures. With a low consolidation stress, the existence of soil bonds lead to a higher strength, and stress ratio. The soil structure breaks down with increasing consolidation stress, consequently, will result in a lower value of strength. Using Lade's method as discussed in Chapter 2, an instability line with a slope of 1.12 can be determined by joining the origin and the q_{peak} values of NC specimens (AGN3, AGN4 and AGN13) as shown in Figure 6.1(a).

The stress-strain responses of artificially cemented soil are presented in Figure 6.1(b). It is compared between normally consolidated (NC) and over-consolidated (OC) specimens. The NC specimens reached the peak deviator stress, q_{peak} at a smaller axial strain (0.18-0.28%) than OC specimens (0.25-0.85%). Strain softening was observed for both NC and OC specimens. For NC specimen, the effective stress ratio, q/p' attained the peak long after the q_{peak} (as shown in Figure 6.1(c)). It indicated the failure of NC specimen occurred after the peak deviator stress. In contrast, the OC specimen failed before the deviator stress reached its peak value, which behaved as a brittle material during undrained shearing. The q_{peak} cannot be compared directly since the specimens were consolidated to different stress levels. Therefore, the stress-strained curves were normalized against $\sigma_1'{}_0$ in Figure 6.1(d). The normalized deviator stress ($q/\sigma_1'{}_0$) increased with OCR. A unique value of 0.62 was obtained for NC specimens. It is evident that the $c_u/\sigma_1'{}_0$ versus OCR relationship could also be applicable to cemented soil. Future studies on the normalized behaviour of OC clays should be carried out for verification.

Another important observation that should be mentioned is that Test AGN3 and AGN13, which consolidated to the same stress level along different stress paths, achieved different q_{peak} and $(q/p')_{\text{peak}}$. It is evident that the undrained behaviour is affected by consolidation methods.

Figure 6.1(d) presents the generation of excess pore water pressure during undrained shearing. For NC specimens (AGN3, AGN4 and AGN13), the excess pore water pressure kept increasing until specimen failed. For OC specimens (AGN6 and AGN18), the excess pore water pressure decreased first due to the dilation of soil specimen. It increased as shearing continued further. For the slightly over-consolidated specimen (AGN7), the generation of excess pore water pressure was in between.

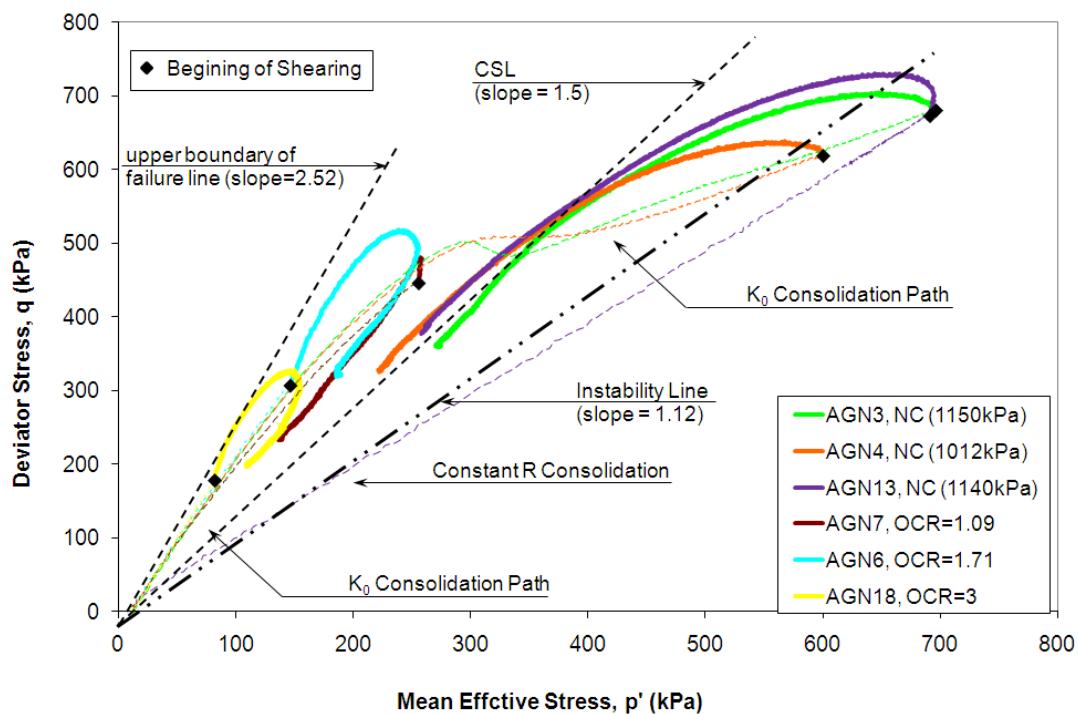


Figure 6.1(a)

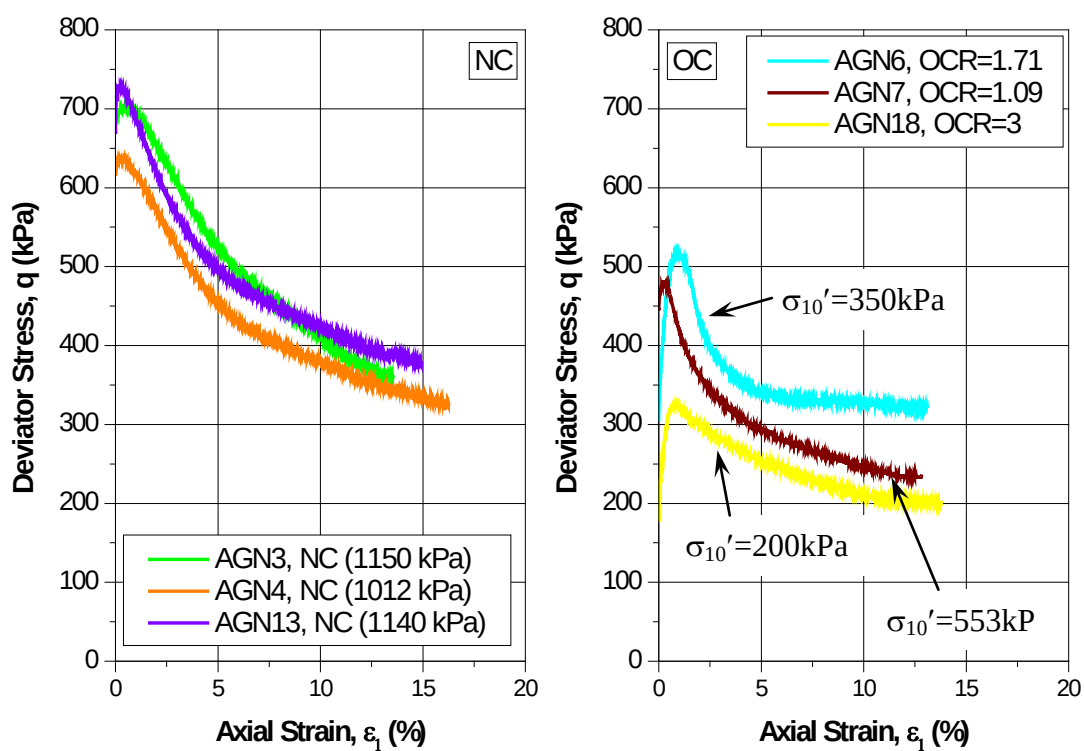


Figure 6.1(b)

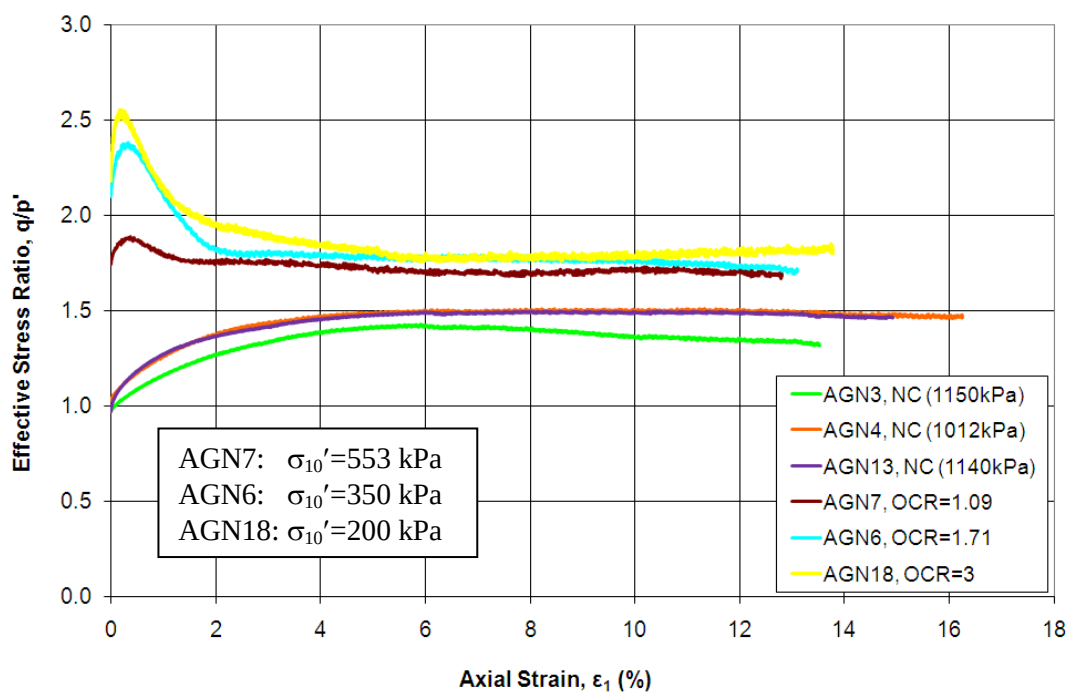


Figure 6.1(c)

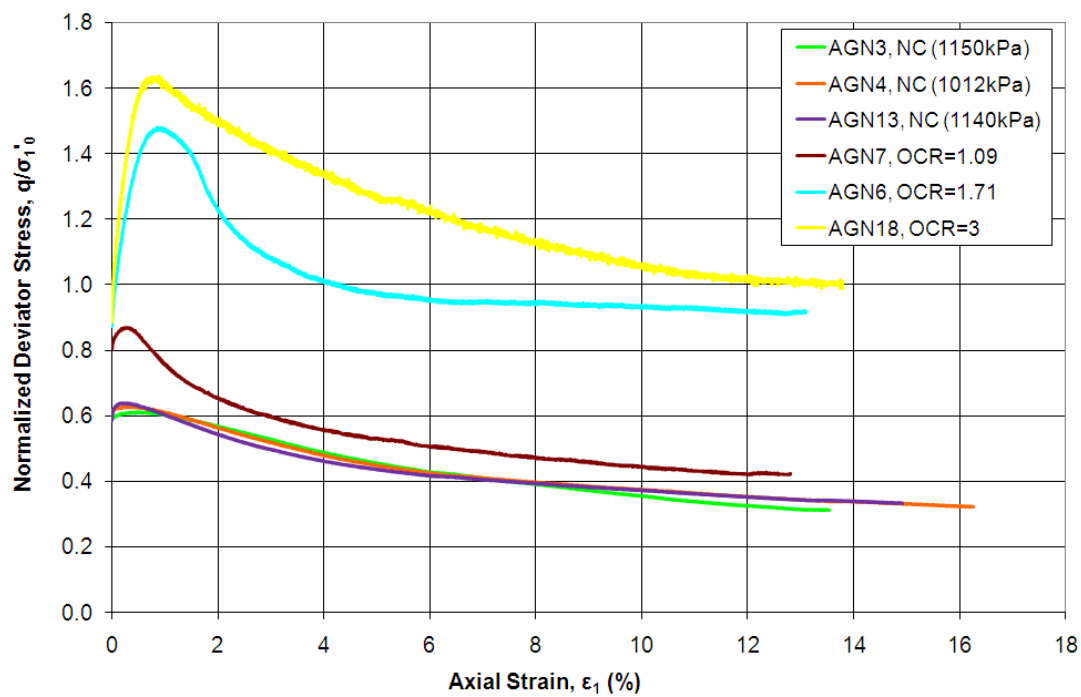


Figure 6.1 (d)

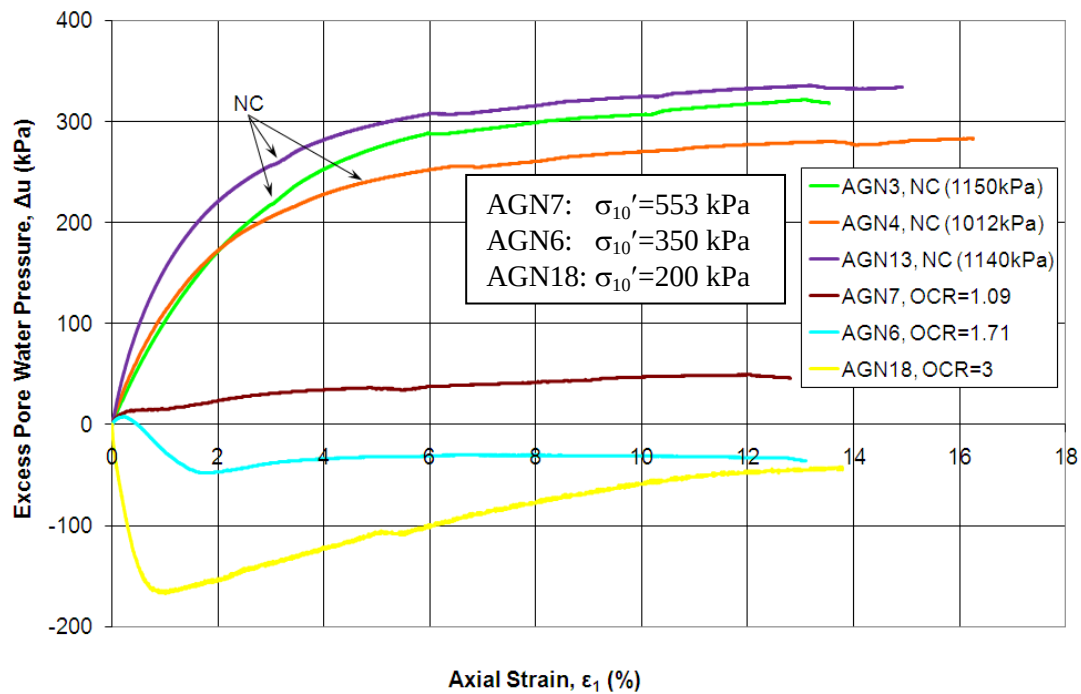


Figure 6.1(e)

Figure 6.1 Undrained Behaviour of Artificially Cemented Soil from CK_0UC Tests:

- (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) q/p' vs. ϵ_1 % Curves; (d) Normalized Stress-Strain Curves; (e) Δu vs. ϵ_1 % Curves

Due to anisotropic consolidation, the effective stress paths started from different q and p' values for each test. A coupled stress $T_0 = M^2 p'_0 + q_0^2 / p'_0$ was used to normalize the behaviour of artificially cemented soil under CK₀UC conditions. (in which M is the slope of critical state line). This is the same method adopted on the results of intact residual soils. The critical state line determined from tests on NC specimens (AGN3, AGN4 and AGN13) is shown by dotted line in Figure 6.2, which has a slope of 1.5. Normalized behaviour was observed for NC specimens. It is also observed that the failure envelope of the artificially cemented soil was highly dependent of OCR. The slope of failure envelope increased with increasing OCR. This could be due to the strong cementation in artificial soil which created during sample preparation process.

The variation of Skempton's pore pressure parameter, A in CK₀UC tests is presented in Figure 6.3(a). The A value increased as shearing carried out for NC specimen. It increased drastically after the peak deviator stress was reached, which resulted from the straining softening of NC specimen. For OC specimen, little variation of A value was observed. A negative A value was obtained at the beginning stage of shearing due to the reduction of pore water pressure. Figure 6.3(b) shows the variation of A values at failure (A_f) with OCR. The term "failure" mentioned here is defined by Das (1998) as the peak deviator stress (q_{pak}) in undrained shearing. The A_f values decreased with OCR, which is consistent with Simons' observation (Simons, 1960) from CIU tests. The difference between A_f value presented in this chapter and the ones reported by Simons' is because the artificially cemented soil specimens were K_0 consolidated, rather than isotropic consolidation. Similar observation was made on the intact residual soils also, as presented in Chapter 5.

The c_u / σ'_{10} determined from CK₀UC tests on artificially cemented soil is plotted with OCR in Figure 6.4. It is noted that the data scattered in the tested stress range. The value determined from test AGN18 may not be reliable, since it was consolidated to a low stress level, which led to an early occurrence of shear band. The formation of shear band may significantly affect the strength behaviour of soil.

In order to determine a reliable relationship between c_u/σ'_{10} and OCR, more tests should be conducted at high OCR state.

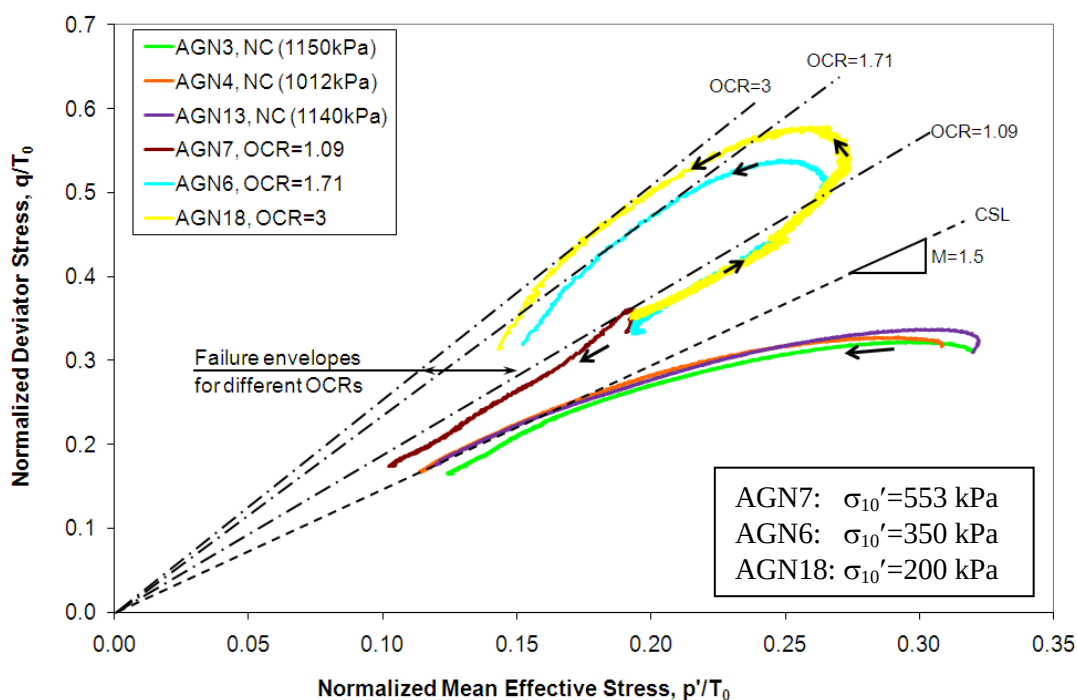


Figure 6.2 Normalized Effective Stress Paths

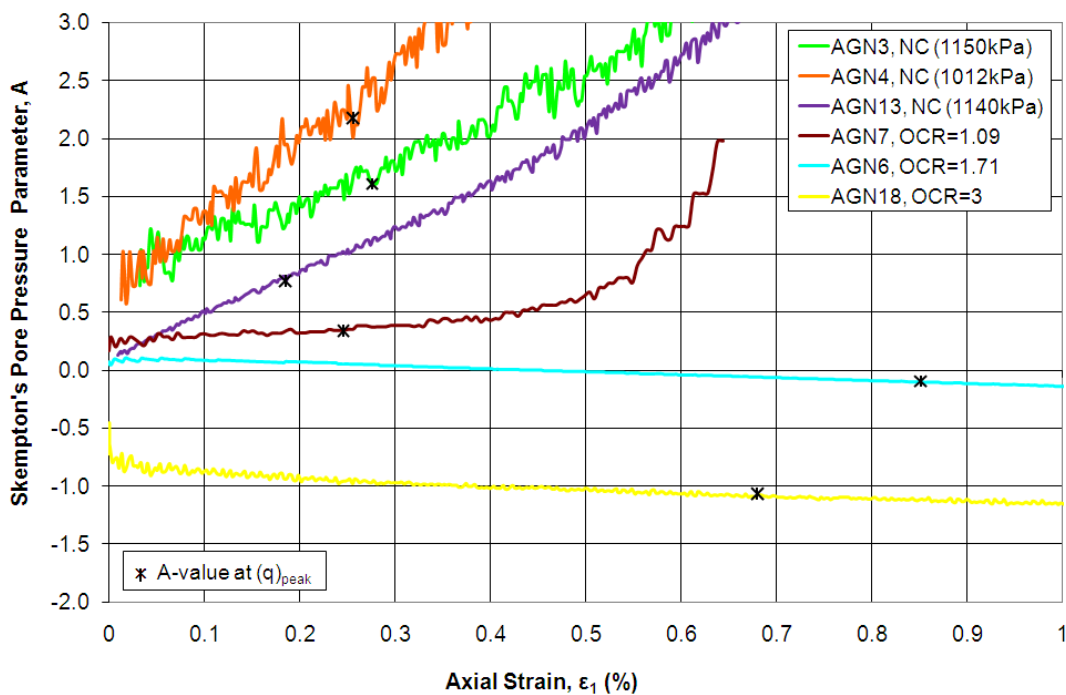


Figure 6.3(a)

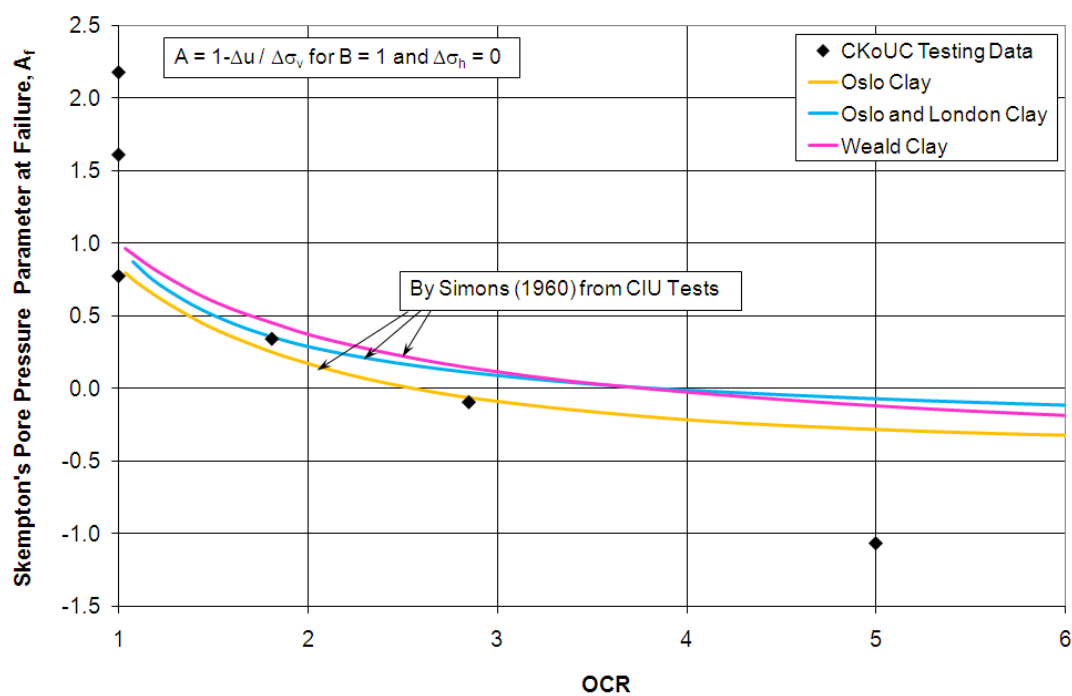


Figure 6.3 (b)

Figure 6.3 Variation of Skempton's Pore Pressure Parameter, A in CK_0UC Tests:

(a) A vs. $\varepsilon_1\%$ Curves; (b) A_f vs. OCR Curves

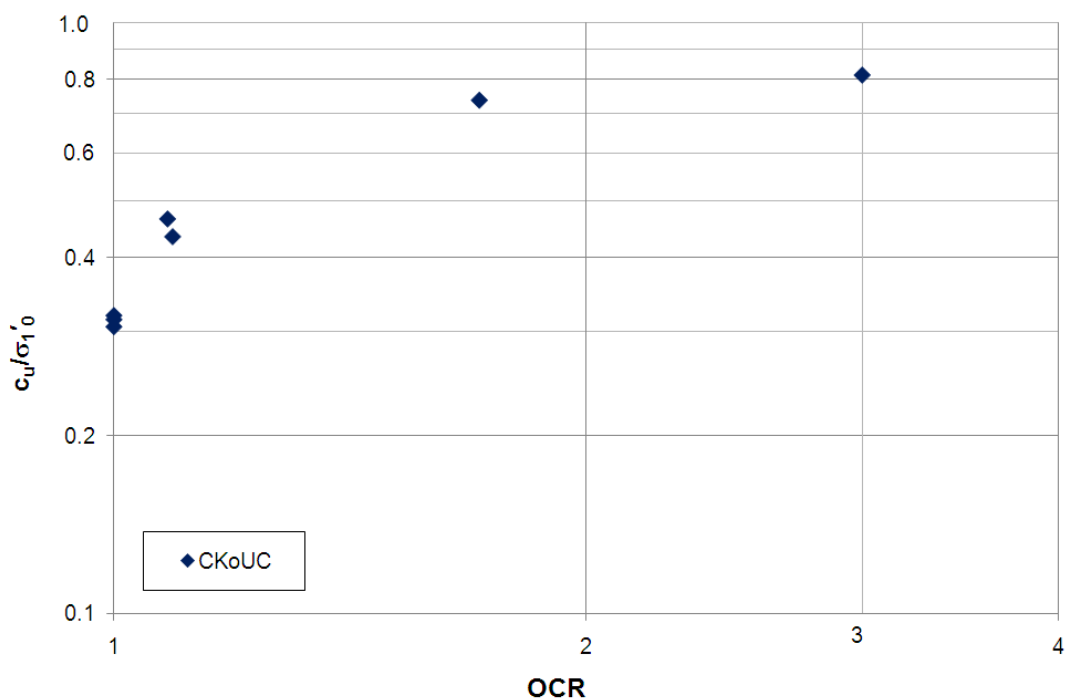


Figure 6.4 $c_u / \sigma'_1 0$ vs. OCR Relationship Obtained from CK_0UC Tests on Artificially Cemented Soil

6.3 K_0 Consolidated Undrained Direct Simple Shear (CK_0 UDSS) Tests

To study the undrained behaviour under plane-strain condition, a series of K_0 consolidated undrained direct simple shear (CK_0 UDSS) tests were conducted on the artificially cemented soil (as summarized in Table 6.2). The test results are presented in Figure 6.5. The stress-strain responses during the CK_0 UDSS tests are presented in Figure 6.5(a). It can be seen that the shear stress reached a peak and followed by strain softening. Figure 6.5(b) shows the response of normal stress during undrained shearing. The change in the vertical stress can be considered as the change in pore pressure (Chu et al., 1999; Chu et al., 2003). It is noted that the normal stress for OC specimens increased first then followed by a reduction, which consistent with the dilation and compression behaviour of OC clay. In contrast, for NC specimen, the normal stress decreased from the beginning, and trended to be constant with further strain increased. The effective stress paths are plotted in Figure 6.5(c). All the stress paths bent to the left. A unique failure envelope with a slope of 0.67 was obtained. The maximum stress ratio was attained after the peak shear stress as shown in Figure 6.5(d).

Table 6.2 Summary of CK_0 UDSS Tests on Artificially Cemented Soil Specimens

Test No.	DSS1	DSS2	DSS3	DSS4	DSS5	DSS7	DSS8
OCR	1.5	1	1	3	5	2	1
Shearing Rate (mm/min)	0.036	0.036	0.036	0.036	0.036	0.036	0.036
σ_{n0} (kPa)	100.16	300.01	499.79	99.91	100.19	250.1	398.7
τ_{max} (kPa)	41.24	86.50	136.28	82.23	110.47	133.32	110.49
Shear Strain at τ_{max} (%)	5.05	2.77	1.87	3.77	2.22	1.48	3.82
σ_n at τ_{max} (kPa)	72.24	166.21	285.63	136.80	189.34	243.12	207.43
$(\tau/\sigma_n)_{max}$	0.62	0.60	0.72	0.75	0.69	0.75	0.6
Secant Friction Angle, ϕ' (°)	31.80	30.96	35.75	36.87	34.61	36.87	30.96

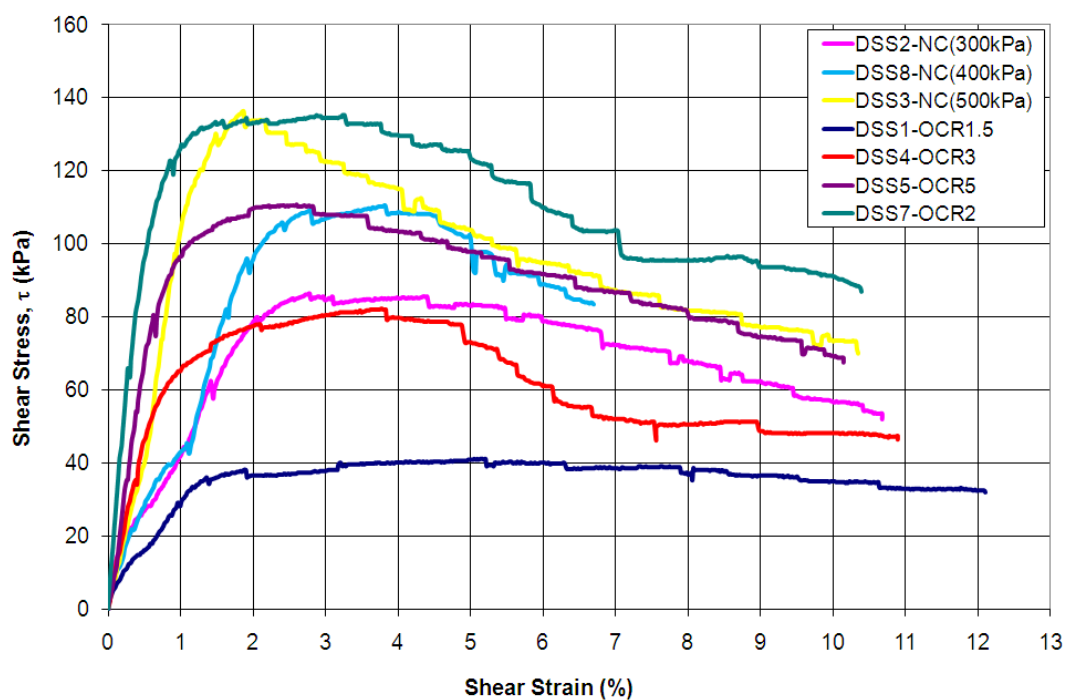


Figure 6.5(a)

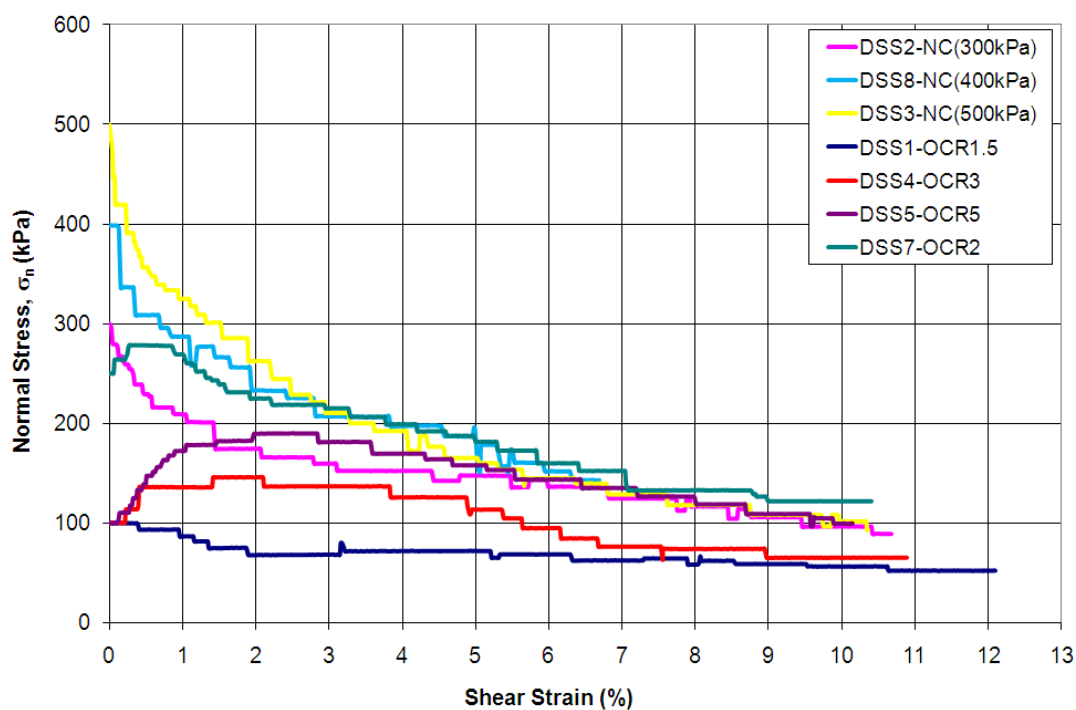


Figure 6.5(b)

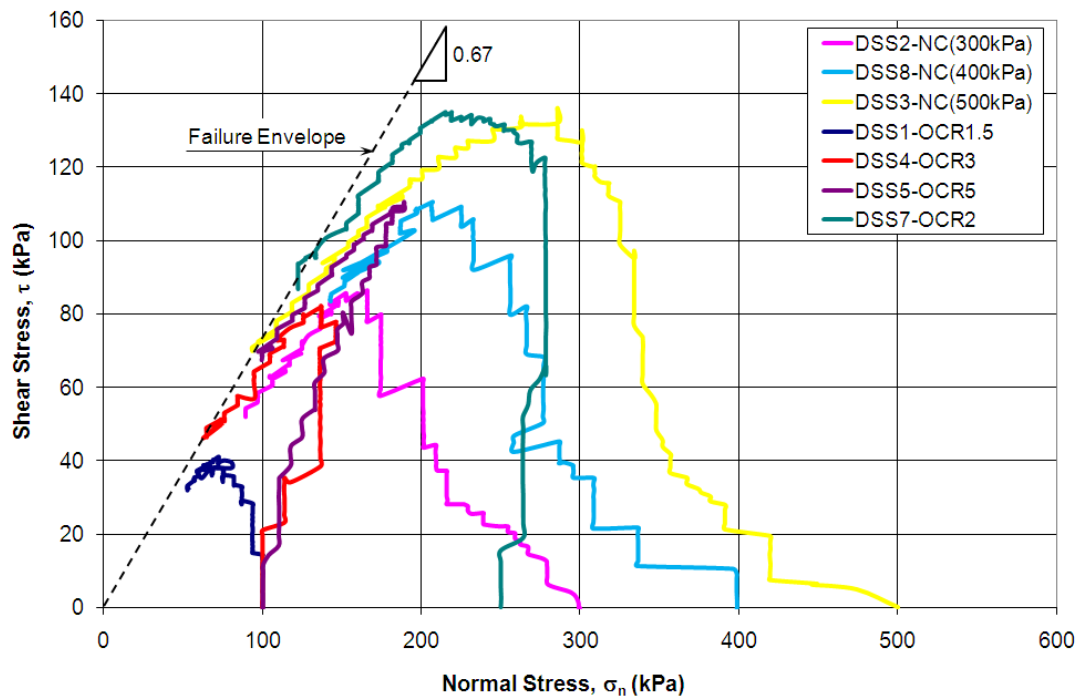


Figure 6.5(c)

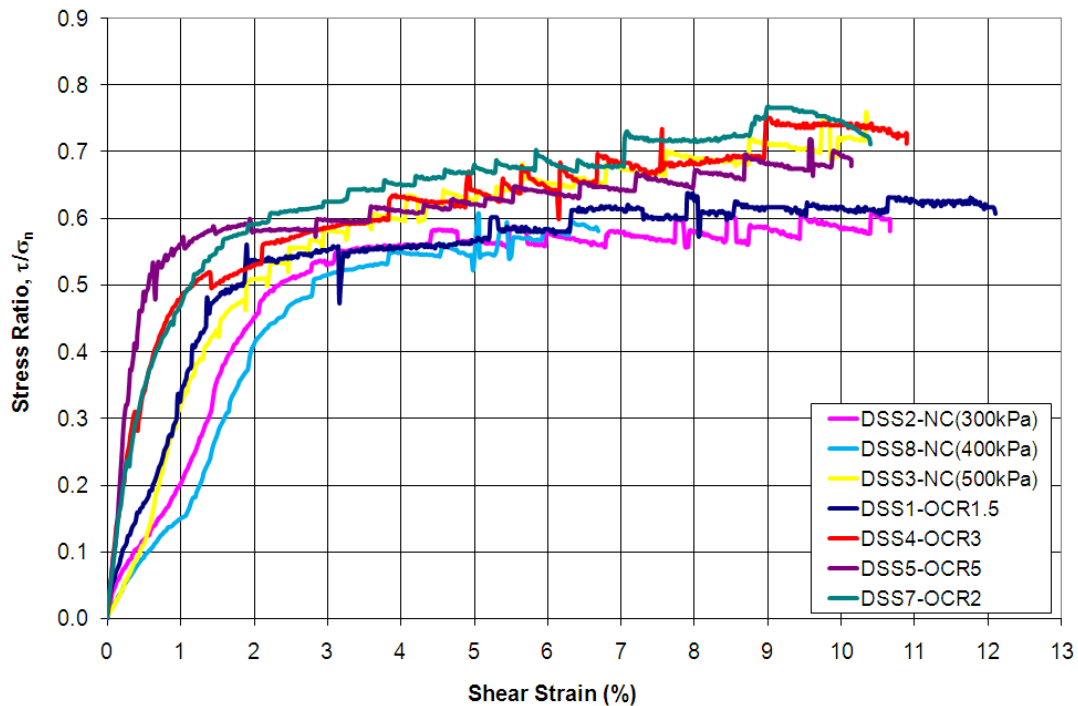


Figure 6.5(d)

Figure 6.5 Undrained Behaviour of Artificially Cemented Soil from CK_0 UDSS Tests:

- (a) Shear Stress-Strain Curves; (b) Normal Stress vs. Shear Strain Curves; (c) Shear Stress vs. Normal Stress Curves; (d) Stress Ratio vs. Shear Strain Curves

To compare the tests conducted under different consolidation stresses, the CK_0UDSS test results are normalized against the initial normal stress, σ_{n0} . The normalized strength and shear stress-strain behaviour are presented in Figure 6.6. The normalized effective stress paths and normalized shear stress versus shear strain curves for NC specimens are unique as shown in Figure 6.6(a) and (b), respectively. For OC clay, the τ/σ_{n0}' increased with OCR.

The τ/σ_{n0} versus OCR relationship determined from CK_0UDSS tests on artificially cemented soil is presented in Figure 6.7. The test results fitted into the well-established power relationship precisely. An average value of 0.279 is taken for $(\tau/\sigma_{n0})_{NC}$. The corresponding equation is shown in Figure 6.7.

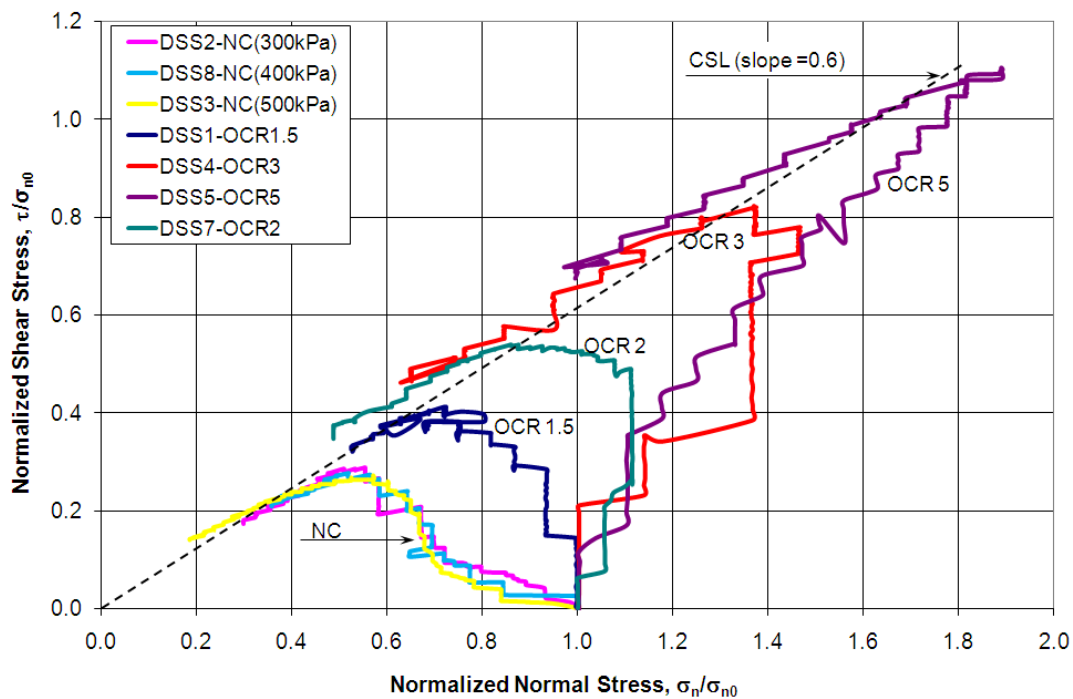


Figure 6.6(a)

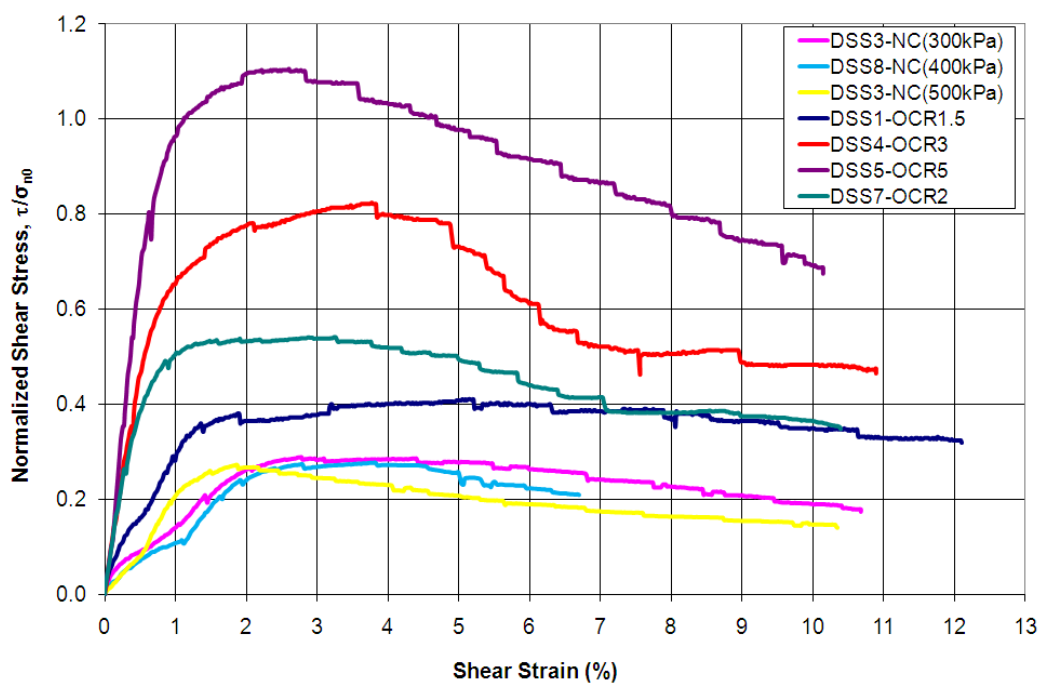


Figure 6.6(b)

Figure 6.6 Normalized Undrained Behaviour of Artificially Cemented Soil from CK_0UDSS Tests: (a) Normalized Shear Stress vs. Normal Stress Curves; (b) Normalized Shear Stress-Strain Curves

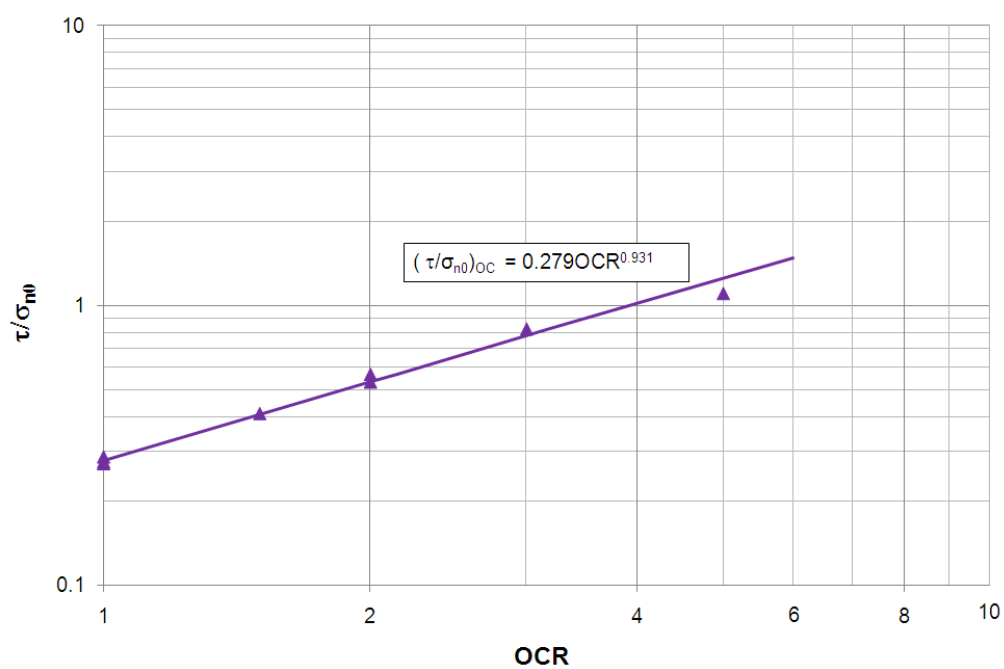


Figure 6.7 τ/σ_{n0} vs. OCR Relationship Obtained from CK_0UDSS Tests on Artificially Cemented Soil

6.4 K_0 Consolidated Undrained Extension (CK_0UE) Tests

Three CK_0UE tests were conducted on the artificially cemented soil specimens for illustration purpose. Test AGN24 and AGN27 were conducted first to study the soil behaviour under different confining pressures. Since the artificially cemented soil was fired at 500°C to create bonding between soil particles, the degree of saturation could always be monitored. Therefore, one more test (AGN30) was conducted to check if the applied pressure is higher enough to saturate the artificially cemented soil specimen. To conduct the tests at same stress level, all three specimens were consolidated along a constant stress ratio path to attain a deviator stress of 120kPa and a mean effective stress of 104kPa. The detail information on the three tests is available in Table 6.3.

Table 6.3 Summary of CK_0UE Tests Conducted on the Artificially Cemented Soil

Test No.		AGN24	AGN27	AGN30
Pressure for Saturation (kPa)	Cell	860	510	860
	Back	850	500	850
OCR		3.25	3.26	3.24
Shearing Rate (mm/min)		0.005	0.005	0.005
Initial Void Ratio		0.914	0.930	0.938
Weight (g)	Pre-Test	273.88	273.73	271.28
	Post-Test	365.75	367.35	364.24
Water Content (Post-Test) (%)	Average	33.58	34.13	33.93
	Top	31.31	34.43	34.39
	Middle	33.73	33.36	34.11
	Bottom	33.76	33.64	32.43
	Shear Band	35.50	35.09	34.78
$\sigma_1'{}_0$ (kPa)		184.7	184.25	185.2
q_0 (kPa)		120.70	120.75	121.70
p'_0 (kPa)		104.24	103.75	104.07
q_{peak} (kPa)		-184.24	-166.32	-174.76
ε_1 at q_{peak} (%)		-2.067	-1.781	-1.976
c_u/σ_{10}'		0.499	0.451	0.472
$(q/p')_{peak}$		-1.594	-1.516	-1.542
ε_1 at $(q/p')_{peak}$ (%)		-0.847	-0.744	-0.842
$R_{peak} = (\sigma_1'/\sigma_3')_{peak}$		4.4	4.06	4.17
Secant Friction Angle, ϕ' (°)		39.03	37.24	37.83

The stress paths obtained from CK_0UE tests are presented in Figure 6.8(a). It is noted that the stress paths are independent of cell pressure levels, all the three tests approached to the same failure envelope with a slope of 1.545. This resulted in consistent stress ratio variations as shown in Figure 6.8(b). However, it can be seen from Figure 6.8(c) that the stress-strain curves diverged after reached the peak deviator stress. This could be due to the formation of shear band or necking after failure. At failure, the peak deviator stress could still increase, due to the rearrangement of soil particles; however, the increment will last until shear band fully developed. It is also observed a variation in peak deviator stress; this may be affected by the initial void ratio of soil specimens. Similar observation was made on the Δu versus axial strain curves, as presented in Figure 6.8(d). The Δu decreased initially until peak deviator stress was reached and increased subsequently with further shearing. Such behaviour was typical for OC clay in undrained extension tests, since the specimen was dilated first, and then compressed.

The variation of Skempton's pore pressure parameter A is presented in Figure 6.9. The curves determined from three CK_0UE tests are very consistent. The variation in A_f value could be due to the variation of q_{peak} , which affected by the initial void ratio of soil specimens.

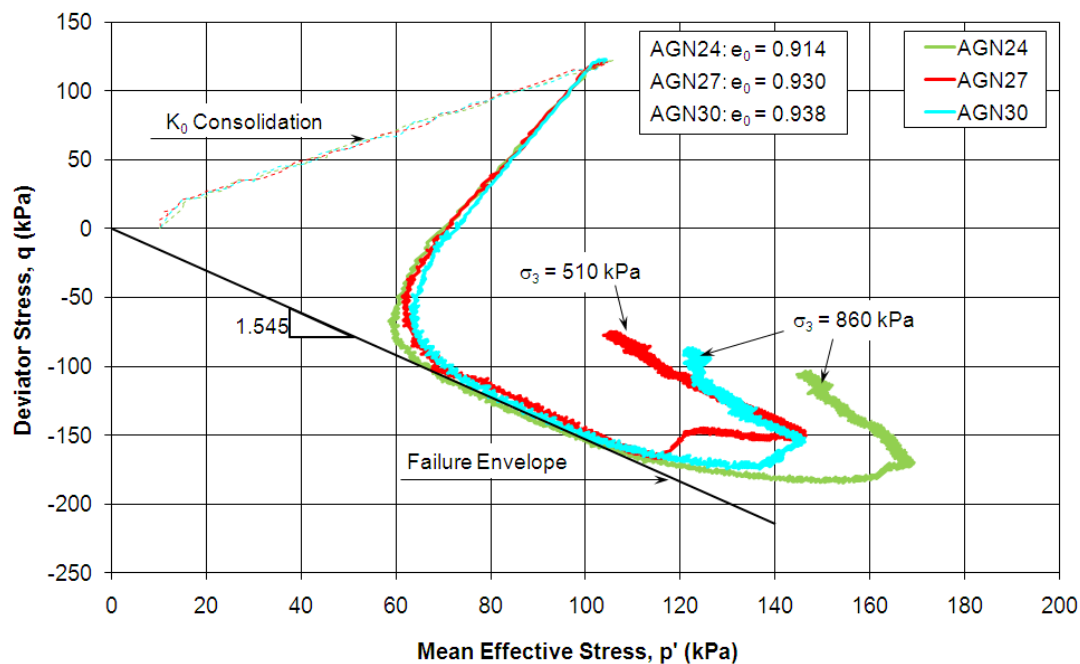


Figure 6.8 (a)

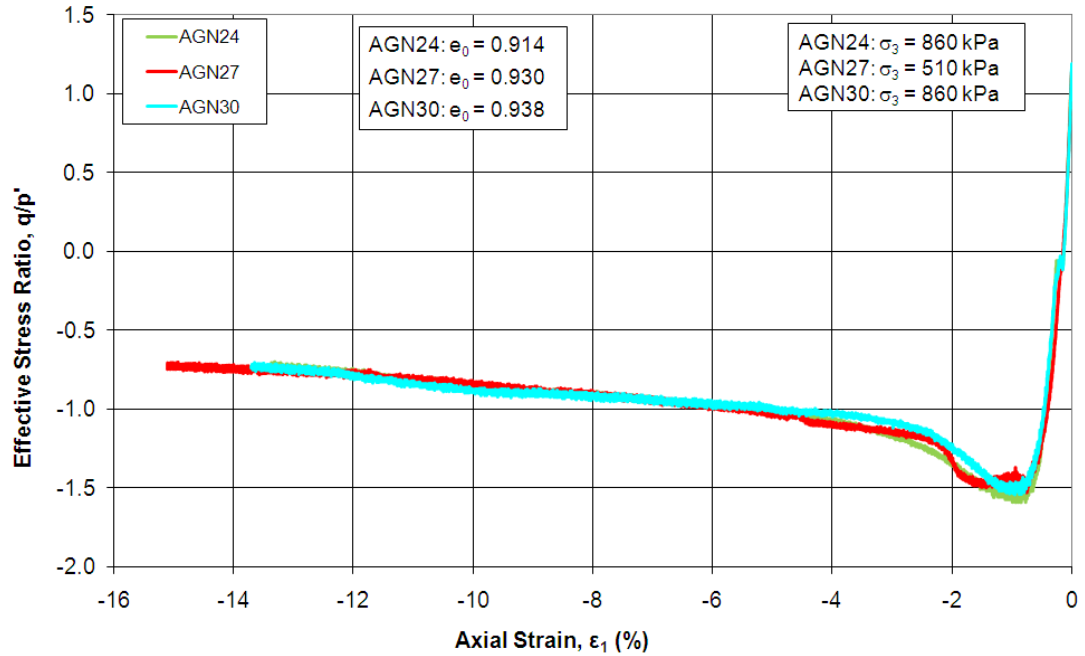


Figure 6.8 (b)

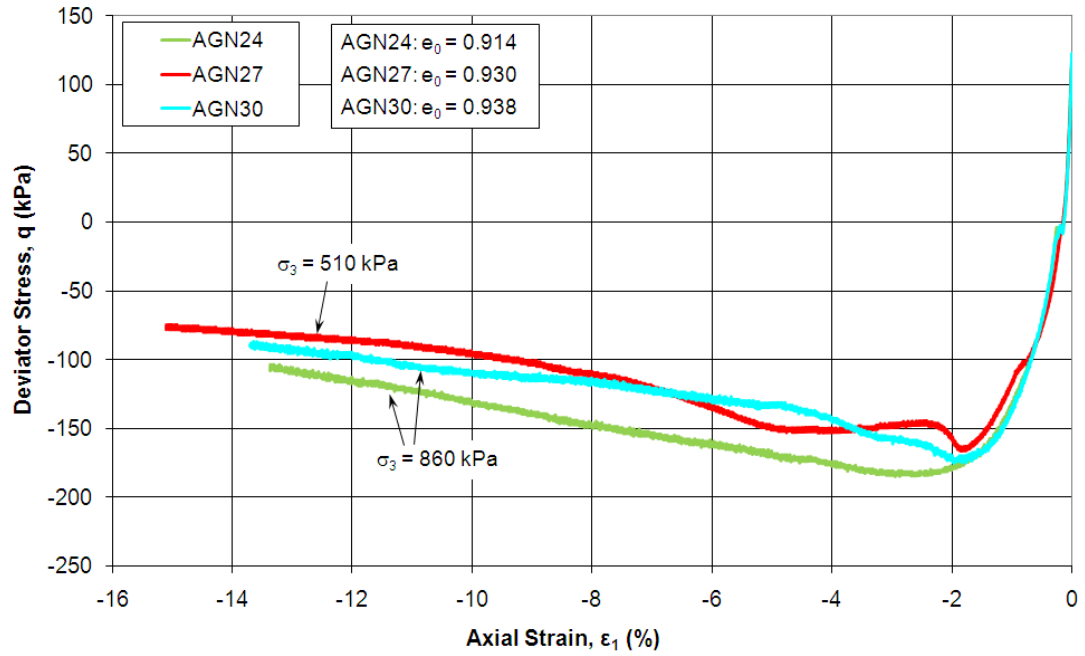


Figure 6.8 (c)

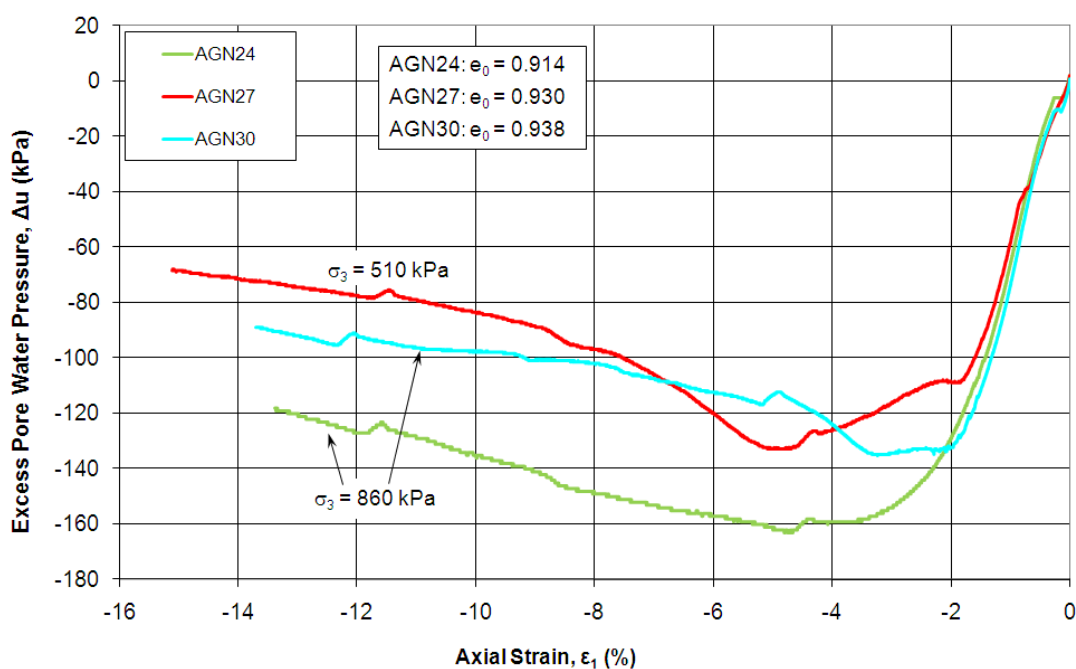


Figure 6.8 (d)

Figure 6.8 Results from CK_0UE Tests on Artificially Cemented Soil Specimens: (a) Effective Stress Paths; (b) q/p' vs. $\varepsilon_1\%$ Curves; (c) Stress-Strain Curves; (d) Δu vs. $\varepsilon_1\%$ Curves

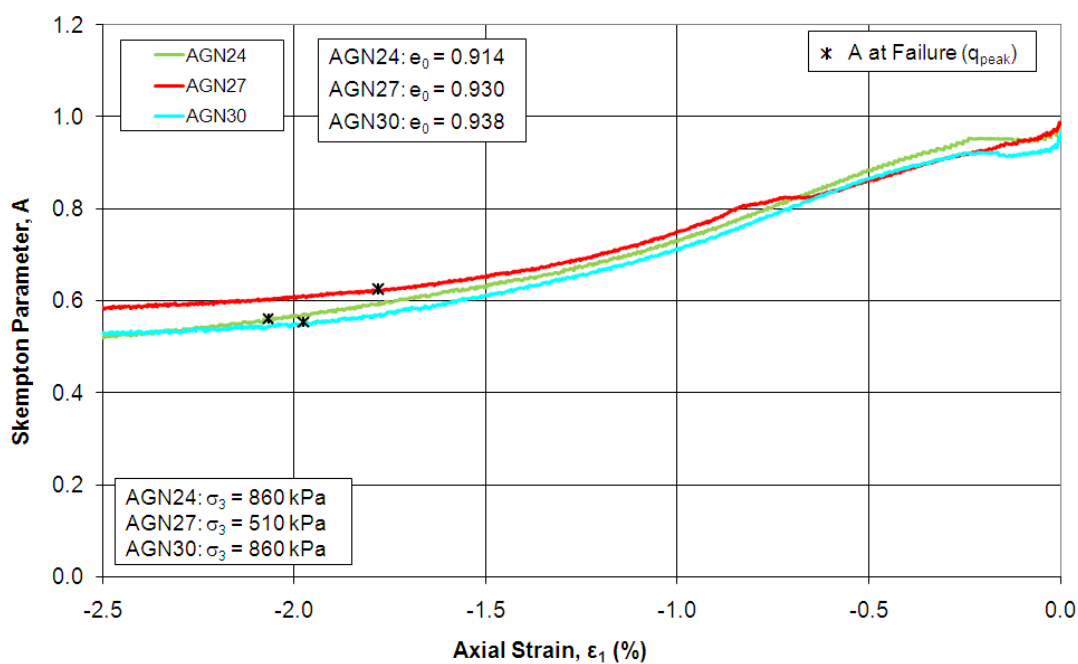


Figure 6.9 Variation of Skempton's Pore Pressure Parameter A with Axial Strain in CK_0UE Tests

6.5 Discussion and Conclusions

6.5.1 Comparison of Tests conducted on Artificially Cemented Soil

The failure envelopes determined from CK₀UC, CK₀UDSS and CK₀UE tests are compared in Figure 6.10. The failure envelope for CK₀UC tests was taken as the highest one determined from the test with OCR=3 (Test AGN18) as shown in Figure 6.1(a). The failure envelope obtained from CK₀UDSS was converted into the q - p' space using the following equation:

$$\tan \alpha' = \frac{q}{p'} = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (6-1)$$

where ϕ' is the angle of internal friction in terms of effective stress under DSS condition. The derivation of the relationship is given in Appendix I. It can be seen from Figure 6.10, the CK₀UC tests gave a highest failure envelope (largest secant friction angle), followed by CK₀UDSS tests. The failure envelopes determined from CK₀UE tests are the lowest. Similar observations were made on the intact residual soil as presented in Chapter 5. Strength anisotropy was pronounced in terms of failure envelope by the rotation of major principal stress during CK₀UC, CK₀UDSS and CK₀UE tests.

The $c_u/\sigma_1'{}_0$ versus OCR relationships are compared in Figure 6.11. It is observed the CK₀UDSS data fitted into a linear relationship on log-log scale. The $c_u/\sigma_1'{}_0$ values obtained from CK₀UC tests were scattered significantly above the line. Again, the extension tests gave the lowest $c_u/\sigma_1'{}_0$ ratio at a given OCR. This is consistent with the study carried out on intact residual soil of Bukit Timah granite, which presented in Chapter 5. It is also consistent with many other previous studies (Ladd & Lambe, 1963; Ladd et al., 1980; Wanatowski & Chu, 2008; Mayne et al., 2009). Strength anisotropy was again observed in terms of $c_u/\sigma_1'{}_0$.

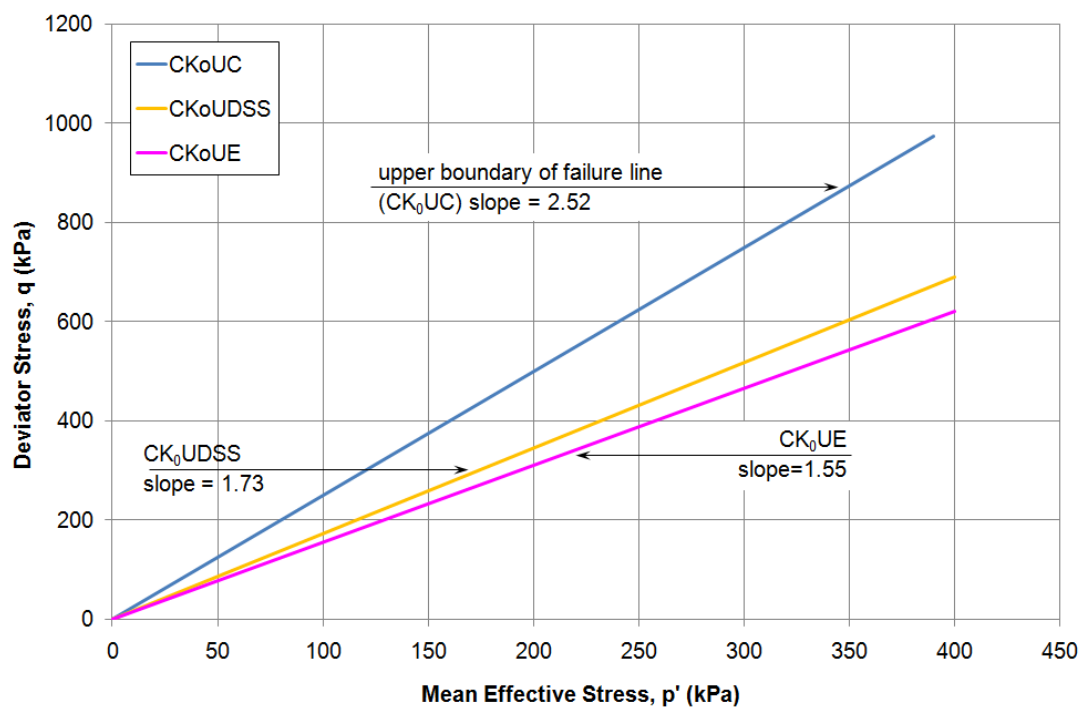


Figure 6.10 Comparison of Failure Envelopes Determined from Different Tests on Artificially Cemented Soil

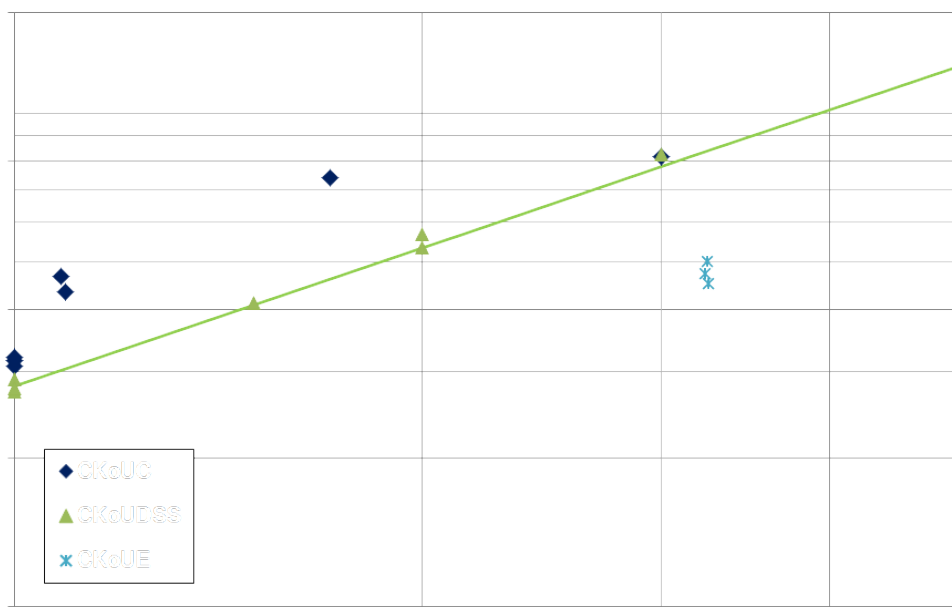


Figure 6.11 Comparison of c_u/σ'_{10} vs. OCR Relationships

6.5.2 Comparison between Intact Residual Soil and Artificially Cemented Soil

A comparison of shear strength behaviour between intact residual soil of Bukit Timah granite (will be referred as IRS in this section) and artificially cemented soil (ACS) was carried out in this section. The failure envelopes and c_u/σ'_{10} vs. OCR relationships were compared in Figure 6.12. It is important to highlight that the failure envelope for ACS is highly dependent of OCR as shown in Figure 6.12(a). Under CK_0UC testing conditions, it is observed that the IRS reached a much lower failure envelope than the ACS. In another word, IRS has a much smaller angle of friction than ACS. This is highly due to the weak cementation of IRS comparing to ACS. Similar observation was made under both CK_0UDSS and CK_0UE testing conditions as presented in Figure 6.12(b) and (c). The failure envelope determined from CK_0UC , CK_0UDSS and CK_0UE tests were plotted together in Figure 6.12 (d). Therefore, the relative locations of failure envelope could be identified.

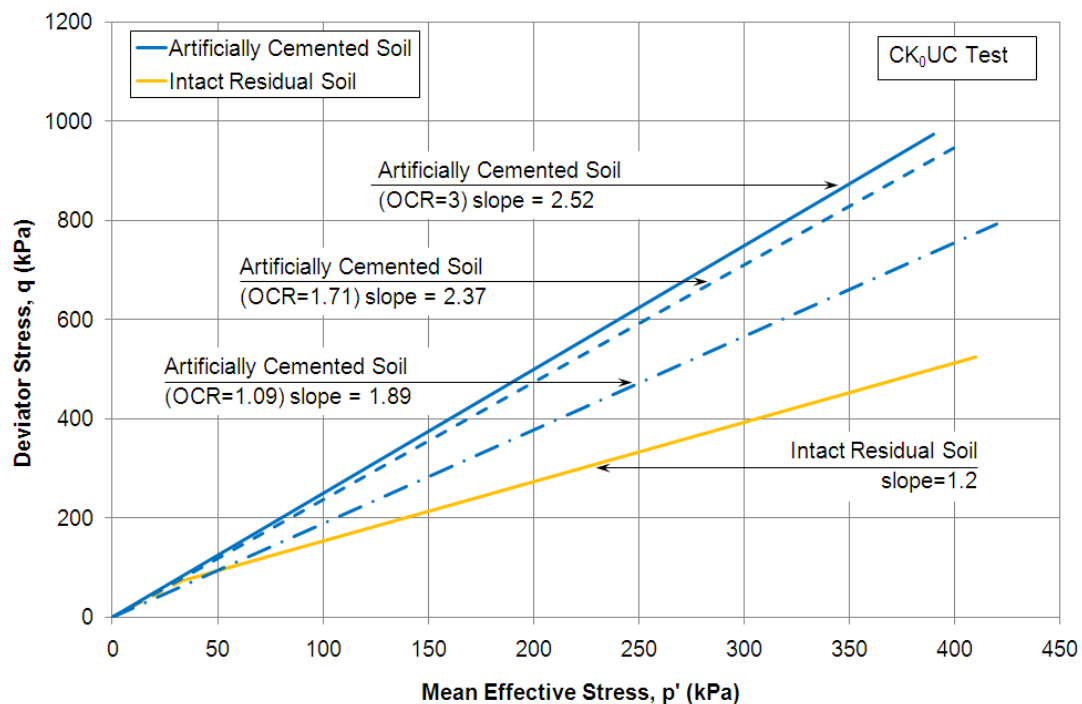


Figure 6.12 (a)

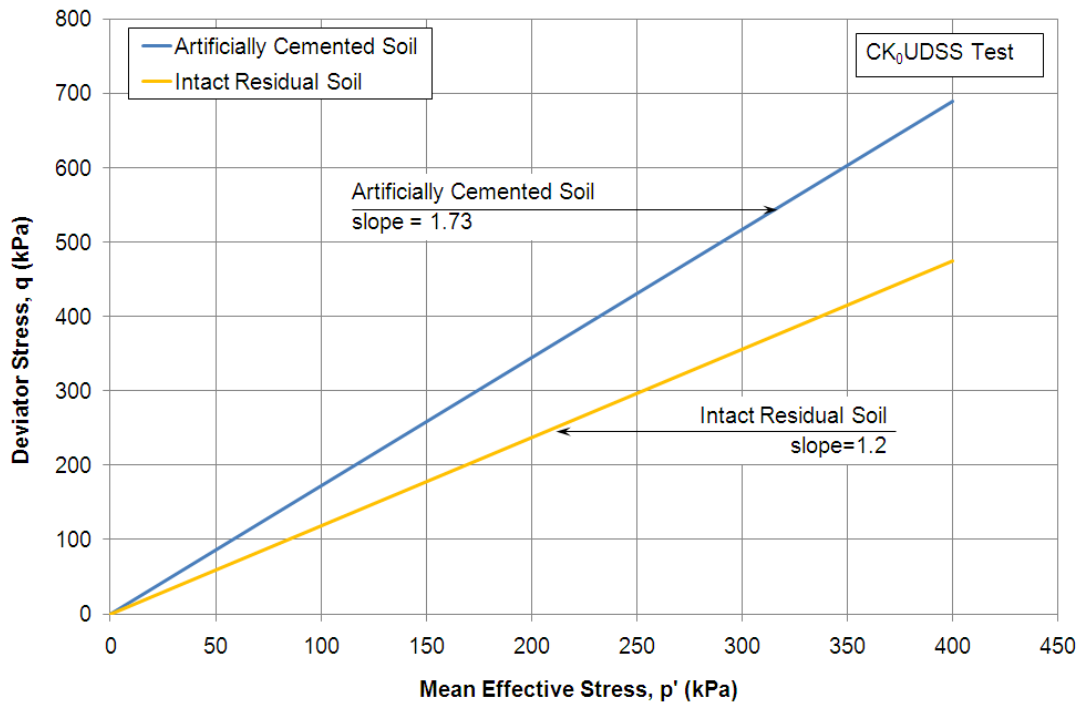


Figure 6.12 (b)

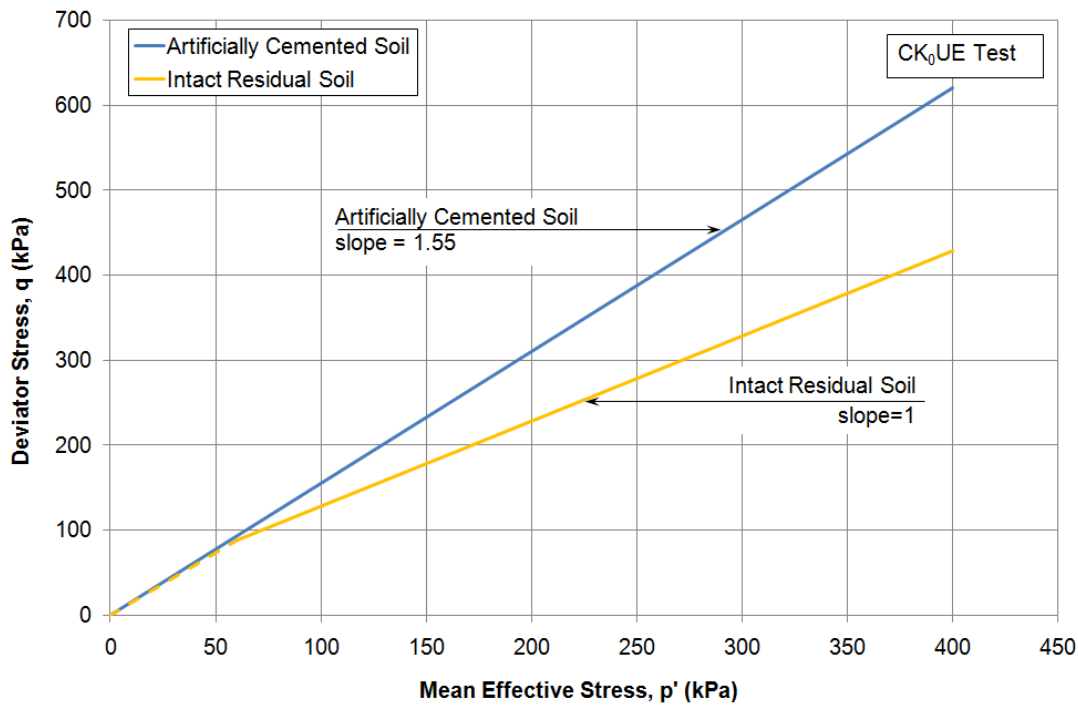


Figure 6.12 (c)

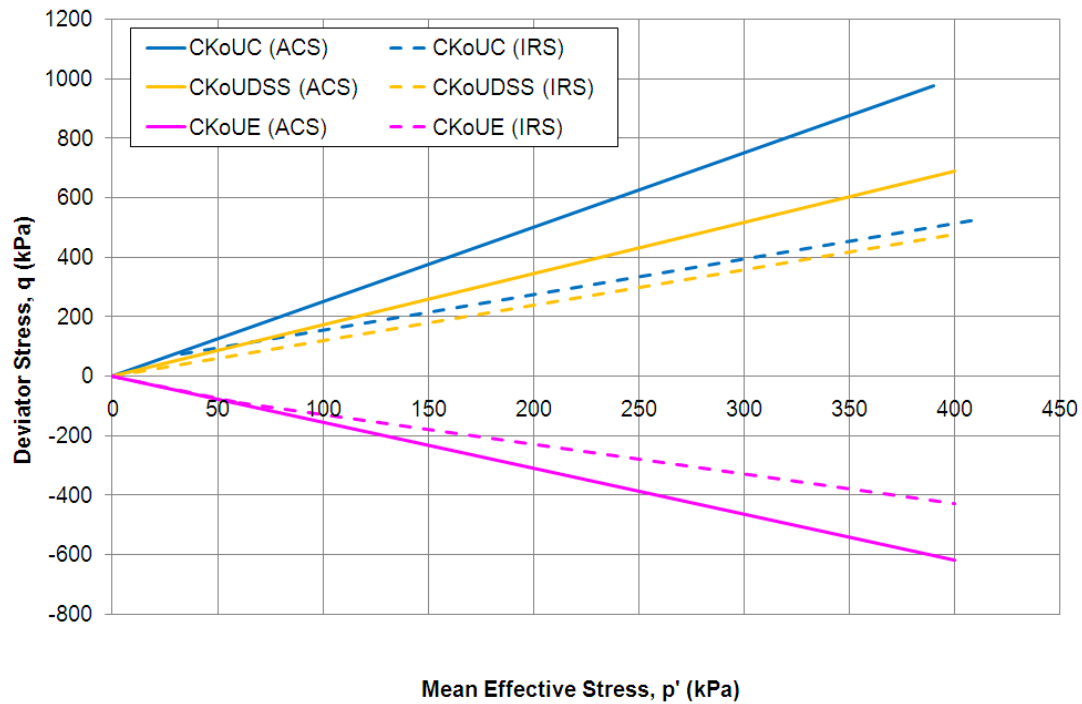


Figure 6.12 (d)

Figure 6.12 Comparisons of Failure Envelopes Determined for Intact Residual Soil and Artificially Cemented Soil: (a) CK₀UC Tests; (b) CK₀UDSS Tests; (c) CK₀UE Tests; (d) Overall View

Another comparison is carried out to investigate the effect of soil cementation on the c_u/σ_{10}' vs. OCR relationships. As seen from Figure 6.13(a), under CK₀UC conditions, the ACS achieved a higher c_u/σ_{10}' than IRS, especially at low OCR region. Since the tests performed in this study is only up to an OCR value of 3, the observation need to be verified at high OCR state. A better interpretation can be made from the CK₀UDSS results as presented in Figure 6.13 (b). The testing data obtained for both ACS and IRS can fit into linear relationships as shown on the plot. It is clearly observed the c_u/σ_{10}' vs. OCR relationship for ACS is higher than that for IRS. It is an explicit indication that the stronger soil cementation can lead to higher value shear strength. However, due to the limited number of tests conducted, there is no conclusive remark can be made under extension conditions as shown in Figure 6.13(c). Future studies on the relevant topics are recommended.

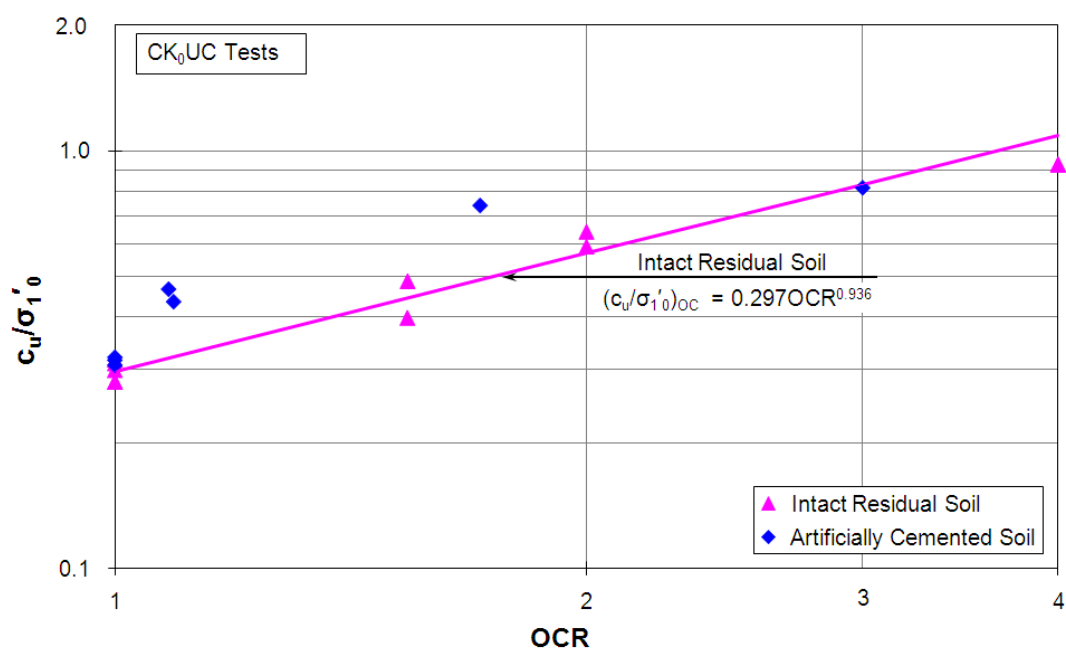


Figure 6.13 (a)

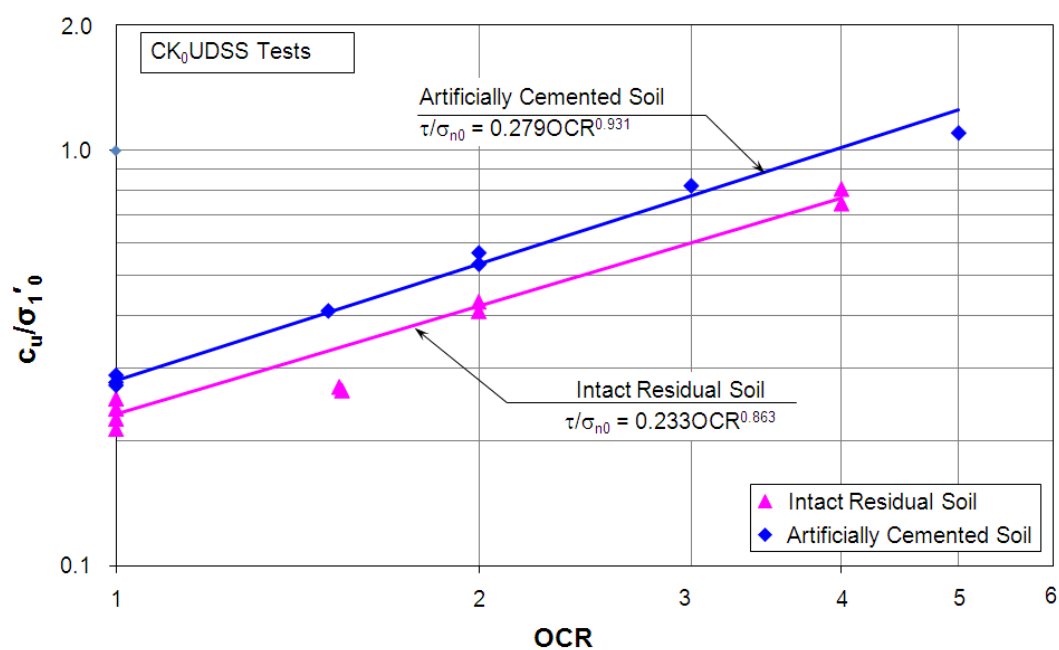


Figure 6.13 (b)

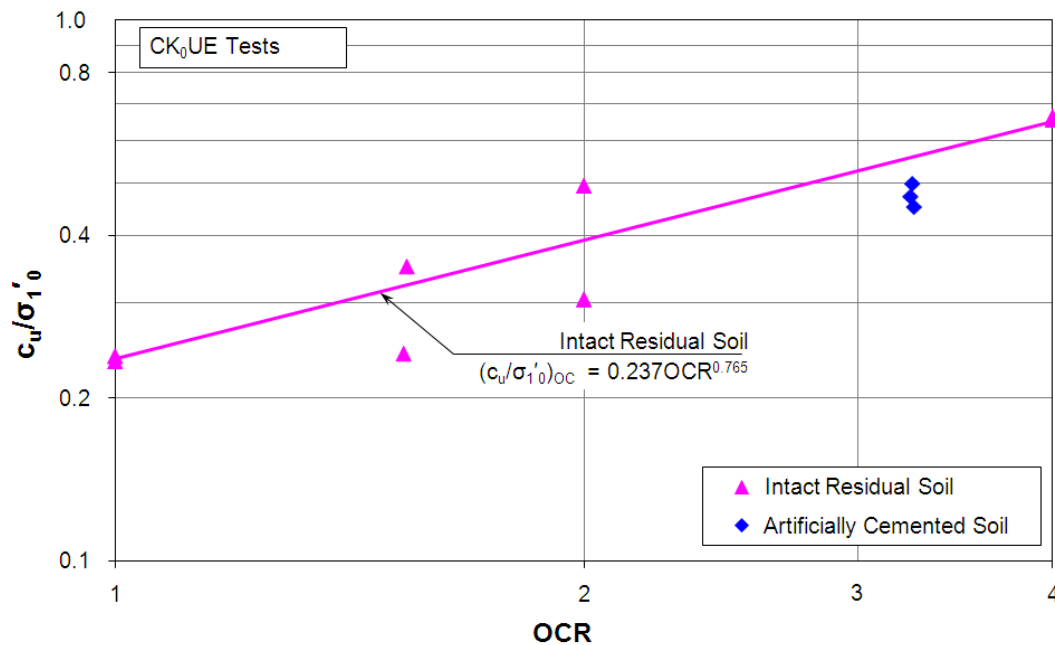


Figure 6.13 (c)

Figure 6.13 Comparison of c_u/σ'_{10} vs. OCR Relationships Determined for Intact Residual Soil and Artificially Cemented Soil: (a) CK_0UC Tests; (b) CK_0UDSS Tests; (c) CK_0UE Tests

6.5.3 Conclusions

To study the cementation effect on the shear strength of soil, an artificially cemented soil was tested under different conditions. By carrying out CK_0UC tests, it is observed that the failure envelope is strongly affected by the OCR value. The slope of failure envelope increased with OCR. The slope can be as high as 2.52 for specimen with OCR value of 3, which resulted in a friction angle of 62.5° . This is highly due to the strong cementation of the artificially cemented soil. After comparing with the results of CK_0UDSS and CK_0UC tests, it is noted the CK_0UC tests resulted in the highest failure envelope, followed by the CK_0UDSS test. CK_0UC tests gave the lowest one. Similar observation was made on the c_u/σ'_{10} vs. OCR relationships also. It is a clear indication of strong strength anisotropy behaviour in terms of failure envelope and c_u/σ'_{10} .

By comparing the behaviour of the artificially cemented soil and the intact residual soil of Bukit Timah granite, it is found that the shear strength can be greatly affected by soil cementation. The failure envelope of artificially cemented soil is much higher than that of intact residual soil, under all tested conditions (CK₀UC, CK₀UDSS and extension conditions). It may also be concluded that the strong cementation of artificially cemented soil led to a higher value of $c_u/\sigma_1'{}_0$. However, due to the limited number of tests conducted, future studies are suggested on the relevant topics to verify the cementation effects on the $c_u/\sigma_1'{}_0$ vs. OCR relationships.

CHAPTER 7

CONSTANT-SHEAR-DRAINED (CSD) TESTS ON RESIDUAL SOILS AND ARTIFICIALLY CEMENTED SOIL

7.1 Introduction

As one of the oldest and most common engineering materials, compacted soil are normally used in the construction of embankments, fills and road. In this study, the trimming of the intact residual soil was compacted and tested. As mentioned by Chu et al. (2003), slope failures are often caused not only by an increase in external load, but also by a reduction in the effective mean stress, which is because of, for example, water infiltration into slopes and release of lateral stress. As suggested by Brand (1981), when investigating the failure mechanisms of a slope, the stress–strain behaviour of the soil along stress paths that simulate water infiltration should be studied. Such stress paths may be idealized as paths with constant shear stress, but decreasing mean effective stress, or the so-called constant shear-drained (CSD) tests performed under constant deviator stress (Brand, 1981; Anderson and Riemer 1995; Chu et al., 2003). Soil behaviour under this type of stress path has seldom been examined. Therefore, a series of constant-shear-drained (CSD) tests were conducted to investigate the deformation and instability behaviour of the residual soil of Bukit Timah granite. In addition, the artificially cemented soil was tested also along the constant stress path to study the effect of the cementation on instability.

The CSD tests were also favorable as element study to understand the mechanism of hydraulic fracture, which is the known or suspected cause of the excessive leakage and failure of many dams. However, the tensile failure hypothesis is normally used to study the boundary condition of hydraulic fracture with water flow inside soil specimens. To simulate the hydraulic fracture inside earth dam by using constitutive models, the behaviour of soil elements must be investigated thoroughly, especially under plane-strain condition. It should also point out that the true cohesion of most of clay, including residual soil, is very small. Therefore, the shear failure criteria of hydraulic fracture are more relevant in most of the cases.

As described by Anderson & Riemer (1995) and Anderson & Sitar (1995), during the CSD test, the effective confining pressure is gradually reduced while the shear stress is held constant. A CSD test condition can be achieved in two ways. The first is to shear a specimen along a constant shear path by increasing the back pressure (or pore water pressure, PWP) while maintaining the confining pressure (σ_3) constant. This testing condition is consistent with the one for hydraulic fracture. The second is to decrease the confining pressure while maintain the back pressure (or PWP) constant. It should be pointed out that although the back pressure or PWP may change during CSD test, it is still considered as a “drained” test, since the volume of the specimen is changing. Several studies (Leong, 2001; Wanatowski, 2005) have proved that there is no difference between the two types of testing methods. A comparison of the two testing methods was also made in this study under plane-strain conditions (as presented in Section 7.2.2).

7.2 CSD Tests on Compacted Residual Soil

7.2.1 Basic Stress-Strain Behaviour at NC State

Before carrying out the CSD tests, the general undrained behaviour of the compacted residual soil at NC state was studied first by conducting CK₀UC tests. The tests conducted are summarized in Table 7.1. The general undrained behaviour is presented in Figure 7.1.

Table 7.1 Summary of CK_0UC Tests Conducted on Compacted Residual Soil Specimens

Test No.		CGN1	CGN2
Pressure for Saturation (kPa)	Cell	860	860
	Back	850	850
OCR		1	1
Shearing Rate (mm/min)		0.005	0.005
Weight (g)	Pre-Test	381.25	379.69
	Post-Test	378.79	377.48
Water Content (Pre-Test) (%)	Average	24.32	23.94
Water Content (Post-Test) (%)	Average	23.08	22.74
	Top	22.94	22.58
	Middle	23.08	22.81
	Bottom	23.22	22.83
σ'_{10} (kPa)		786.94	958.76
q_0 (kPa)		434.94	534.76
p'_0 (kPa)		496.98	602.25
e_0		0.612	0.589
q_{peak} (kPa)		470.26	575.35
ε_1 at q_{peak} (%)		0.662	0.663
$(q/p')_{peak}$		1.356	1.347
$R_{peak} = (\sigma'_1/\sigma'_3)_{peak}$		5.357	8.083
ε_1 at $(q/p')_{peak}$ (%)		3.476	3.446
Secant Friction Angle, ϕ' (°)		33.58	33.38

Two specimens were anisotropically consolidated to mean effective stresses of 497 and 602kPa respectively. Both specimens were normally consolidated. The effective stress paths obtained from CK_0UC tests are presented in Figure 7.1(a). A straight failure envelope with a slope of 1.35 was determined by using $(q/p')_{peak}$ value. The deviator stress reached the peak at very small axial strain (about 0.7%), followed by an insignificant strain softening (as shown in Figure 7.1(b)).

The peak deviator stress increased as effective confining stress increased. Figure 7.1(c) shows the variations of q/p' with axial strain during CK_0UC tests. A typical NC behaviour was observed. The q/p' approached the peak value of 1.35 at about

3.5% of axial strain, and it remained constant afterwards. The stress-strain curves were normalized by the initial effective vertical stress (σ'_{10}) in Figure 7.1(d). The consistency of the normalized stress-strain curves confirmed the normalized behaviour for NC compacted residual soil. The generation of excess pore water pressure (Δu) during undrained shearing is presented in Figure 7.1(e). The Δu value reached the peak first, then maintained at the constant level. Generation of excess pore water pressure increased with effective confining stress.

As reviewed in Chapter 2, Lade (1993) defined the instability line as a line connecting the peak of effective stress paths obtained from undrained tests. When the stress state is located on or above the instability line, the soil material becomes unstable. Numerous experiments have shown that the instability line is straight and passes through the origin of the stress diagram (Lade, 1992, 1994, 1999; Leong et al., 2000; Chu et al., 2003; Wanatowski, 2005). Based on the peak deviator stresses shown in Figure 7.1(b), an instability line (IL) with a slope of 1.17 was obtained for the compacted residual soil as shown in Figure 7.1(b). By comparing with the CK₀UC tests conducted on NC intact residual soil specimens in Figure 7.4, the same IL was achieved.

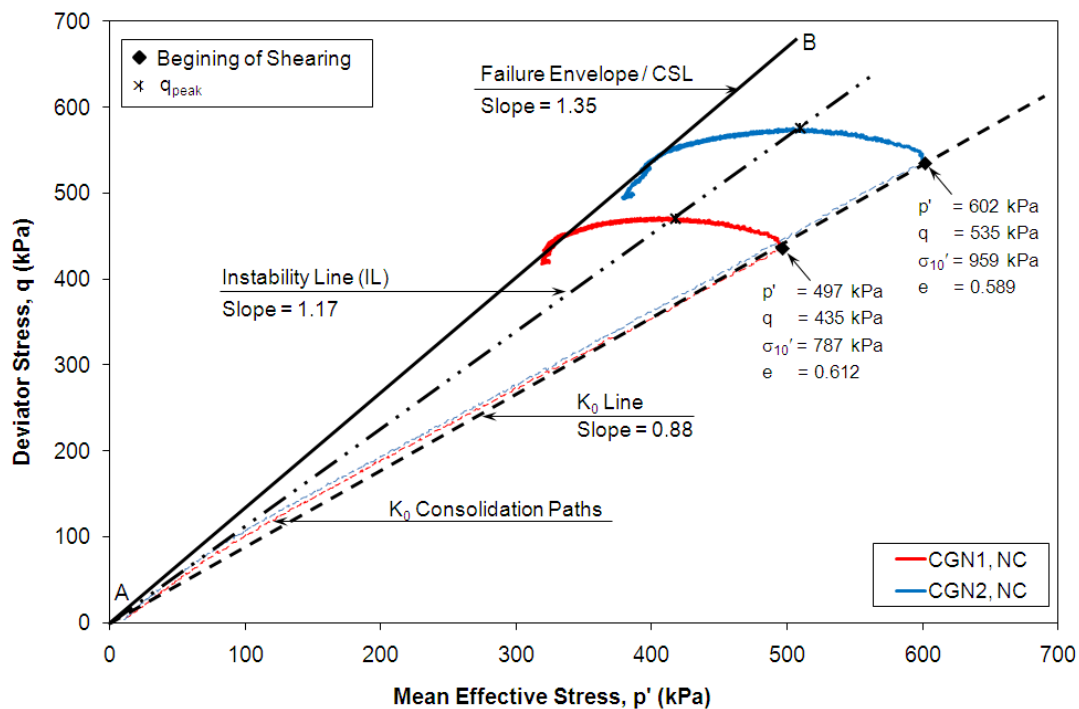


Figure 7.1(a)

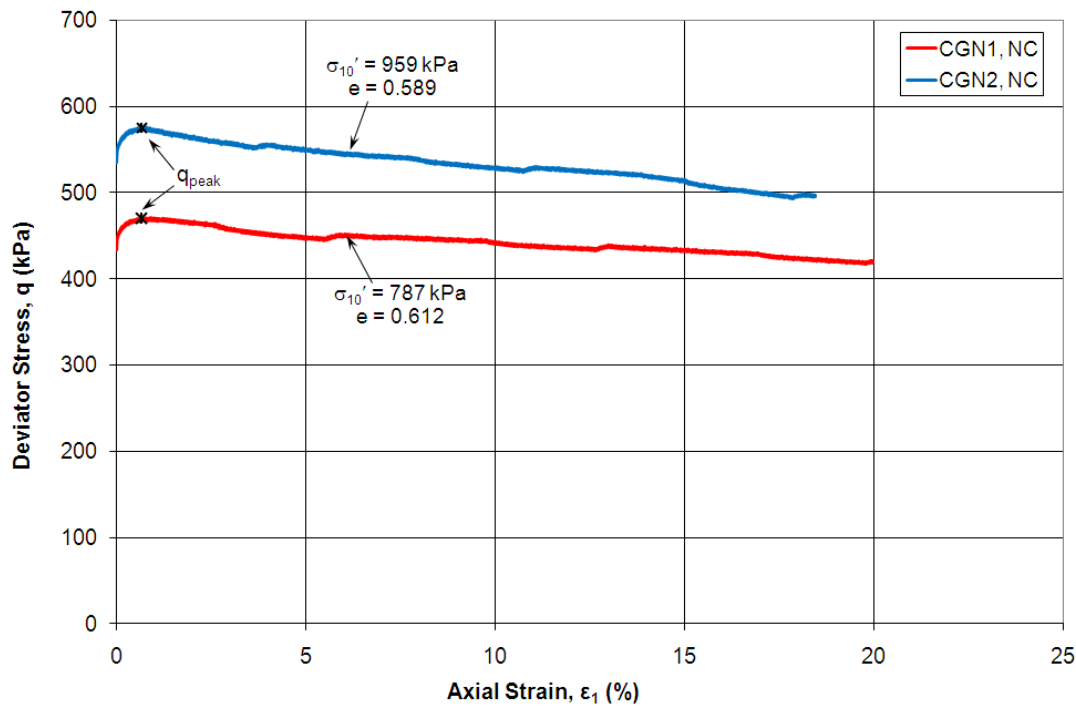


Figure 7.1(b)

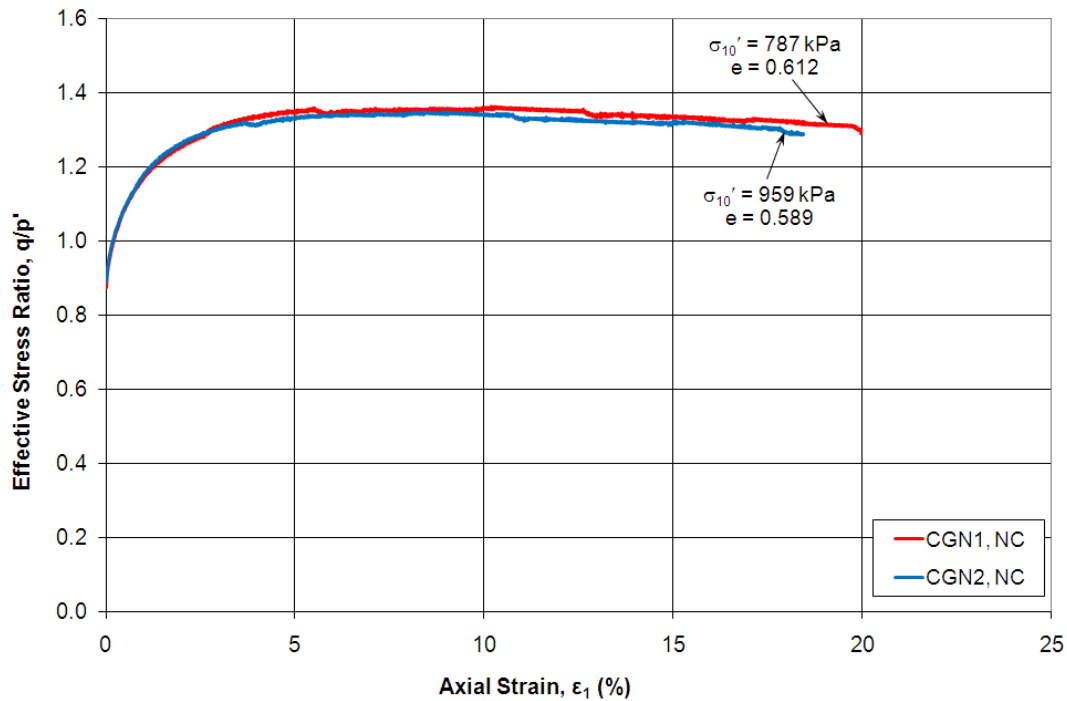


Figure 7.1(c)

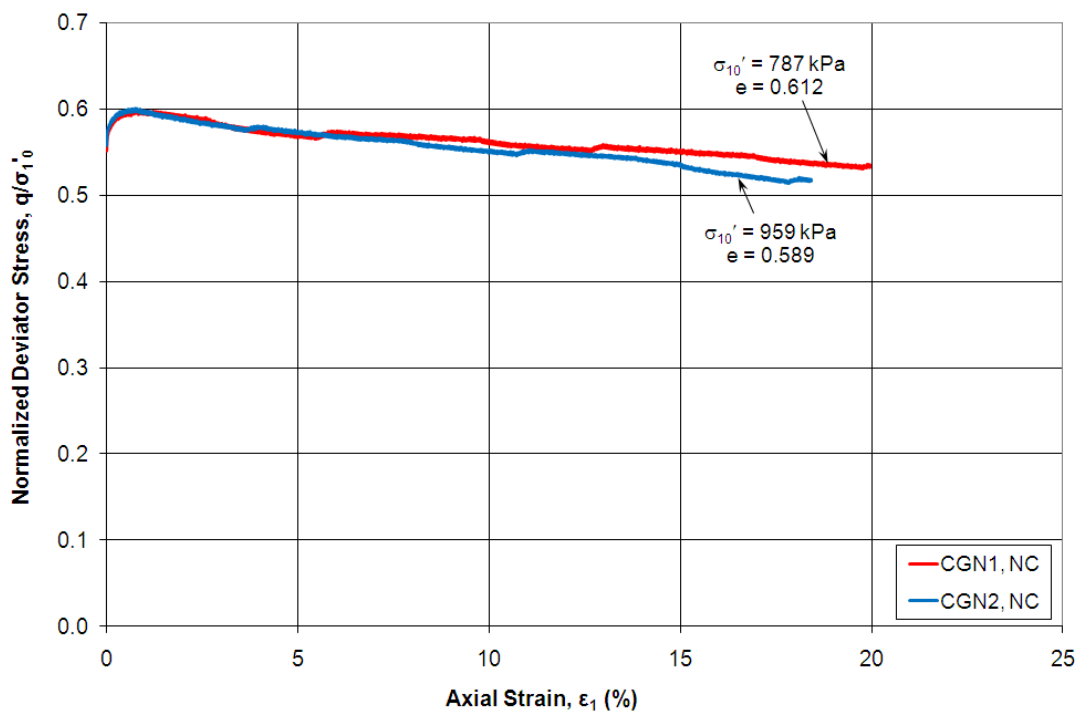


Figure 7.1 (d)

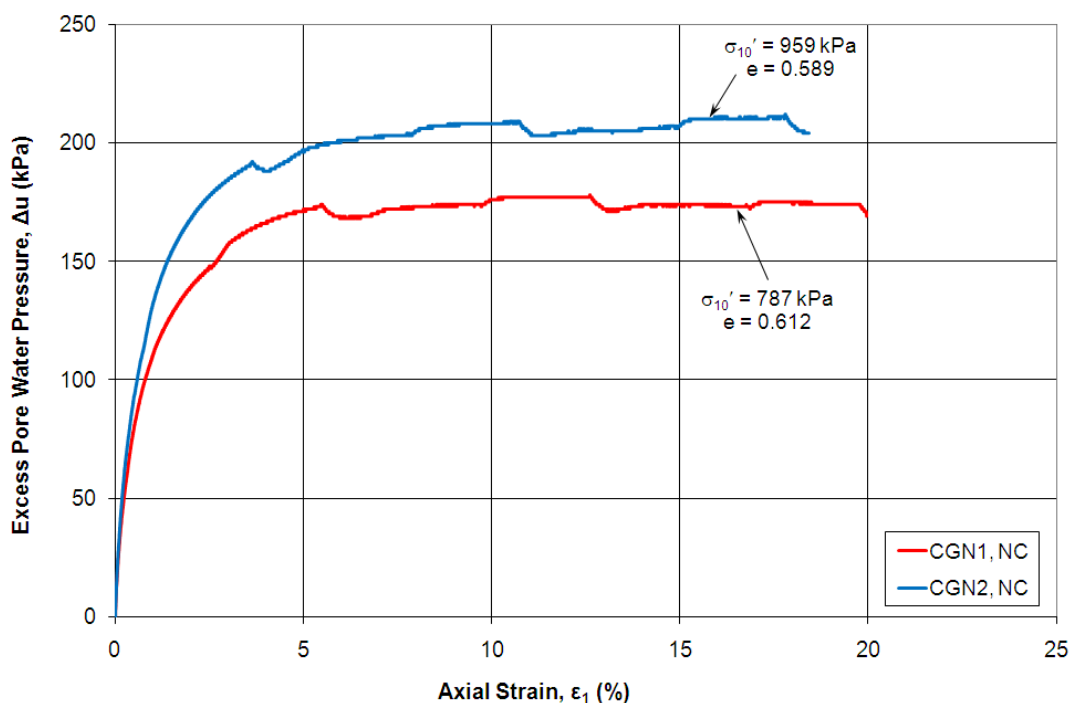


Figure 7.1(e)

Figure 7.1 Undrained Behaviour of the Compacted Residual Soil from CK_0UC Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves; (c) q/p' vs. $\epsilon_1\%$ curves; (d) Normalized Stress-Strain Curves; (e) Δu vs. $\epsilon_1\%$ curves

Same as the test results presented in Chapter 5, the effective stress paths determined from CK₀UC tests on the compacted residual soil specimens were normalized against a coupled stress as proposed by Yao & Zhu (2009), which can be calculated as follow:

$$T_0 = M^2 p'_0 + q_0^2 / p'_0 \quad (7-1)$$

where M is the slope of the critical state line.

As shown in Figure 7.2, the normalized effective stress paths were coincided for both tests. A unique failure envelope (or CSL) was obtained for NC specimens. The Skempton Parameter A was calculated as $A = \Delta u / \Delta q$, and presented in Figure 7.3. The A value increased as axial strain increased. It was consistent for both tests conducted on NC specimens.

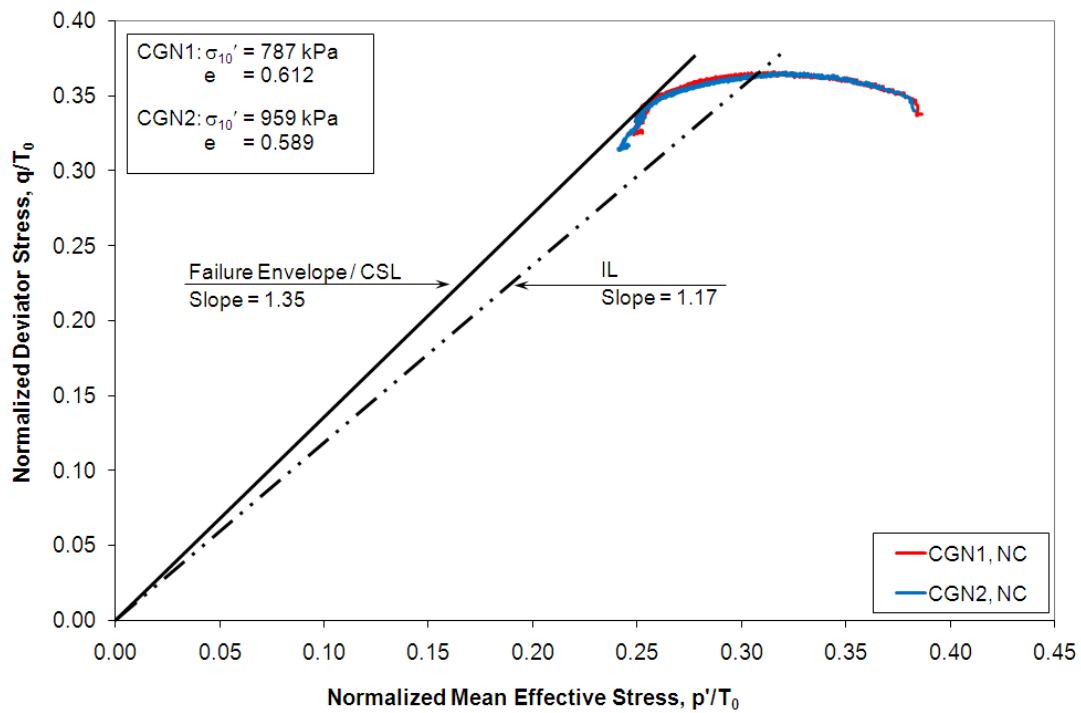


Figure 7.2 Normalized Stress Paths of the Compacted Residual Soil from CK₀UC Tests

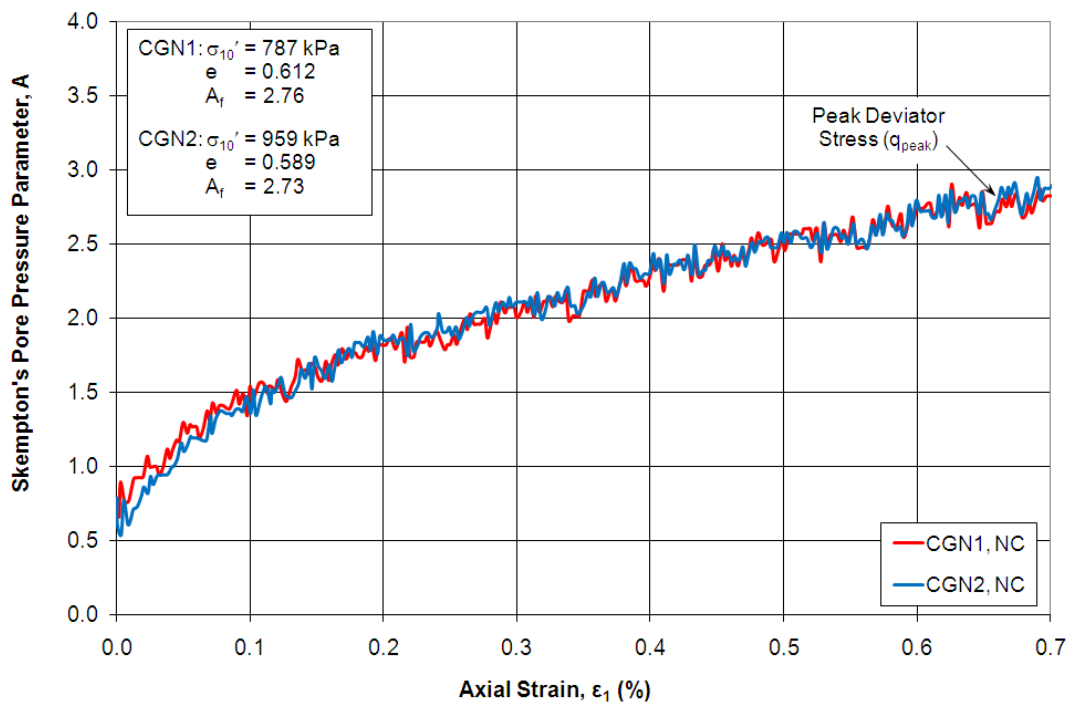


Figure 7.3 Variations of Skempton's Pore Pressure Parameter, A, during CK_0UC Tests on Compacted Residual Soil

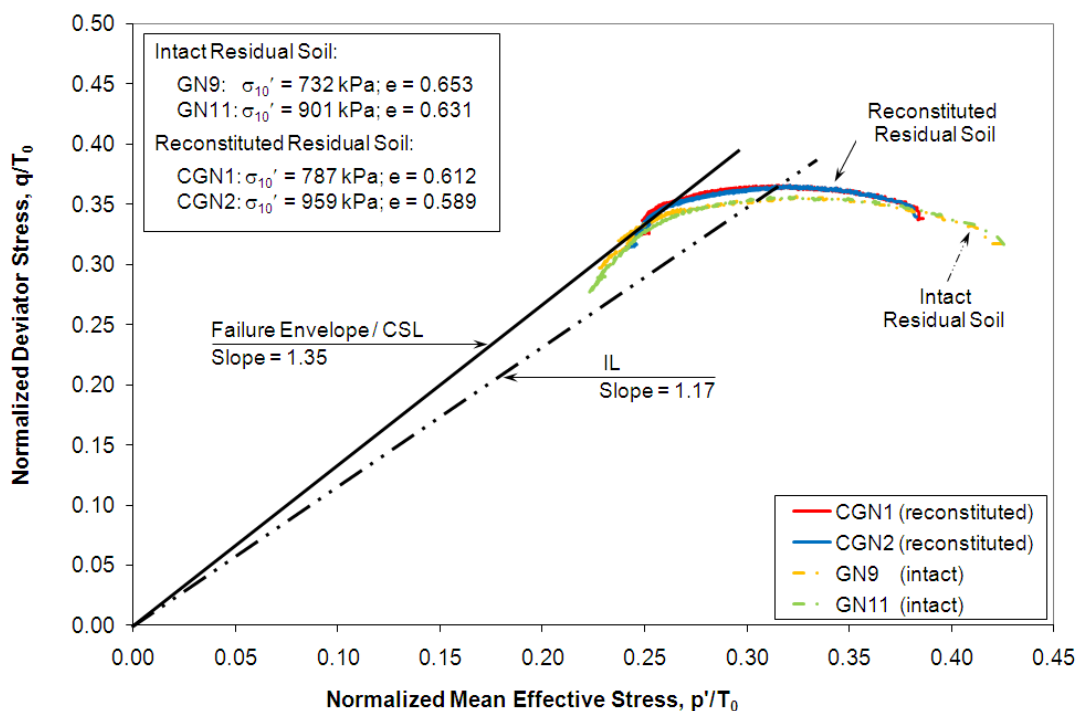


Figure 7.4 (a)

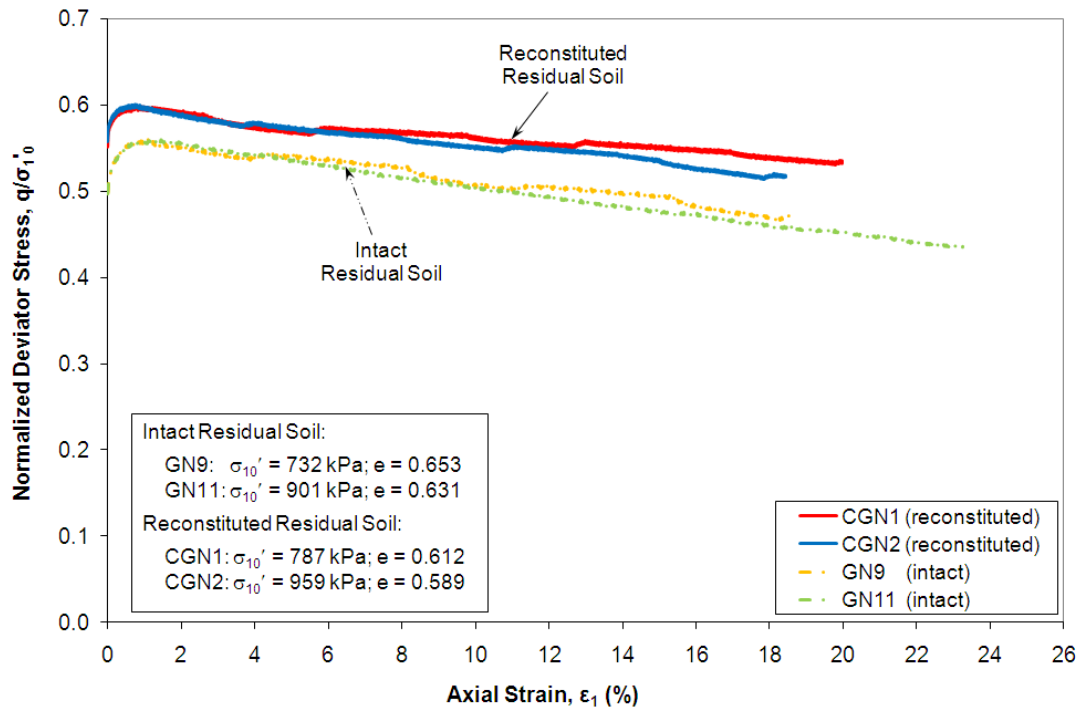


Figure 7.4 (b)

Figure 7.4 Comparison of the CK_0UC Test Results of the Intact and Compacted Residual Soil: (a) Normalized Effective Stress Paths; (b) Normalized Stress-Strain Curves

7.2.2 Comparison of Testing Methods for CSD Tests

As mentioned in Section 7.1, a constant-shear-drained (CSD) testing condition can be achieved in two ways. The first is to shear a specimen along a constant shear path by increasing the back pressure (or pore water pressure, PWP) while maintaining the confining pressure (σ_3) constant. The second is to decrease the confining pressure while maintain the back pressure (or PWP) constant. Several studies (Leong, 2001; Wanatowski, 2005) have proved that there is no difference between the two types of testing methods. A comparison of the two testing methods was also made for compacted residual soil in this study under plane-strain conditions. The test results are presented in Figure 7.5.

As shown in Figure 7.5(a), two different testing procedures were used to impose a constant shear path. In test HFGN2, the confining stress (σ_3) was reduced with pore

water pressure kept constant ($\Delta u=0$); whereas in test HFGN4, the confining stress maintained constant ($\Delta \sigma_3=0$) while the pore water pressure was increased. A constant changing rate of 0.5kPa/min was used in both tests.

The effective stress paths determined from the above two tests are presented in Figure 7.5(b). Due to the reduction of the effective confining stress, the effective stress paths moved horizontally towards the failure envelope. The gradual deviator stress reduction is due to the gradual increase in the cross-sectional area of the specimen. Both specimens reached the same failure envelope with a slope of 1.4. When the failure state is reaching, the deviator stress dropped rapidly, and the axial strain increased abruptly as shown in Figure 7.5(c) and (d). The effective stresses measured during the two tests are plotting against time in Figure 7.5(e). It can be seen that the effective stress variations were consistent between two different testing methods. The largest difference observed was less than 10% of the corresponding effective stress (σ_2' before failure state). The method of reducing the confining pressure while maintaining the back pressure (or PWP) constant was adopted in this study. Because this method resulted in more uniformly distributed pore water pressure inside soil specimen than the other method.

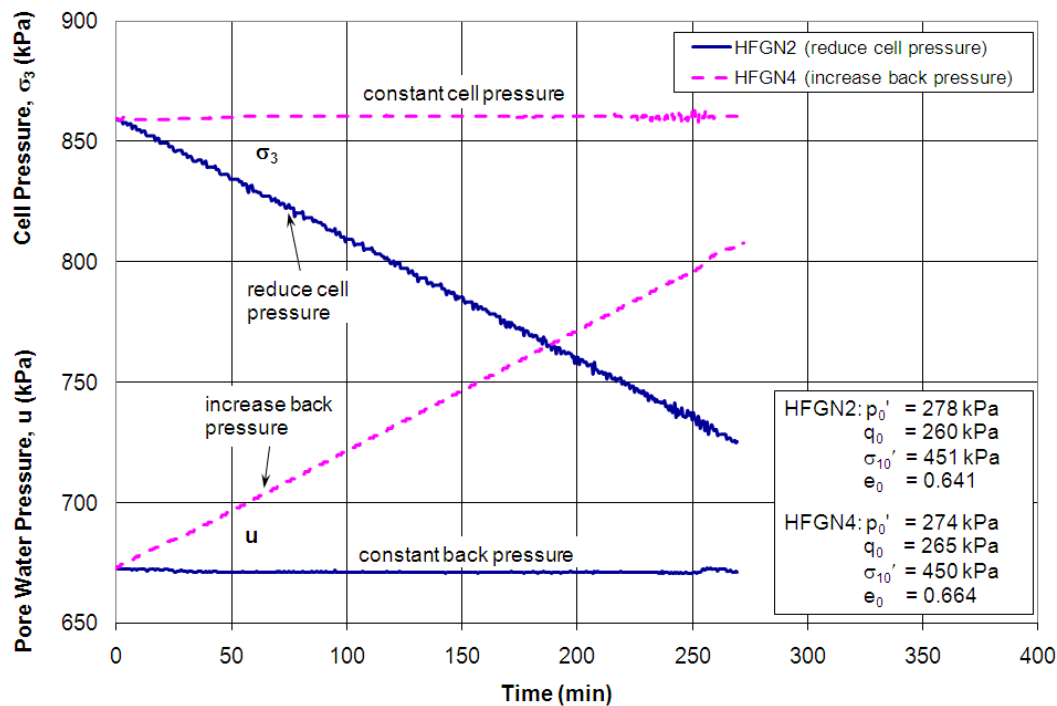


Figure 7.5(a)

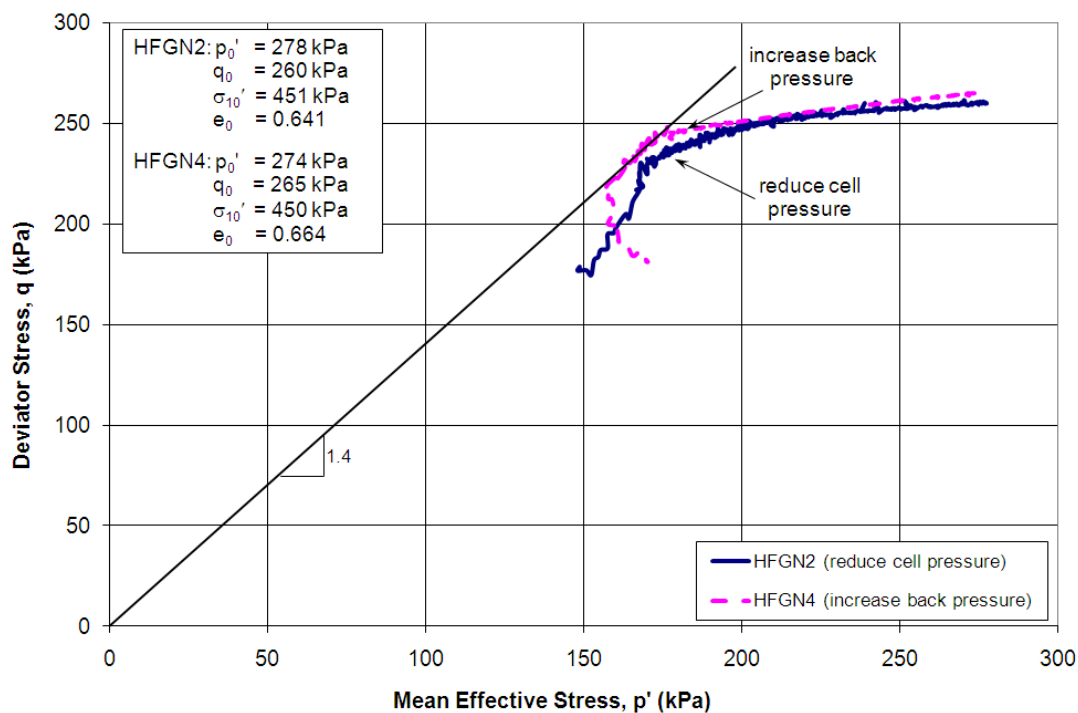


Figure 7.5(b)

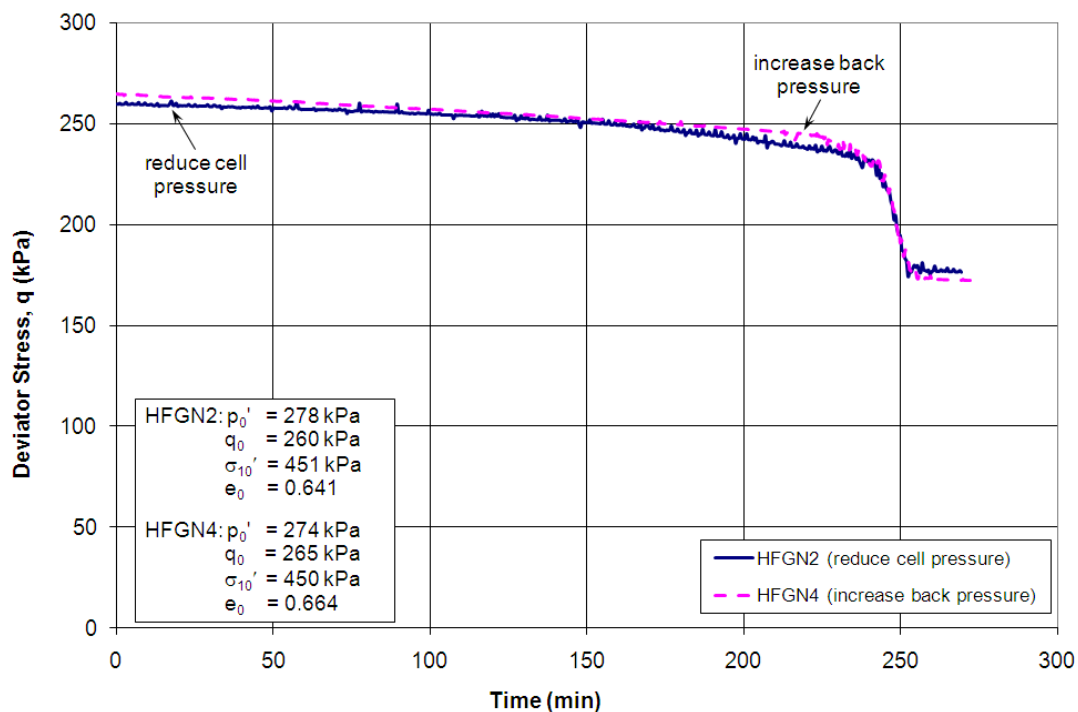


Figure 7.5(c)

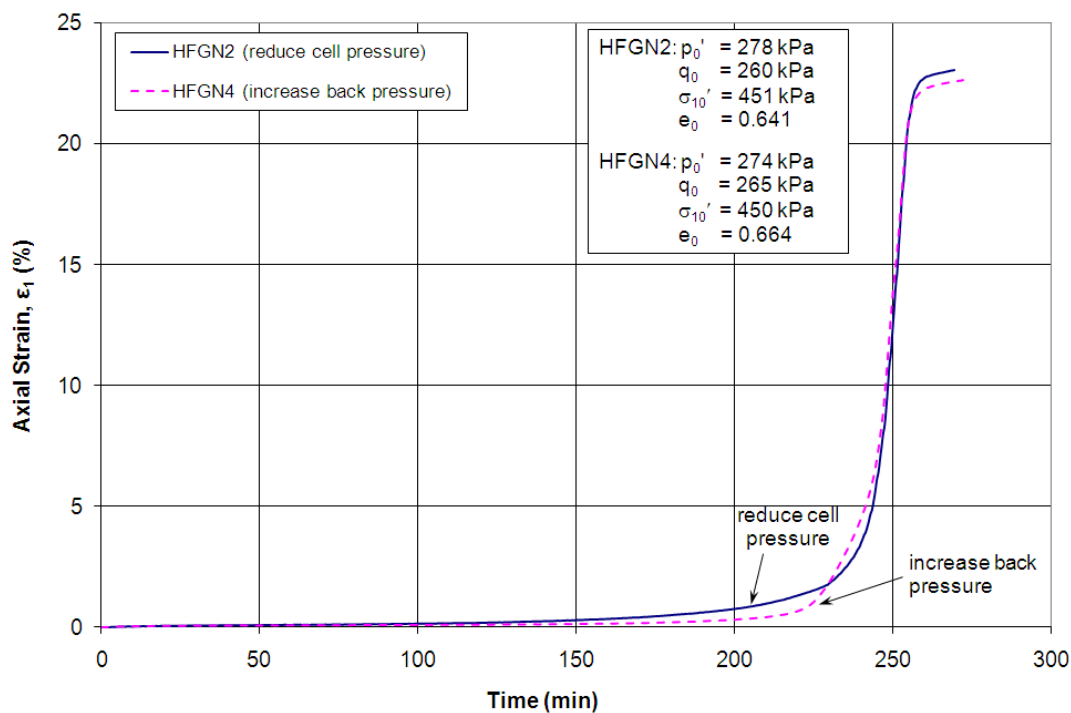


Figure 7.5 (d)

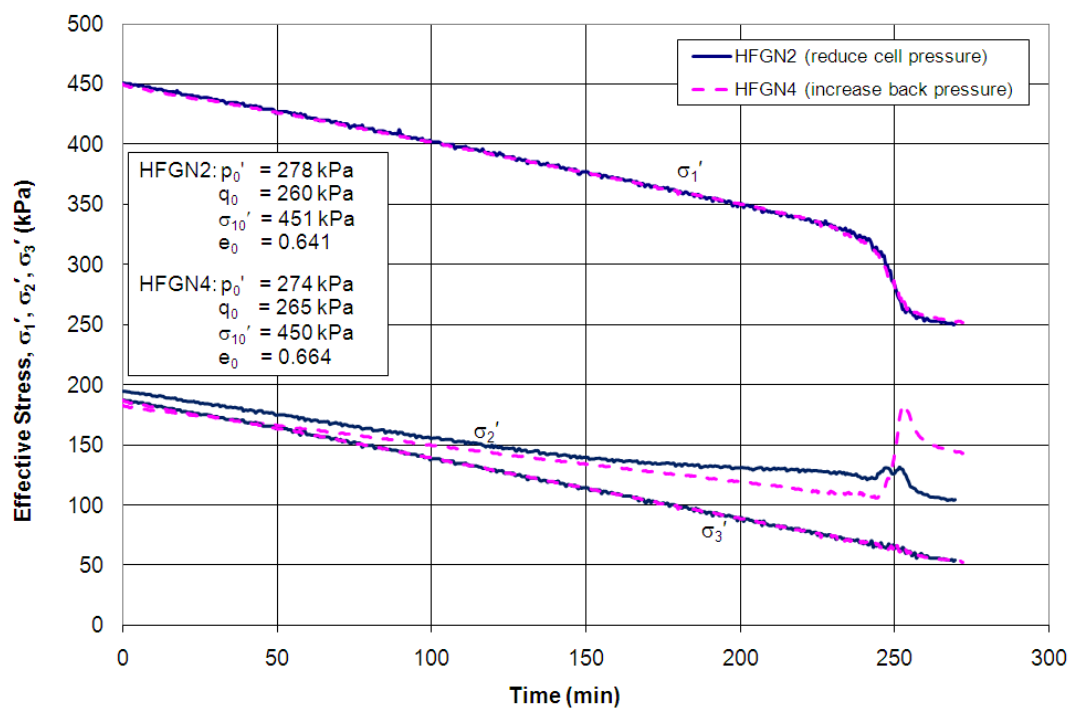


Figure 7.5(e)

Figure 7.5 Comparison of Two Testing Methods: (a) σ_3 and Δu vs. Time Curves; (b) Effective Stress Paths; (c) q vs. Time Curves; (d) $\epsilon_1\%$ vs. Time Curves; (e) Variations of Effective Stresses

7.2.3 Tests under Axisymmetric Conditions

The instability behaviour of the compacted residual soil was studied under triaxial conditions first. Three constant-shear-drained tests conducted by using a triaxial cell (will be referred as CSD-T tests) are summarized in Table 7.2. The results of three CSD-T tests are presented in Figure 7.6. The specimen TXLQ4 was normally consolidated. The other two specimens (TXLQ1 and TXLQ3) were anisotropically consolidated to OCR of 1.17 and 2.66 respectively. Same consolidation method as described in Chapter 4 was adopted. The specimens were then sheared under drained condition by reducing the cell pressure (σ_3) at a constant rate of 0.5kPa/min, while maintaining the back pressure constant.

Table 7.2 Summary of CSD-T Tests on Compacted Residual Soil Specimens

Test No.		TXLQ1	TXLQ3	TXLQ4
Pressure for Saturation (kPa)	Cell	860	860	860
	Back	850	850	850
OCR		1.17	2.66	1
Weight (g)	Pre-Test	378.94	377.51	354.46
	Post-Test	386.9	-	374.69
Water Content (Pre-Test) (%)	Average	22.94	22.69	21.07
Water Content (Post-Test) (%)	Average	25.82	31.28	27.71
	Top	-	31.15	27.53
	Middle	-	31.71	27.86
	Bottom	-	30.99	27.75
σ'_{10} (kPa)		504.8	234.08	319.16
q_0 (kPa)		262.8	59.08	173.16
p'_0 (kPa)		329.6	194.69	203.72
e_0		0.622	0.636	0.672
$(q/p')_{\text{peak}}$		1.525	4.225	1.514
$R_{\text{peak}} = (\sigma'_1/\sigma'_3)_{\text{peak}}$		4.101	-	4.058
ε_1 at $(q/p')_{\text{peak}}$ (%)		7.748	0.239	9.766
Secant Friction Angle, ϕ' (°)		37.44	-	37.20

The onset on instability was marked in all the figures. The instability is defined as a condition in which large plastic strains are generated rapidly due to the inability of a soil element to carry or sustain a give load (Lade et al., 1987, 1988; Lade & Pradel, 1990; Chu et al., 1993; Yamamuro & Lade, 1997), which is based on the yielding condition. It was identified as the point when axial strain (ε_1) started to increase rapidly as shown in Figure 7.6(c). However, the point could be observed more easily from the variation of axial strain rate ($\Delta\varepsilon_1/\Delta t$) presented in Figure 7.6(d).

Figure 7.6(a) presents the effective stress paths obtained from CSD-T tests under triaxial conditions. A failure envelope AC with a slope of 1.28 was determined as a envelope to all the effective stress paths and passed through the points of peak effective stress ratio, $(q/p')_{\text{peak}}$. The critical state line (AD) and instability line (AE) determined from CK₀UC tests as presented in Figure 7.1(a) were also shown for reference. As seen from Figure 7.6(a), instability occurred before reaching the failure envelope for NC specimen (Test TXLQ4). For specimen with high OCR value (TXLQ3), the onset of instability located very close the failure envelope, followed by a significant drop in the deviator stress. Same behaviour can be observed from the variation of effective stress ratio with axial strain presented in Figure 7.6(e). It should be pointed out that test TXLQ3 was conducted at a very low σ_3' value. Due to strain hardening effect, the q/p' kept increasing towards the end of test. The variation of void ratio (e) with effective vertical stress (σ_1') during CSD-T tests are presented in Figure 7.6(f). The soil specimens tended to dilate after the initiation of instability, especially the highly over-consolidated specimen (TXLQ3). This could be due to the tendency for the soil specimen to collapse.

Figure 7.7 presents the effective stress changes during CSD-T tests. It is noted that effective stresses reduced rapidly and the minor effective stress (σ_3') approaches to zero at the end of test TXLQ3. However, instability occurs when effective stresses reached the failure line, and is not controlled by σ_1' or σ_3' .

It should be mentioned that there is no visible shear band observed during the CSD-T tests. However, abrupt increases in volumetric strain and volumetric strain rate were noted at after the onset of instability. This is consistent with the practical criteria of initiation of hydraulic fracture. In laboratory fracture tests, hydraulic fracturing is generally believed to initiate when volumetric strain rate increased abruptly with a constant increment in back pressure (Bjerrum et al., 1972; Bjerrum & Andersen, 1972; Jaworski et al., 1981; Mori & Tamura, 1987; Alfaro & Wong, 2001; Ohta et al., 2007). According to the comparison made in Section 7.2.2, reduce cell pressure during a CSD test is equivalent to increase in back pressure. Abrupt increase in volumetric strain rate together with increase in back pressure could lead to hydraulic fracture inside the compacted residual soil specimen. The tendency for the soil specimen to fracture was also verified by the sudden increase in void ratio. It is indicated the hydraulic fracture of soil may be related to the instability behaviour.

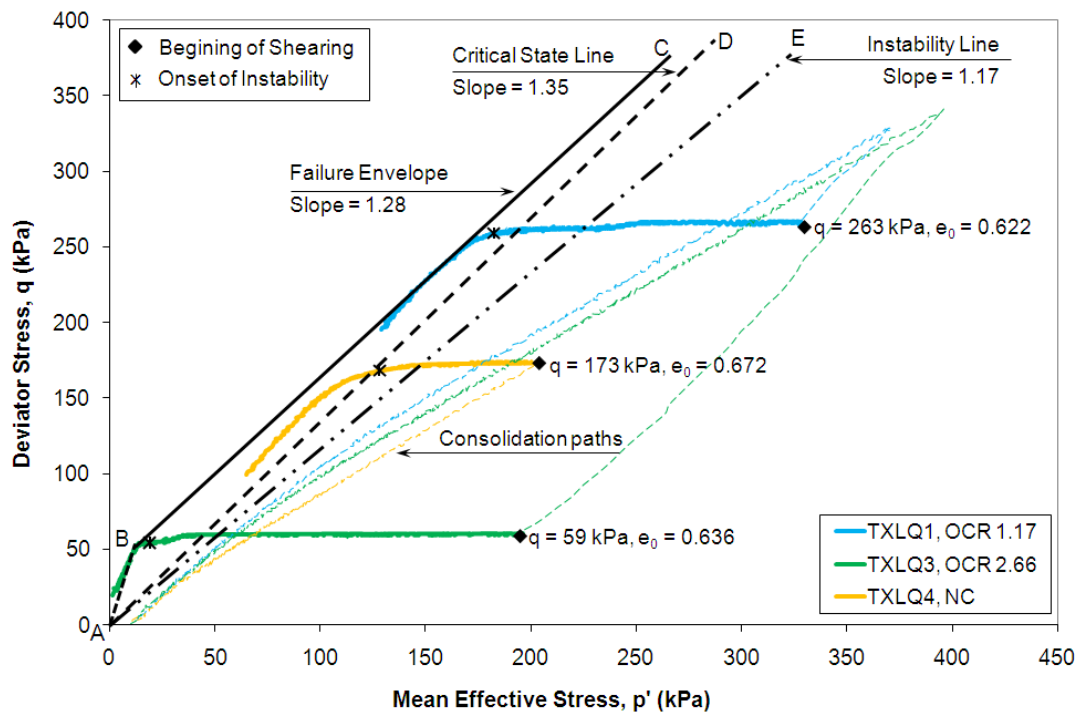


Figure 7.6(a)

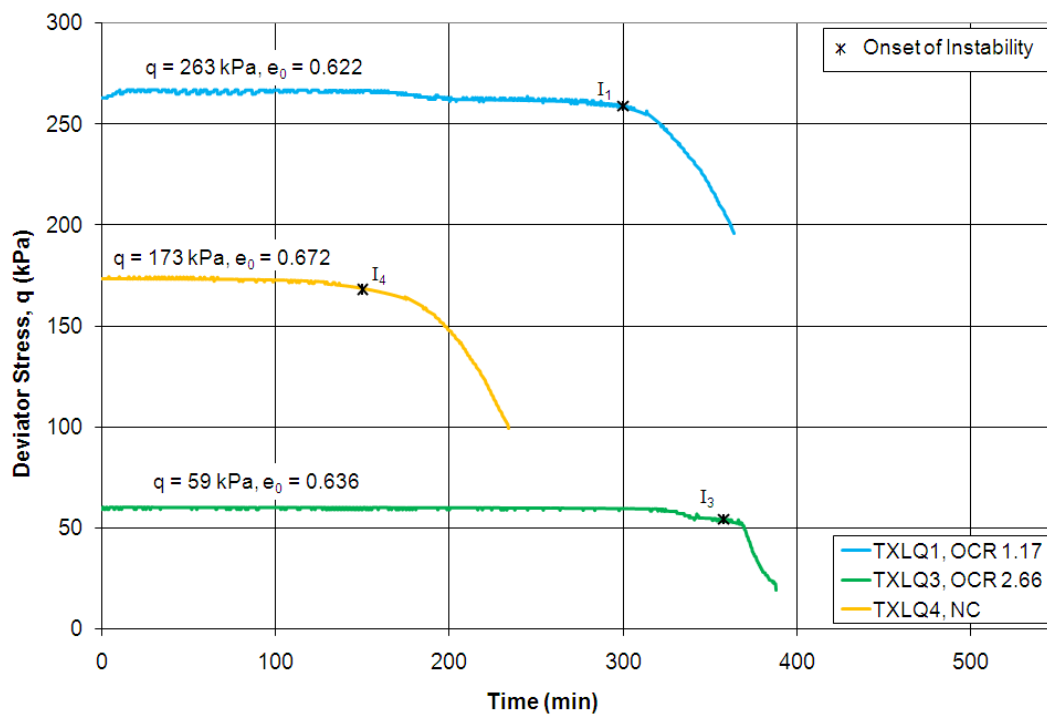


Figure 7.6(b)

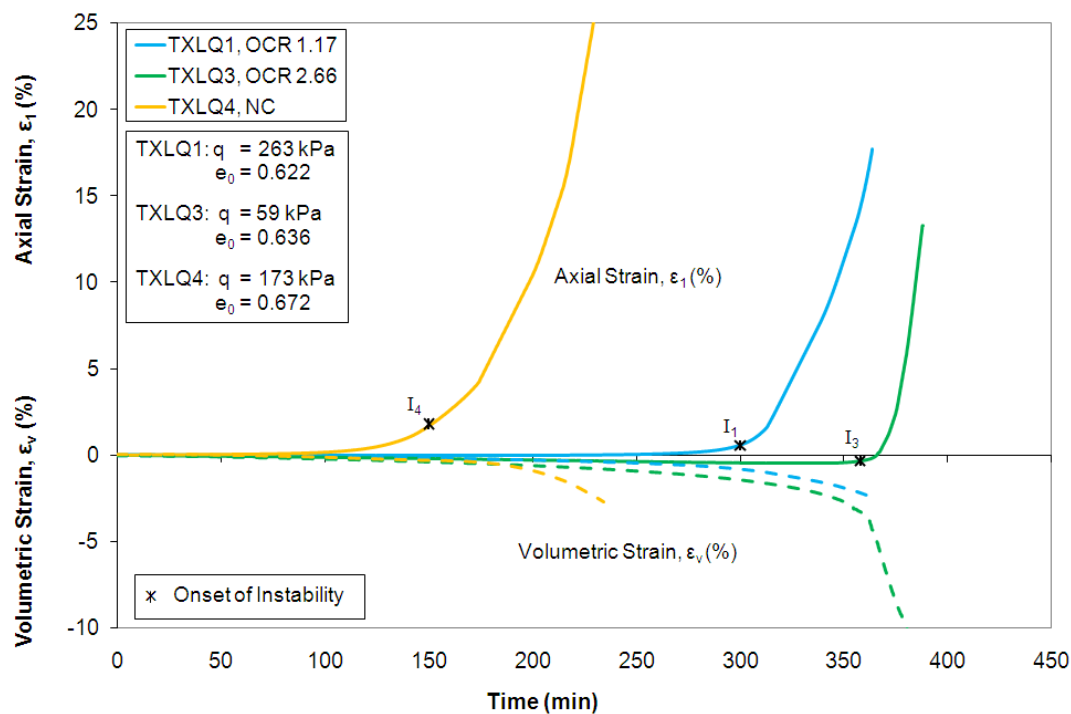


Figure 7.6(c)

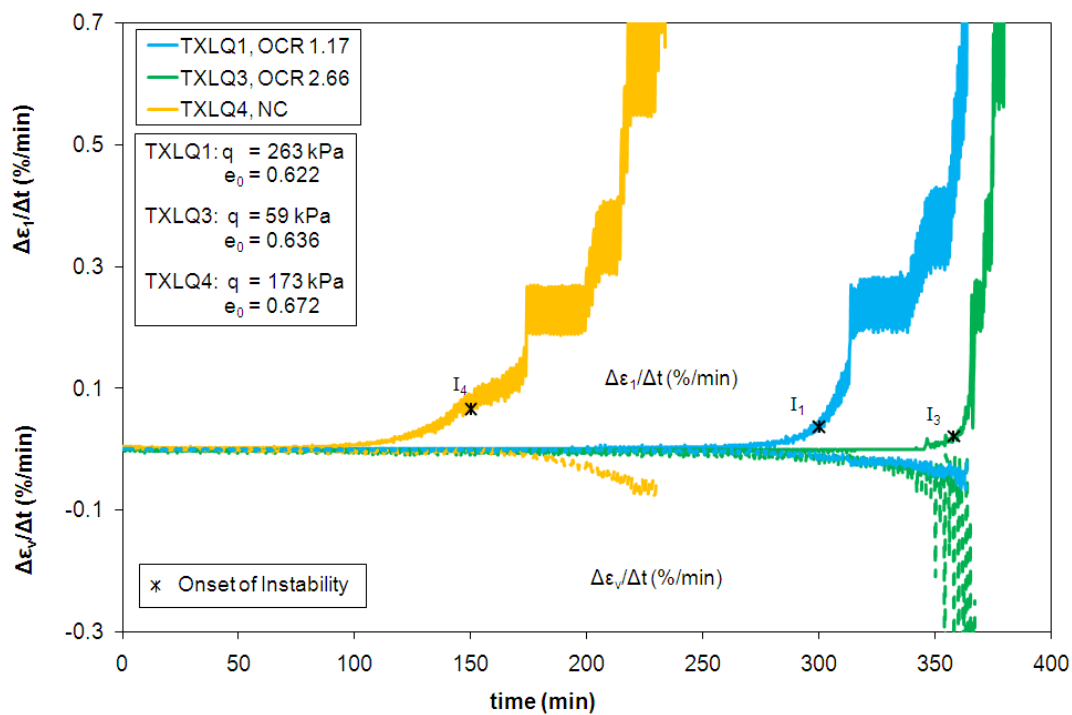


Figure 7.6(d)

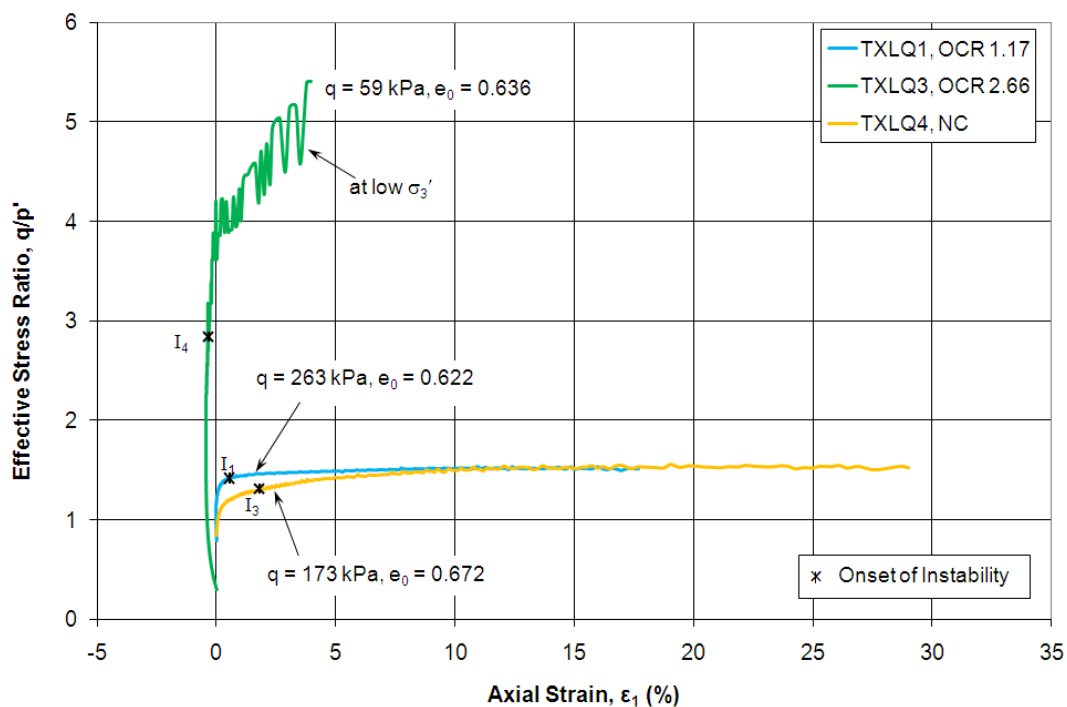


Figure 7.6(e)

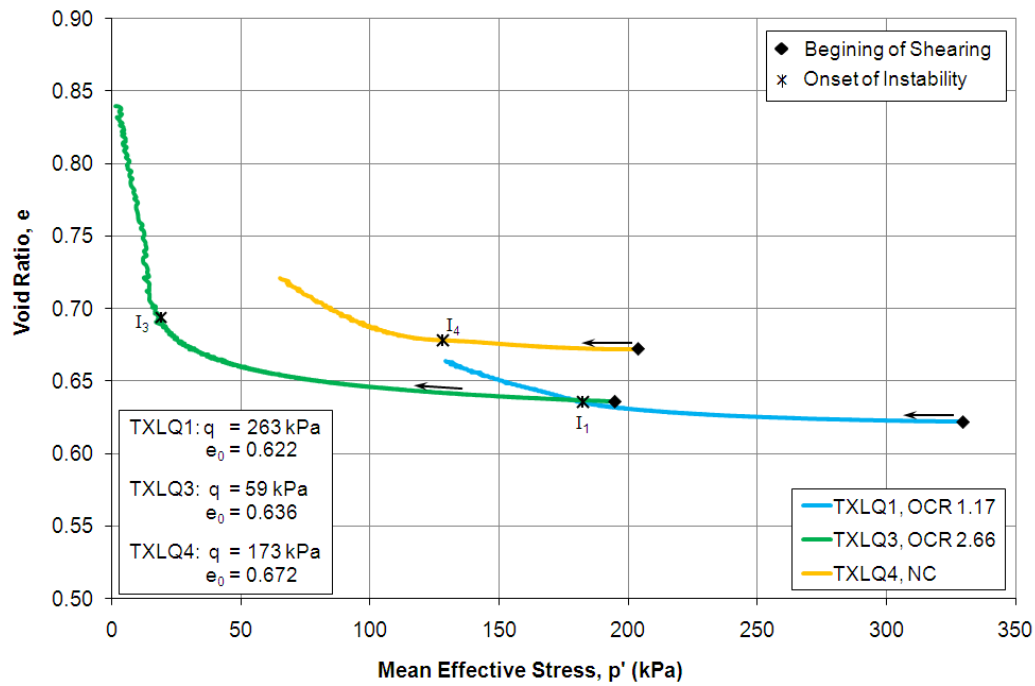


Figure 7.6 (f)

Figure 7.6 General Drained Behaviour of Compacted Residual Soil Resulted from CSD-T Tests: (a) Effective Stress Paths; (b) q vs. Time Curves; (c) $\varepsilon_v\%$ and $\varepsilon_1\%$ vs. Time Curves; (d) $\Delta\varepsilon_v/\Delta t$ and $\Delta\varepsilon_1/\Delta t$ vs. Time Curves; (e) q/p' vs. $\varepsilon_1\%$ Curves; (f) e vs. p' Curves

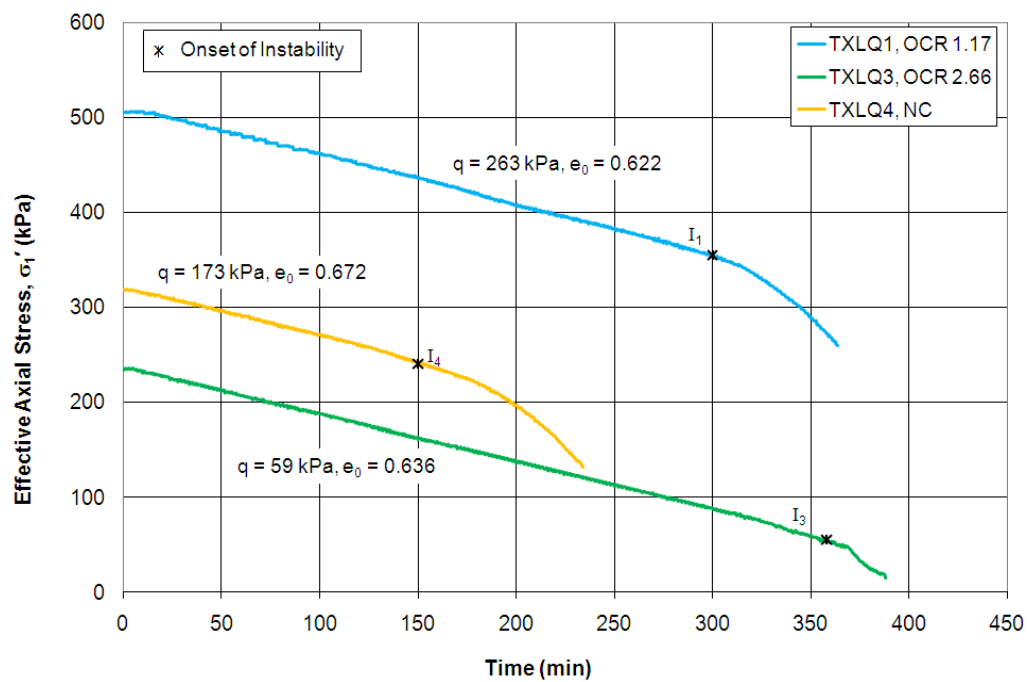


Figure 7.7(a)

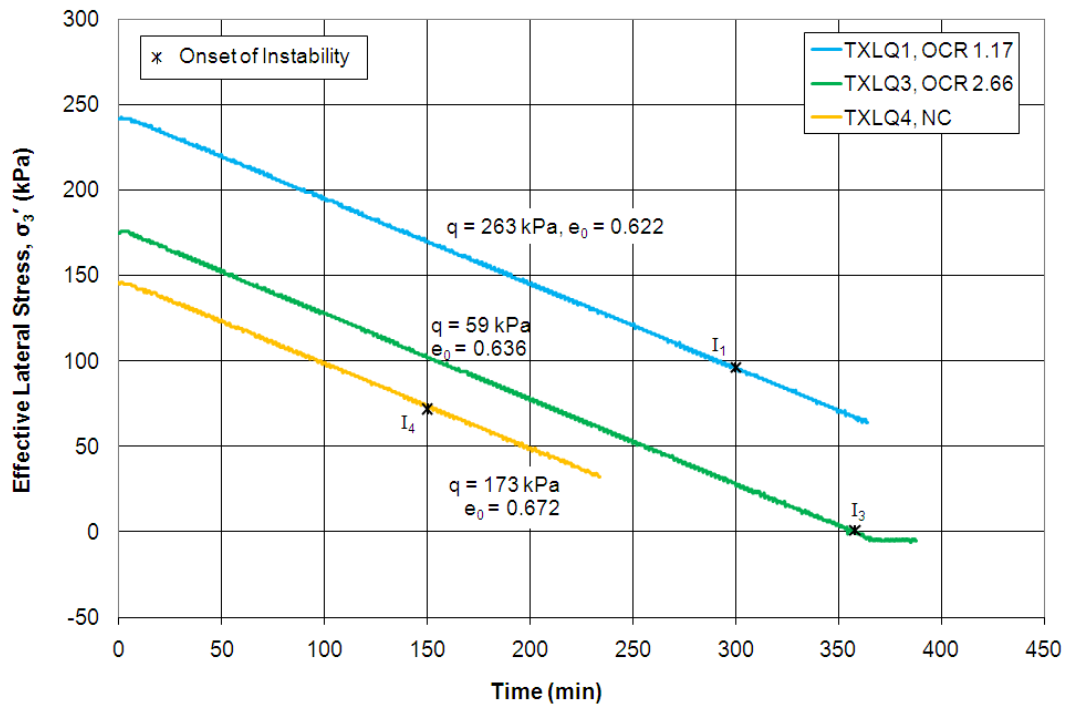


Figure 7.7(b)

Figure 7.7 Variations of Effective Stresses during CSD-T Tests on Compacted Residual Soil: (a) σ_1' ; (b) σ_3'

7.2.4 Tests under Plane-Strain Conditions

Many geotechnical problems, such as earth dam and embankment, can only be idealized into plane-strain conditions at the best, rather than axisymmetric conditions. Therefore, it is more relevant to study the instability behaviour or behaviour under CSD tests under plane-strain conditions. Five CSD tests were conducted on compacted residual soil under plane-strain conditions, which will be referred as CSD-PS tests in following sections. All the tests are summarized in Table 7.3. The test results are presented in Figure 7.8.

Table 7.3 Summary of CSD-PS Tests on Compacted Residual Soil Specimens

Test No.		HFGN2	HFGN3	HFGN4	HFGN5	HFGN6
Pressure for Saturation (kPa)	Cell	860	860	860	860	860
	Back	850	850	850	850	850
OCR		1	3	1	1	2
Shearing Rate (kPa/min)		0.5	0.1-0.3	0.5	0.5	0.5
Testing Methods		reduce cell	increase back	increase back	reduce cell	reduce cell
Weight (g)	Pre-Test	825.94	825.63	830.68	829.51	834.1
	Post-Test	-	-	-	849.77	866.03
Water Content (Pre-Test) (%)	Average	23.42	23.24	23.25	23.82	23.51
Water Content (Post-Test) (%)	Average	26.48	30.7	25.31	26.58	27.94
	Top	26.43	-	25.19	26.61	27.55
	Middle	26.145	-	25.5	26.33	SB
	Bottom	26.855	-	25.26	26.59	27.48
	Shear Band	26.145	-	-	26.81	28.80
$\sigma_1'{}_0$ (kPa)		450.89	260.83	450.15	315.94	136.72
q_0 (kPa)		259.97	58.19	264.98	185.75	85.9
p'_0 (kPa)		277.63	222.04	273.51	192.11	79.51
e_0		0.641	0.598	0.664	0.709	0.738
$(q/p')_{\text{peak}}$		1.365	2.25	1.421	1.4	1.65
$R_{\text{peak}} = (\sigma_1'/\sigma_3')_{\text{peak}}$		3.816	-	2.222	4.408	2.875
ε_1 at $(q/p')_{\text{peak}}$ (%)		4.705	3.06	4.58	4.54	7.673
Secant Friction Angle, ϕ' (°)		40.5	-	39.91	39.72	50.3

Five compacted residual soil specimens were consolidated anisotropically to different mean effective stresses to create different OCRs. The consolidation method was described in Chapter 4 and 5. Specimens HFGN2, HFGN4 and HFGN5 were normally consolidated to mean effective stresses of 228kPa, 274kPa and 192kPa respectively. The effective stress paths during consolidation are shown as thin dotted curves in corresponding colors for reference in Figure 7.8(a). After consolidation, shearing was carried out along a constant-shear-drained path under the plane-strain condition. Due to the reduction of effective confining stress, the effective stress paths move horizontally towards the left. Test HFGN3 and HFGN4

were conducted by increasing the back pressure at a constant rate, while maintaining the cell pressure constant. And the rest were carried out at a constant back pressure, while reducing the cell pressure at the same rate of 0.5 kPa. As seen from Figure 7.8(a) and (b), a gradual reduction in the deviator stress was observed. This was due to the gradual increase in the cross-sectional area of the soil specimen.

The onset of instability was determined from the point when variation of axial strain rate and axial strain rate increased abruptly, as shown in Figure 7.8(c) and (d). Sudden increases in volumetric strain and volumetric strain rate were also noted, which is similar to the observation made under axisymmetric conditions.

It is shown in Figure 7.8(a), the deviator stress dropped drastically when the failure state is approached. Failure envelope AC was determined as an envelope to all the effective stress paths. Within the testing stress range, the failure envelope was a straight line with a slope of 1.29 (BC in Figure 7.8(a)). The instability line determined from CK₀UC tests (Figure 7.1(a)) is also shown for reference. For both NC and OC specimens, the onset of instability located closely to the failure envelope. Dilation of compacted residual soil specimens was noted after onset of instability, and the specimen with higher OCR dilated more significantly. Similar observation has been made under triaxial conditions. This may be due to the tendency for soil to fracture.

Figure 7.8(e) and (f) present the effective stress ratio, q/p' and b value measured during CSD-PS tests. Both q/p' and b value increased as axial strain increased, and reached a constant value when stress path approaching to the failure envelope. The increment of b value towards the end of the test (as shown in Figure 7.8(f)) is due to the non-homogeneity of σ_2 after shear band generation.

The σ_2 readings were recorded by four individual pressure transducers located on the sides of the plane-strain specimen. The σ_2 distribution curves are presented in Figure 7.9. The testing method of increasing back pressure was adopted for test HFGN4, therefore, increment of σ_2 reading was observed in Figure 7.9(b). In

contrast, the σ_2 values decreased during test HFGN2, HFGN5 and HFGN6, since different testing method was adopted. It is important to mention that the σ_2 value used for calculation was taken as the average of four individual load cells readings.

Based on an independent study using the same plane-strain cell, Wanatowski (2005) and Wanatowski & Chu (2007) proposed a method to identify the formation of shear band by comparing the four σ_2 measurements. As shown in Figure 7.9, it is observed that the σ_2 values measured by the four individual pressure transducers were essentially the same until point F. It indicated the shear band had formed at this point. Once the shear band was fully developed, the σ_2 readings diverged significantly. The formation of shear band, along with abrupt increase in volumetric strain rate, took place under increasing back pressure. This is consistent with the practical criteria for hydraulic fracture. It is indicated that the hydraulic fracture could be consistent with soil failure after onset of instability.

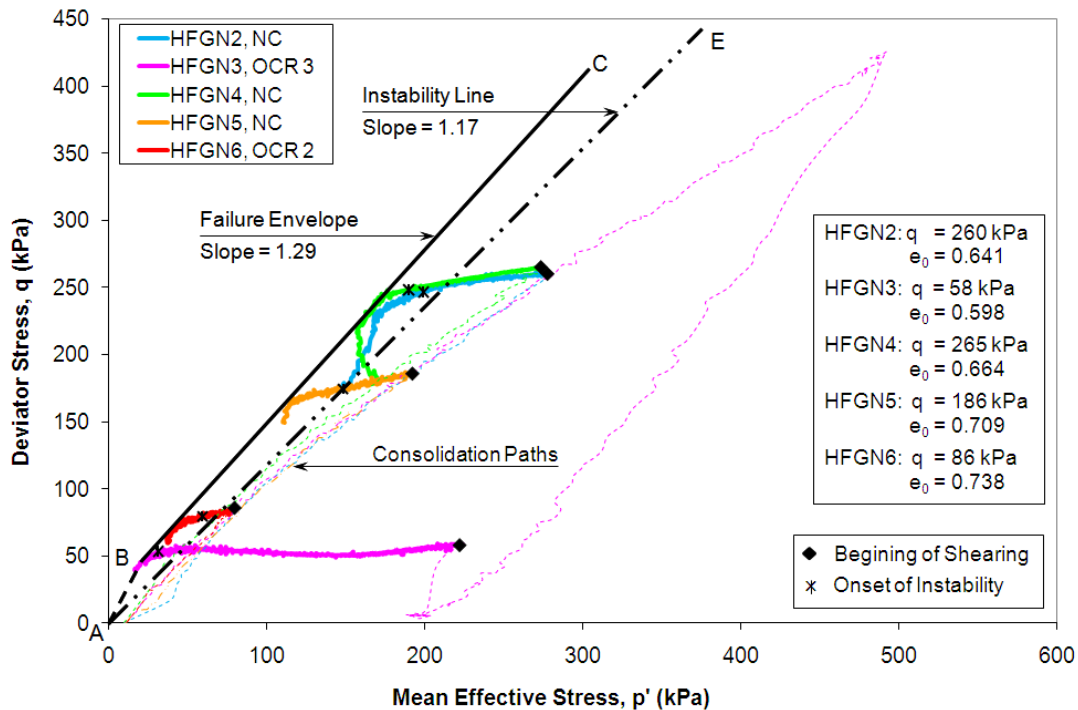


Figure 7.8(a)

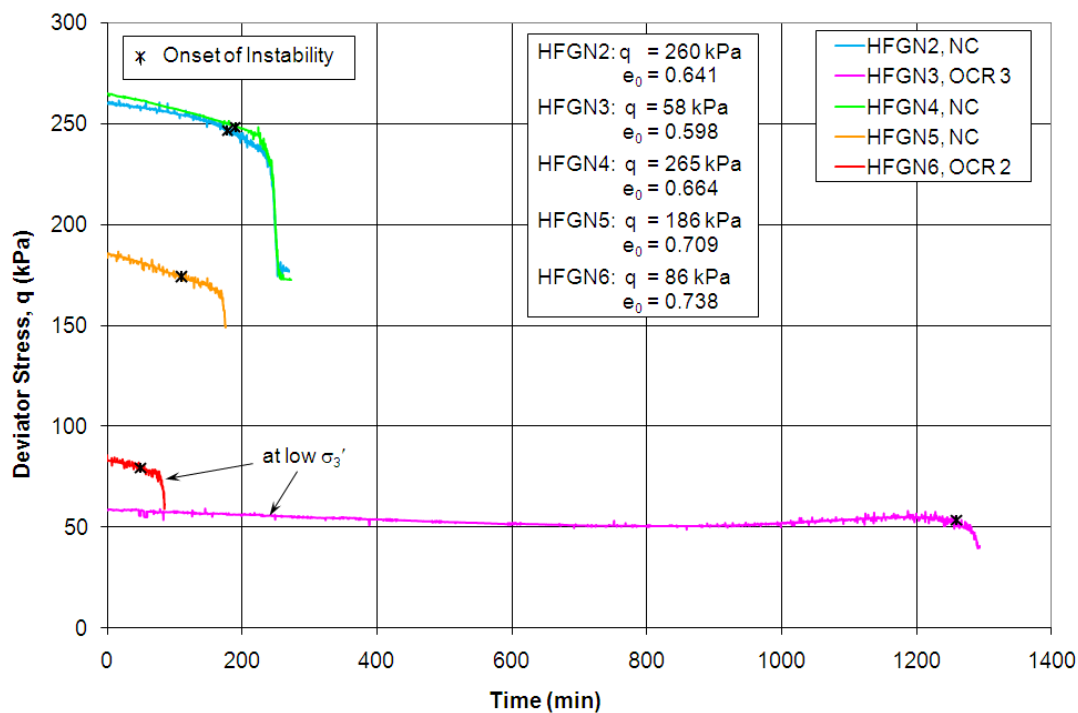


Figure 7.8(b)

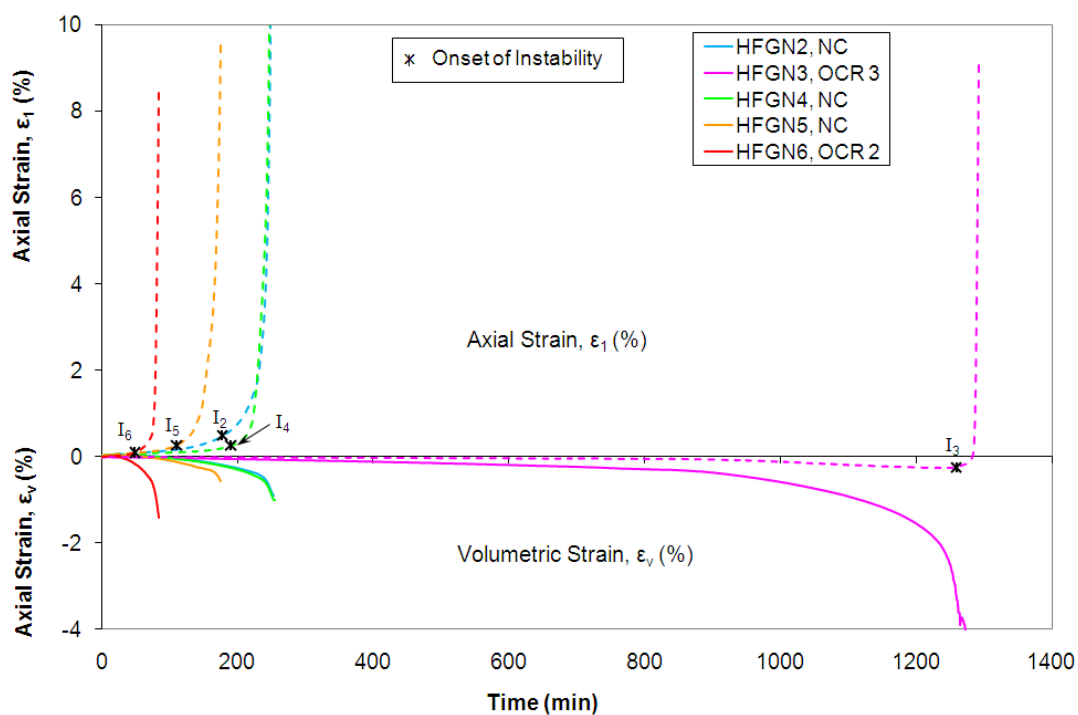


Figure 7.8(c)

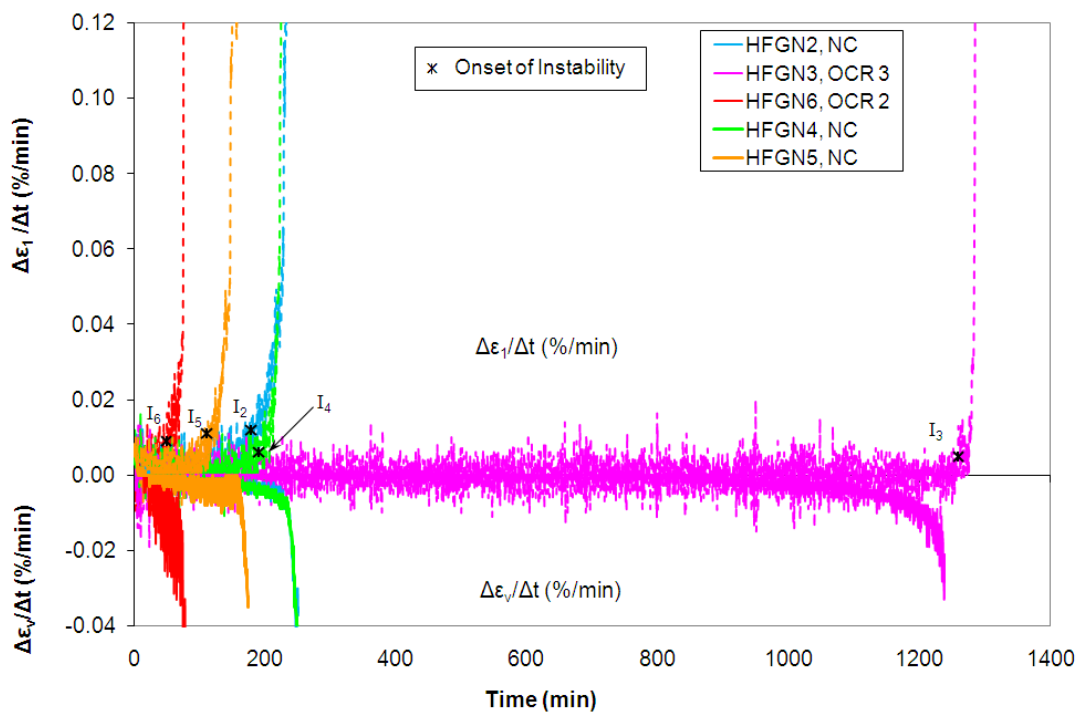


Figure 7.8 (d)

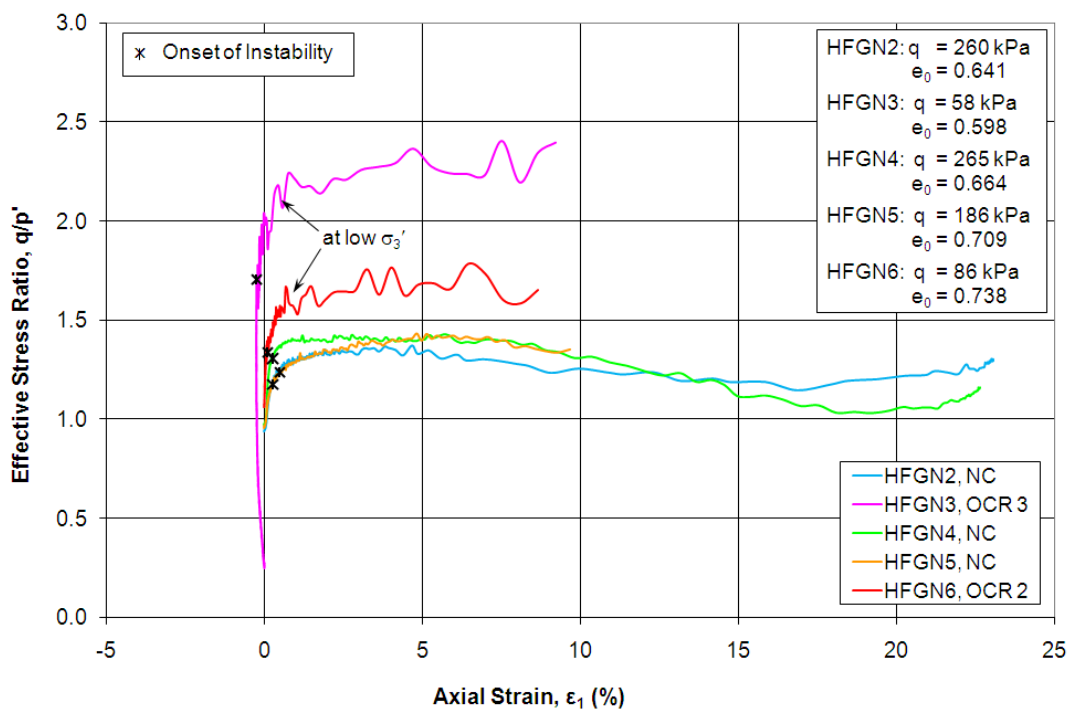


Figure 7.8(e)

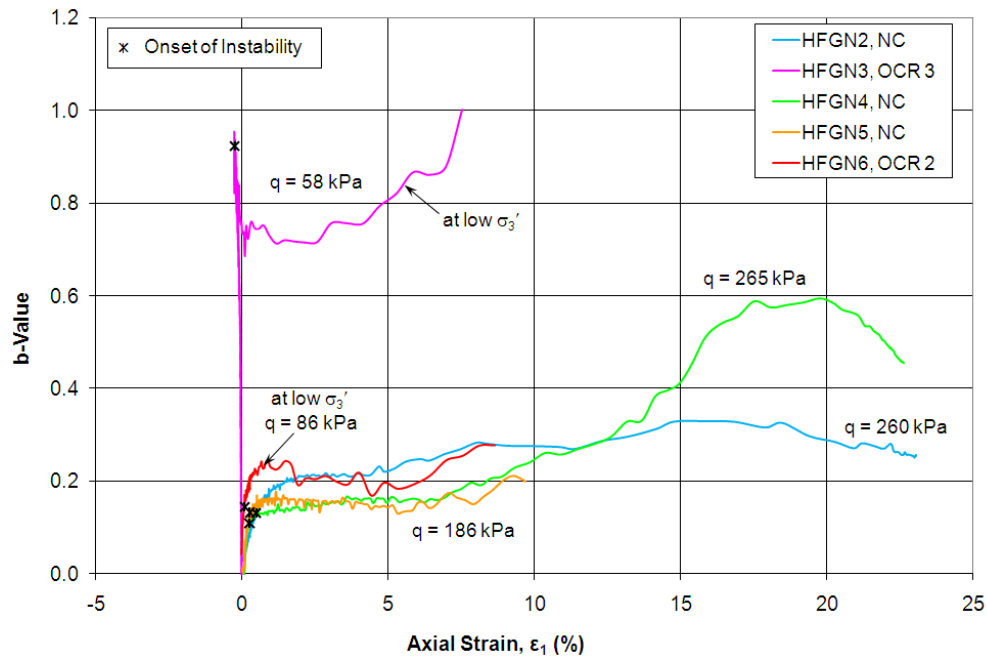


Figure 7.8(f)

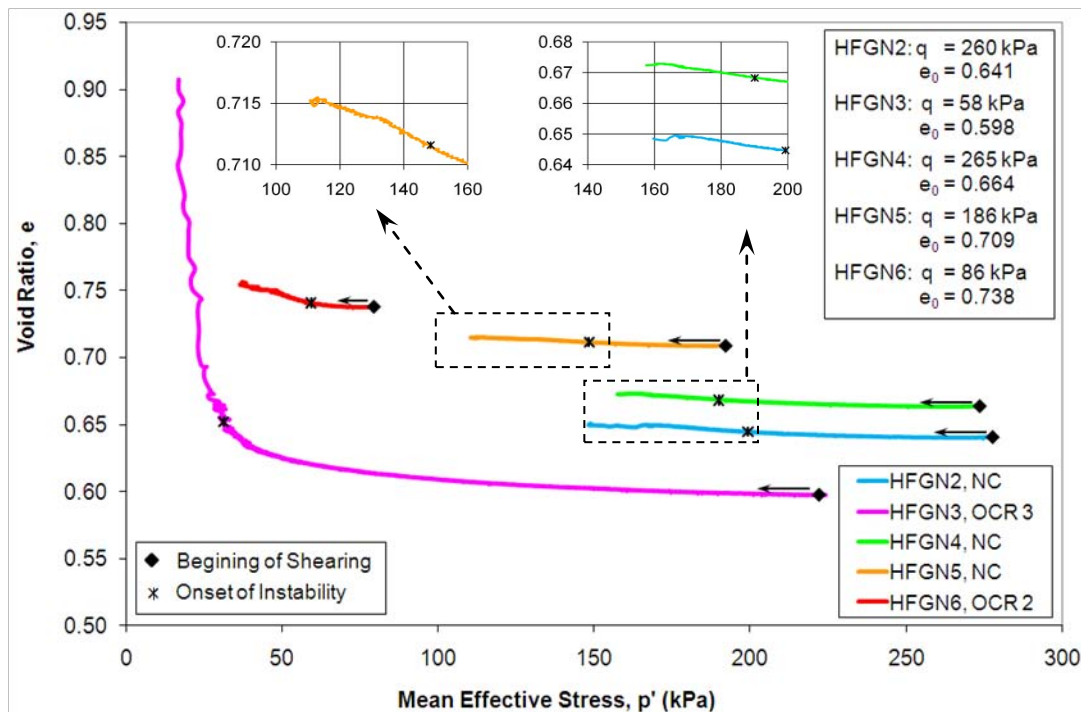


Figure 7.8 (g)

Figure 7.8 General Drained Behaviour of Compacted Residual Soil in K_0 CS-PS Tests: (a) Effective Stress Paths; (b) q vs. Time Curves; (c) $\Delta\epsilon_v\%$ and $\epsilon_1\%$ vs. Time Curves; (d) $\Delta\epsilon_v/\Delta t$ and $\Delta\epsilon_1/\Delta t$ vs. Time Curves; (e) q/p' vs. $\epsilon_1\%$ Curves; (f) b value vs. $\epsilon_1\%$ Curves; (g) e vs. p' Curves

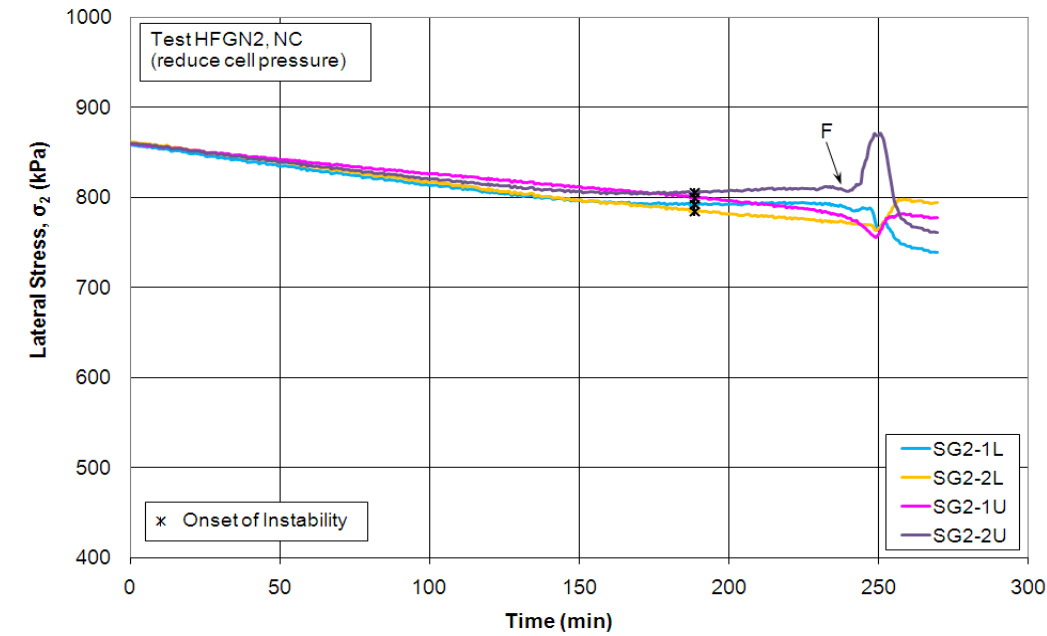


Figure 7.9 (a)

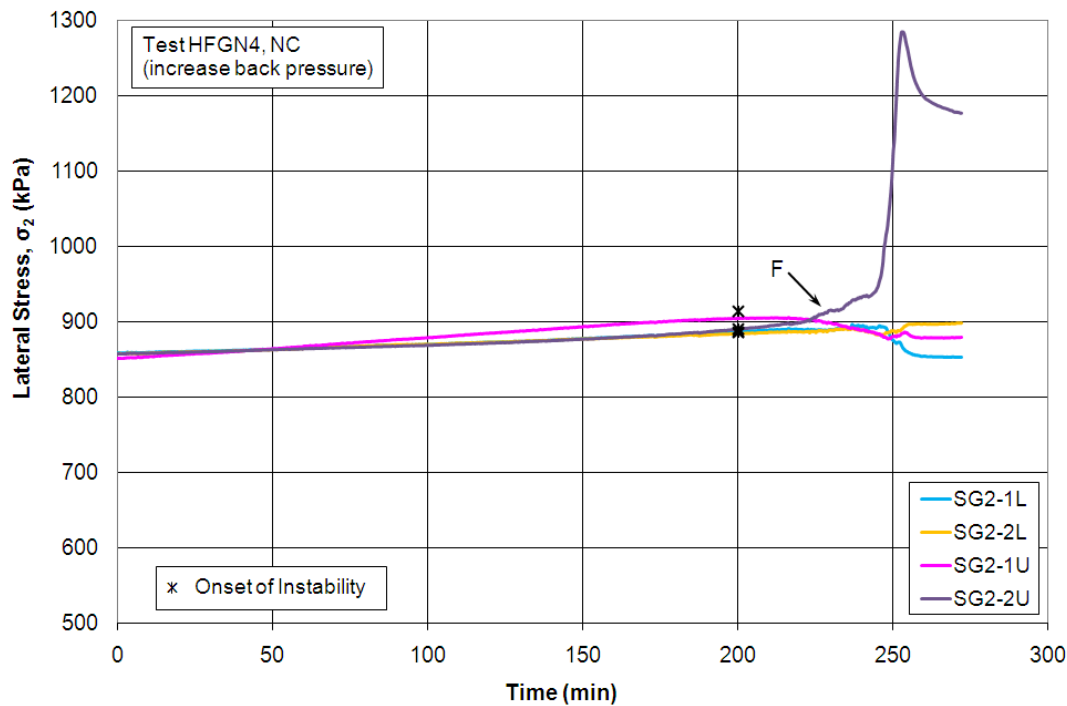


Figure 7.9(b)

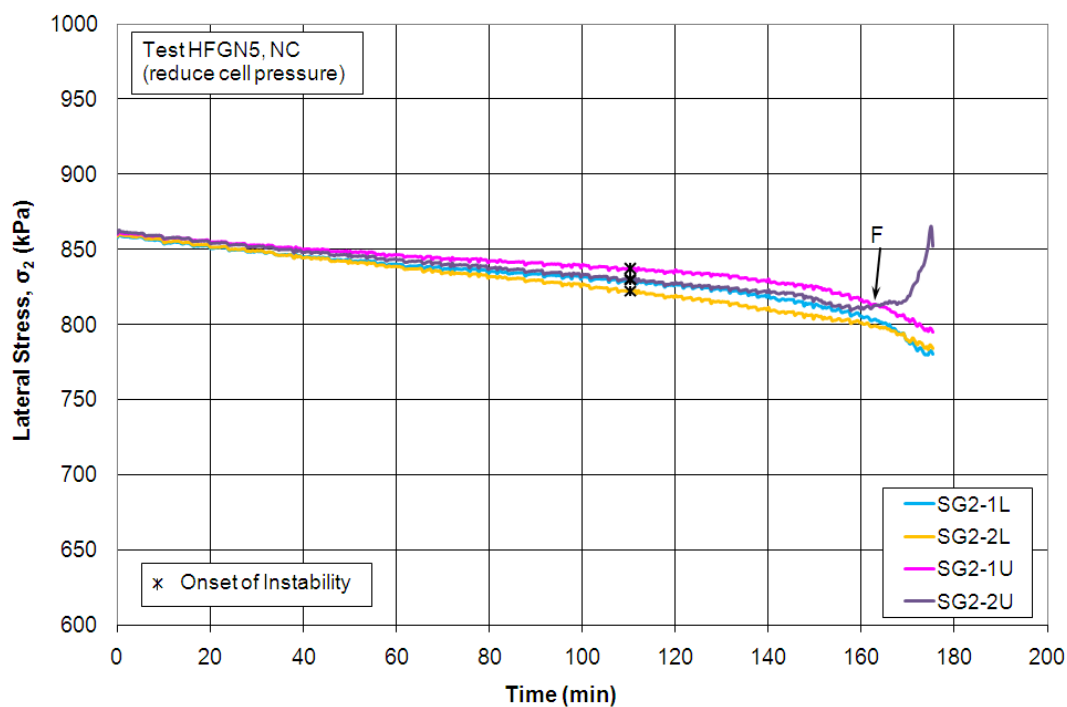


Figure 7.9 (c)

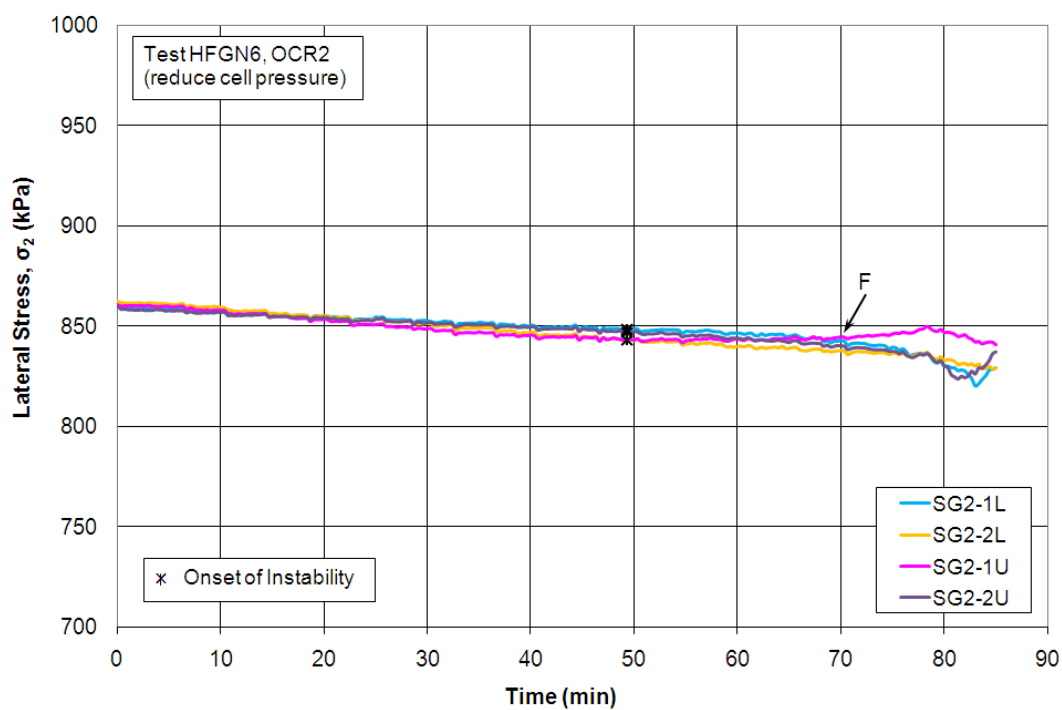


Figure 7.9(d)

Figure 7.9 Lateral Stress σ_2 vs. Time Curves Obtained from (a) Test HFGN2; (b) Test HFGN4; (c) Test HFGN5; (d) Test HFGN6

Figure 7.10 presents the variation of effective stresses during CSD-PS tests. It is observed that the minor effective stress (σ_3') approaches to zero only at the end of test HFGN3. It is verified that the instability occurs when effective stresses reached the failure line, and is not controlled by σ_1' or σ_3' . Similar observation was made under triaxial conditions. As pointed out by many researchers (Carter et al. 1986; Lockner and Byerlee 1977; Morgenstern and Vaughan 1963; Mori and Tamura 1987; Panah and Yanagisawa 1989), hydraulic fracture could be initiated by the shear failure at high confining pressure and differential stress. The hydraulic fracture of the compacted residual soil is brought by reducing the effective stresses until they satisfy the Mohr-Coulomb failure criterion. Similar observation has been made from CSD-T tests under triaxial conditions.

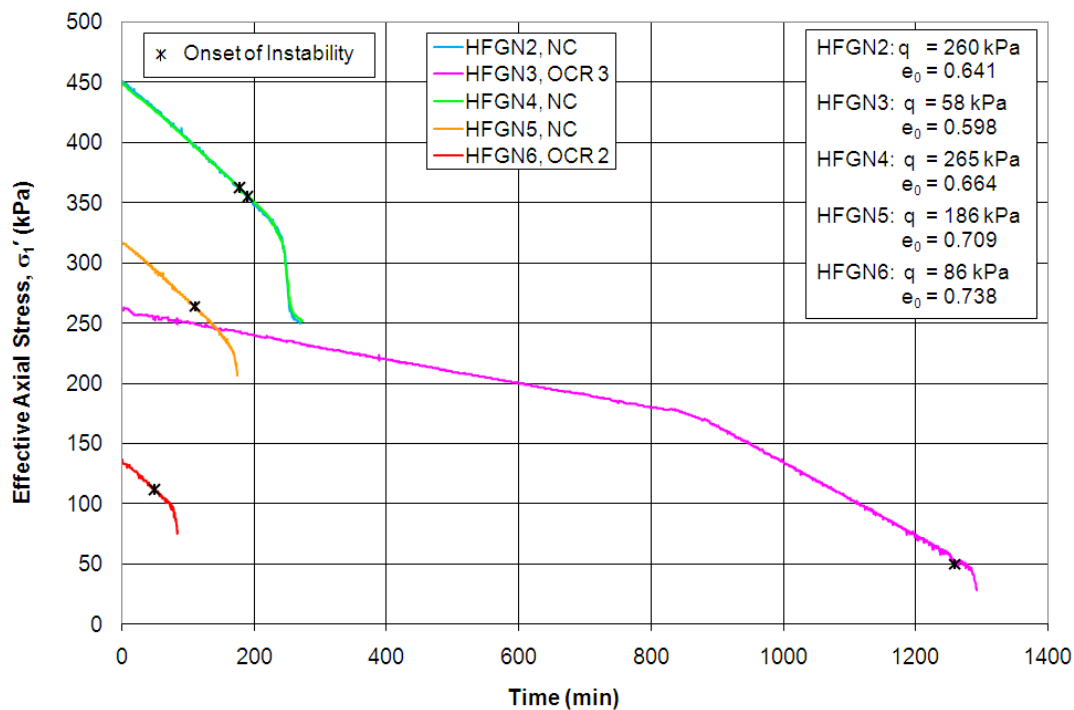


Figure 7.10(a)

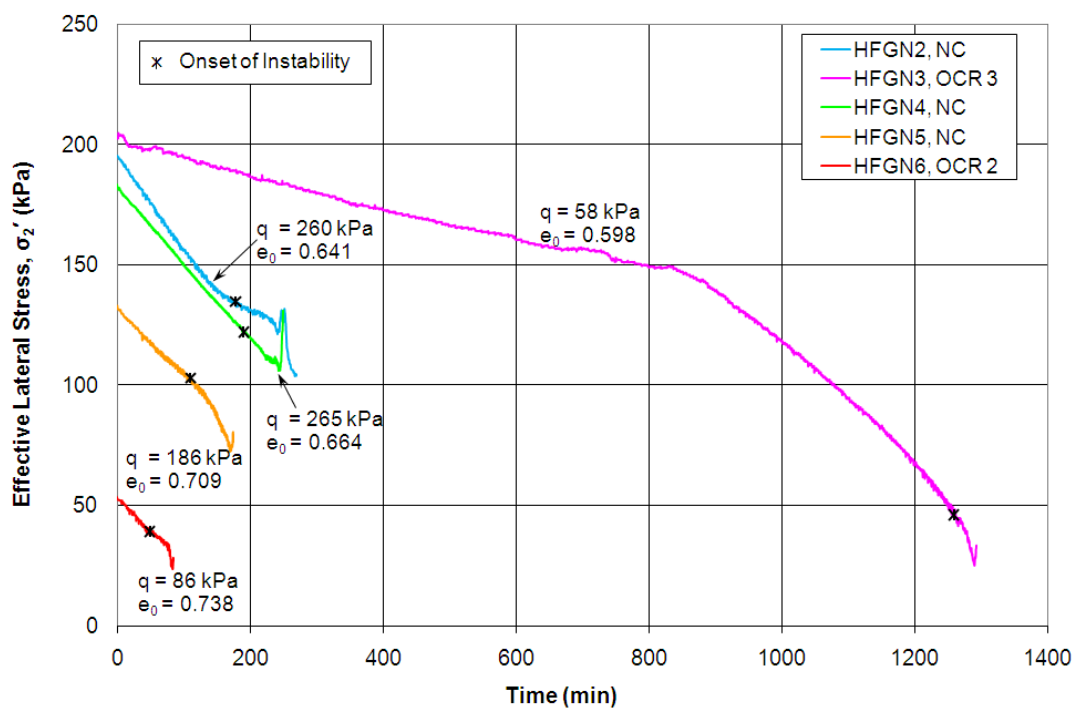


Figure 7.10(b)

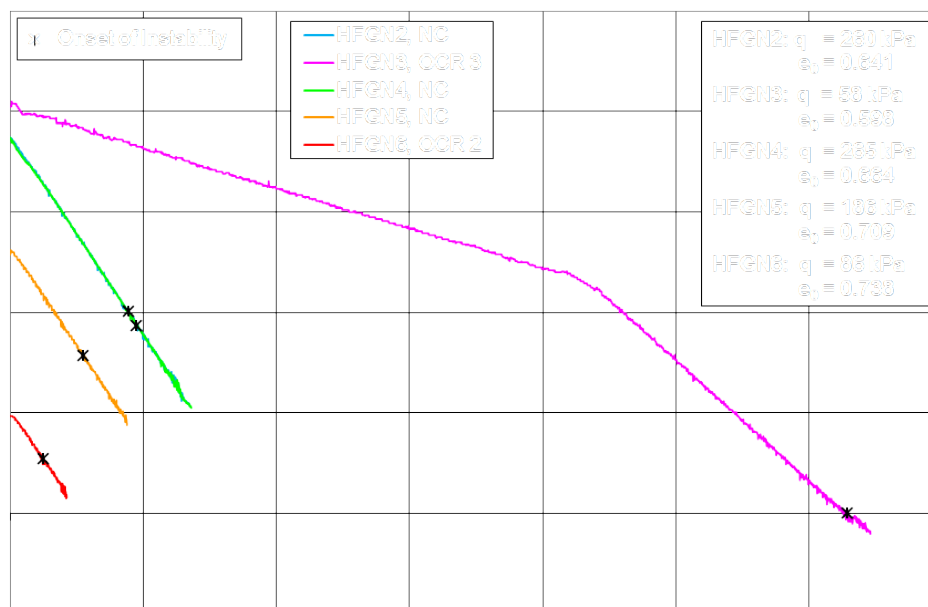


Figure 7.10(c)

Figure 7.10 Variations of Effective Stresses during CSD-PS Tests on Compacted Residual Soil: (a) σ'_1 ; (b) σ'_2 ; (c) σ'_3

7.2.5 Summary

The data presented in the preceding section showed that the instability may take place in both NC and OC compacted residual soil under triaxial conditions. This is consistent with previous experimental studies conducted on sand under axisymmetric conditions (Chu & Leong, 2001; Chu et al., 2003; Loke, 2004). They also showed that the instability line under axisymmetric condition is defined based on yielding conditions. Large plastic strain developed after onset of instability. By carrying out CSD-PS tests using plane-strain apparatus, the instability behaviour was also observed for both NC and OC specimens. Similar observations have been made for sand under plane-strain conditions (Wanatowski, 2005).

Under triaxial / axisymmetric conditions, the instability appears to occur closely to failure state. Although the axial strain was increasing at accelerating rate, the compacted residual soil specimen did not collapse instantly during drained instability. As shown in Figure 7.9, the formation of shear band was some time after onset of instability. Similar observation has been reported by Leong (2001), Chu & Leong (2001), Chu et al. (2003) and Loke (2004) under axisymmetric conditions, and also by Wanatowski (2005) under plane-strain conditions.

It is also important to mention that large volumetric strain was observed in both CSD-T and CSD-PS tests. In laboratory fracture tests, hydraulic fracturing is generally believed to initiate when volumetric strain rate increased abruptly with a constant increment in back pressure (Bjerrum et al., 1972; Bjerrum & Andersen, 1972; Jaworski et al., 1981; Mori & Tamura, 1987; Alfaro & Wong, 2001; Ohta et al., 2007). According to the comparison made in Section 7.2.2, reduce cell pressure during a CSD test is equivalent to increase in back pressure. Abrupt increase in volumetric strain rate together with increase in back pressure could lead to hydraulic fracture inside the compacted residual soil specimen. The tendency for the soil specimen to fracture was also verified by the sudden increase in void ratio. It is indicated the hydraulic fracture of soil should be related to the instability behaviour.

7.3 CSD Tests on Intact Residual Soil

To compare the instability behaviour between the intact and compacted residual soil, three CSD-PS tests as summarized in Table 7.4 were conducted on intact residual soil specimens using plane-strain apparatus. The testing data are presented in Figure 7.11.

Table 7.4 Summary of CSD-PS Tests on Intact Residual Soil Specimens

Test No.		PSGNQ1	PSGNQ2	PSGNQ3
Pressure for Saturation (kPa)	Cell	860	860	860
	Back	850	850	850
OCR		1	2.1	1
Testing Method		reduce cell	reduce cell	reduce cell
Shearing Rate (kPa/min)		0.5	0.5	0.5
Weight (g)	Pre-Test	801.07	824	-
	Post-Test	796.09	839.56	806.51
Water Content (Pre-Test) (%)	Average	32.01	32.02	30.50
Water Content (Post-Test) (%)	Average	31.43	34.68	30.69
	Top	30.82	34.51	29.75
	Middle	31.26	34.59	31.28
	Bottom	31.66	34.94	30.54
	Shear Band	31.60	34.50	30.66
$\sigma_1'{}_0$ (kPa)		453.19	257.91	702.00
q'_0 (kPa)		250.94	60.18	350.20
p'_0 (kPa)		285.91	217.81	468.54
e_0		0.880	0.913	0.767
$(q/p')_{\text{peak}}$		1.44	1.70	1.30
$R_{\text{peak}} = (\sigma_1'/\sigma_3')_{\text{peak}}$		4.34	-	3.79
Secant Friction Angle, ϕ' (°)		38.71	-	35.62

The effective stress paths obtained from CSD-PS tests on intact residual soil are shown in Figure 7.11(a). Specimen PSGNQ1 and PSGNQ3 were normally consolidated to mean effective stresses of 453kPa and 702kPa, respectively. Specimen PSGNQ2 was anisotropically consolidated to create OCR of 2.1. The same consolidation method described in Chapter 4 and 5 was adopted here. To

shear the specimens along a constant stress path, the testing method of reducing cell pressure, while maintaining back pressure constant was adopted. This is because it is difficult to control σ_3 without affecting σ_2 in plane-strain tests.

The instability line and critical state line on the intact residual soil presented in Figure 7.11(a) were determined from CK_0 UPS tests as presented in Chapter 5. The onset of instability was defined as the point when axial strain and axial strain rate start to increase drastically, as shown in Figure 7.11(d). It is clearly observed that NC specimens started to undergo large deformation before the stress state reaches the critical state line (CSL). When a specimen is subjected to a high consolidation stress well into the virgin compression zone, some soil structure or cementation bonding may have been broken down. When a specimen is in an overconsolidated state, the soil structure is relative stable and large plastic deformation will not occur until the stress level well exceeds the yielding stress. Therefore, the instability for OC specimen will occur at a stress level higher than the CSL, as observed from experiments. It is also noted in Figure 7.11(g) that the OC specimens dilated significant after the onset of instability. This may be an indication of tendency for soil to fracture. Similar behaviour was made from the CSD tests conducted on the compacted residual soil (as presented in Section 7.2). The variations of q/p' and b -value with axial strain are presented in Figure 7.11(e) and (f). Both of q/p' and b -value increased rapidly at beginning stage of shearing. They tended to reach constant values with increasing axial strain. It also observed from Figure 7.11(g), the b -value increased with effective confining stress. It varied in a large range (from 0.1 to 1.2).

The measurements of lateral stress σ_2 during test PSGNQ1 and PSGNQ3 are plotted against time in Figure 7.12. The readings taken by four individual pressure transducers were almost identical until point F. The divergence of lateral stress σ_2 was an indication of shear band formation (Wanatowski, 2005; Wanatowski & Chu, 2007). It should be highlighted that the formation of shear band was some time after the onset of instability. The soil specimen does not collapse immediately due to drained instability. Similar observation has been made on the compacted residual

soil, as presented in Section 7.2. This is consistent with the studies carried out by Leong (2001), Chu & Leong (2001), Chu et al. (2003), Loke (2004) and Wanatowski (2005). It should also be pointed out that a visible shear band was only observed under plane-strain condition, but not under triaxial conditions, which is the same as compacted residual soil did. Pictures of the post-test specimens under different testing conditions were compared in Figure 7.13.

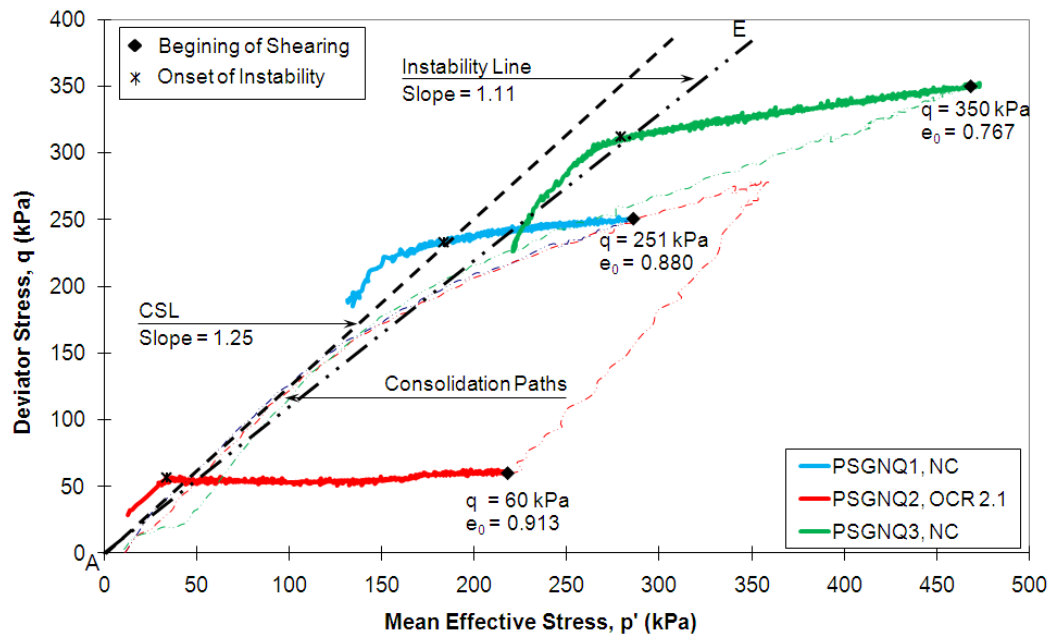


Figure 7.11(a)

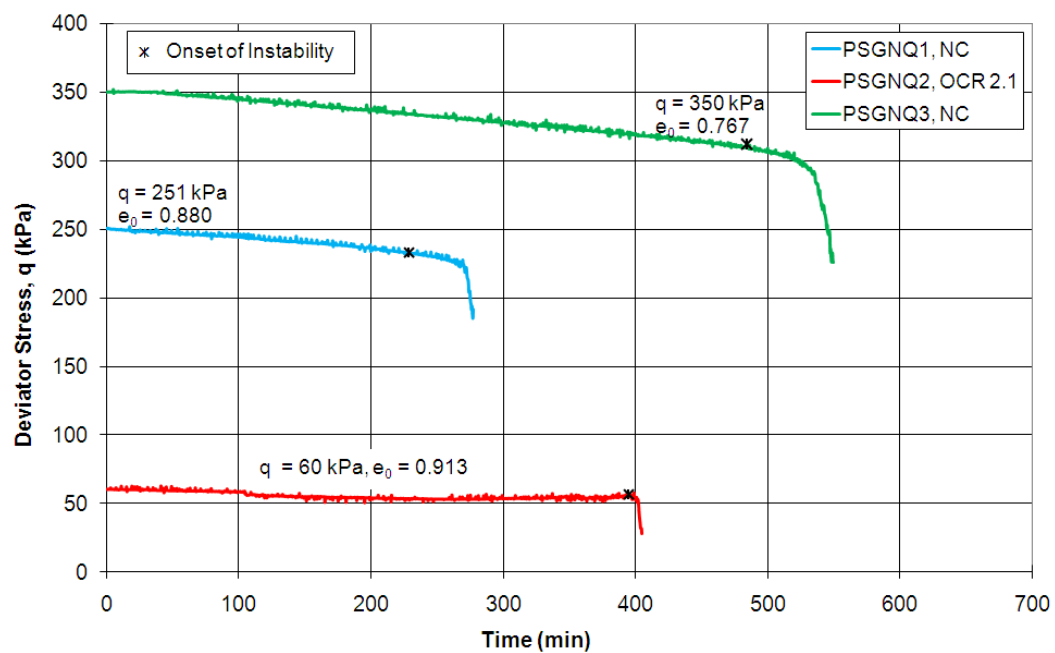


Figure 7.11(b)

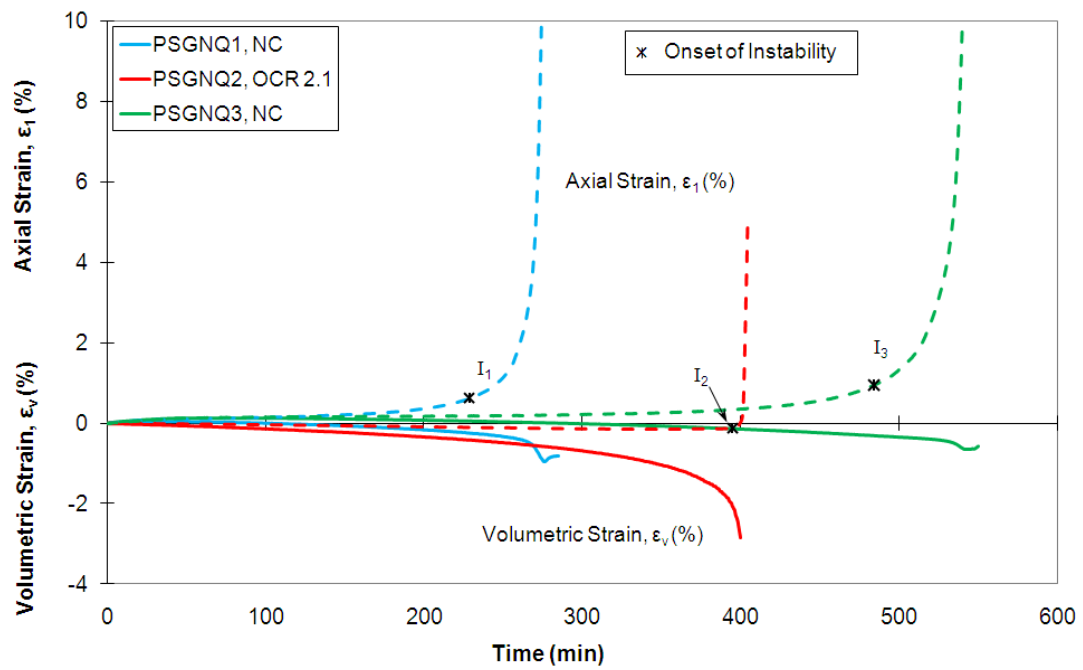


Figure 7.11(c)

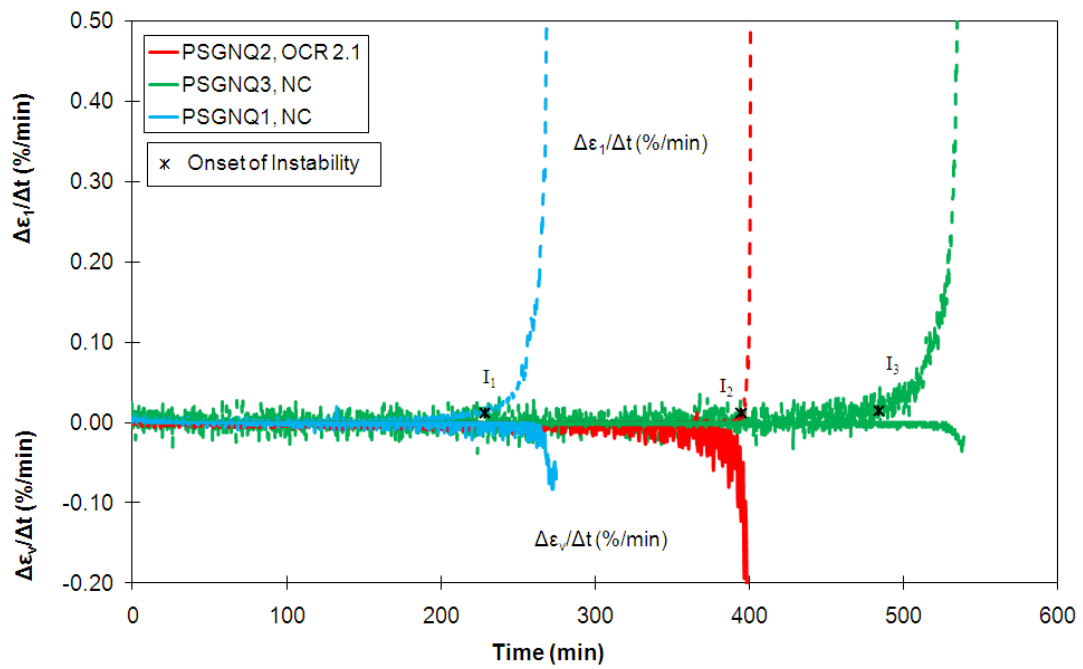


Figure 7.11 (d)

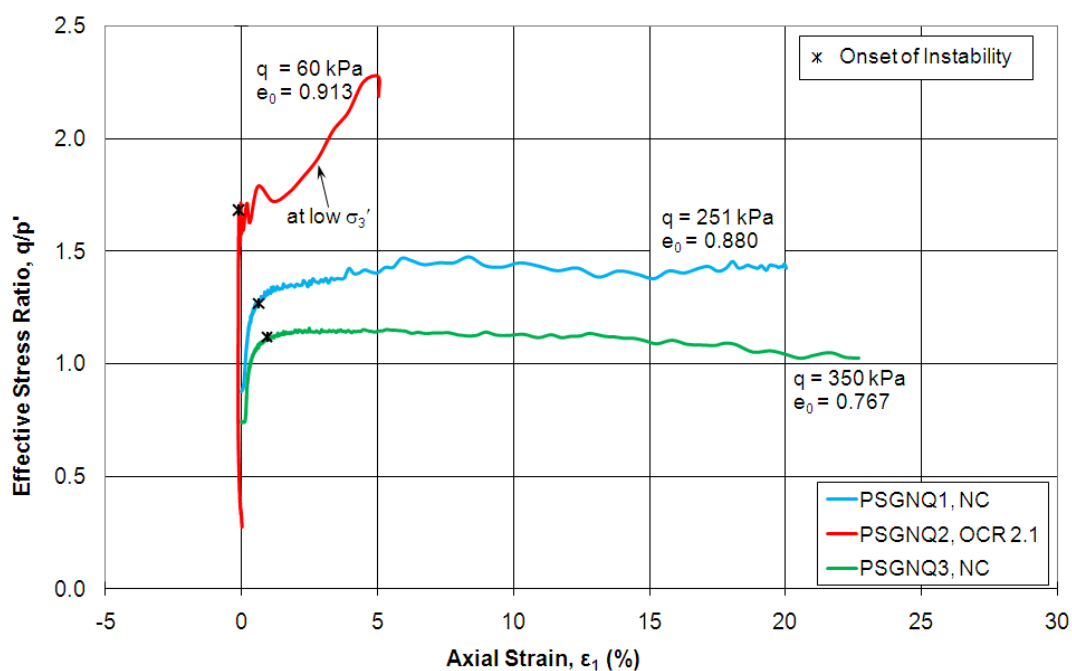


Figure 7.11(e)

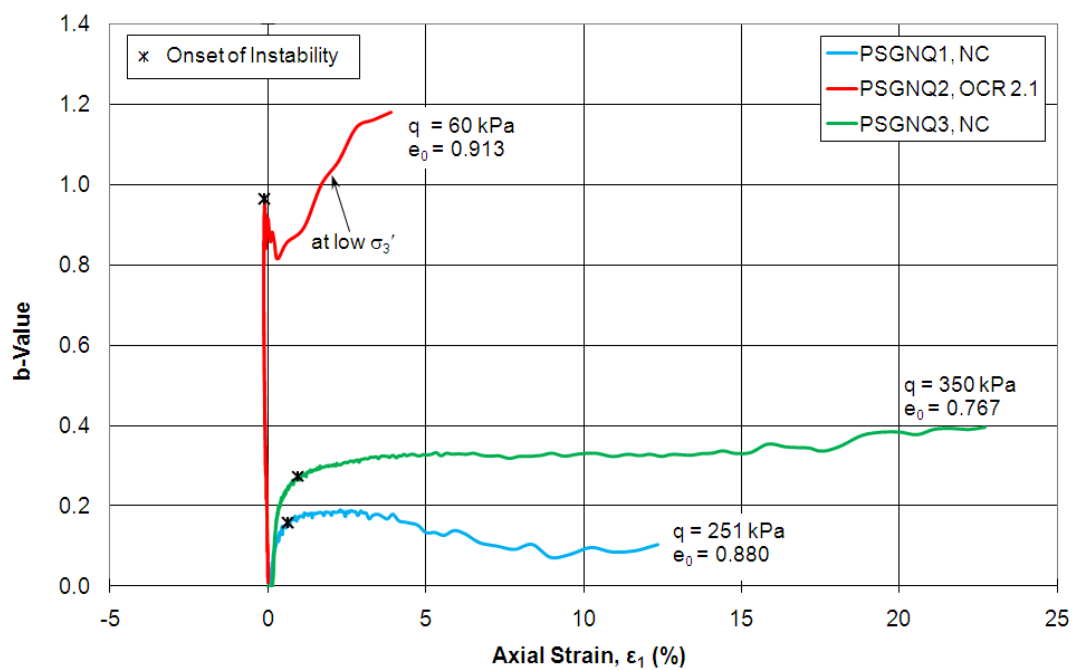


Figure 7.11(f)

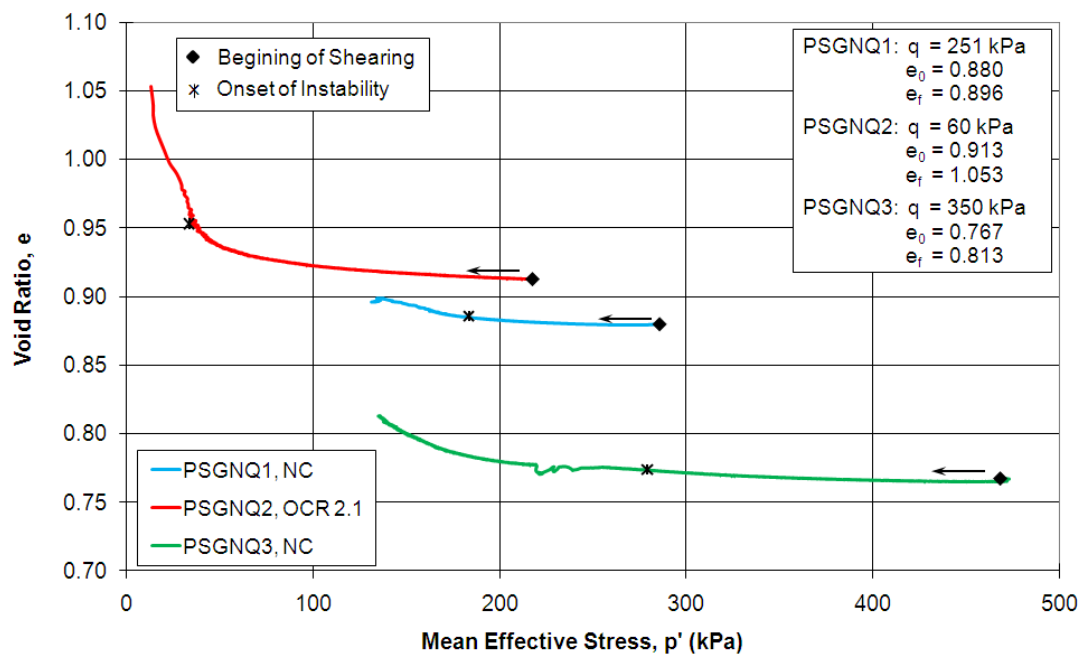


Figure 7.11 (g)

Figure 7.11 General Drained Behaviour of Intact Residual Soil Resulted from CSD-PS Tests: (a) Effective Stress Paths; (b) q vs. Time Curves; (c) $\varepsilon_v\%$ and $\varepsilon_1\%$ vs. Time Curves; (d) $\Delta\varepsilon_v/\Delta t$ and $\Delta\varepsilon_1/\Delta t$ vs. Time Curves; (e) q/p' vs. $\varepsilon_1\%$ Curves; (f) b Value vs. $\varepsilon_1\%$ Curves; (g) e vs. p' Curves

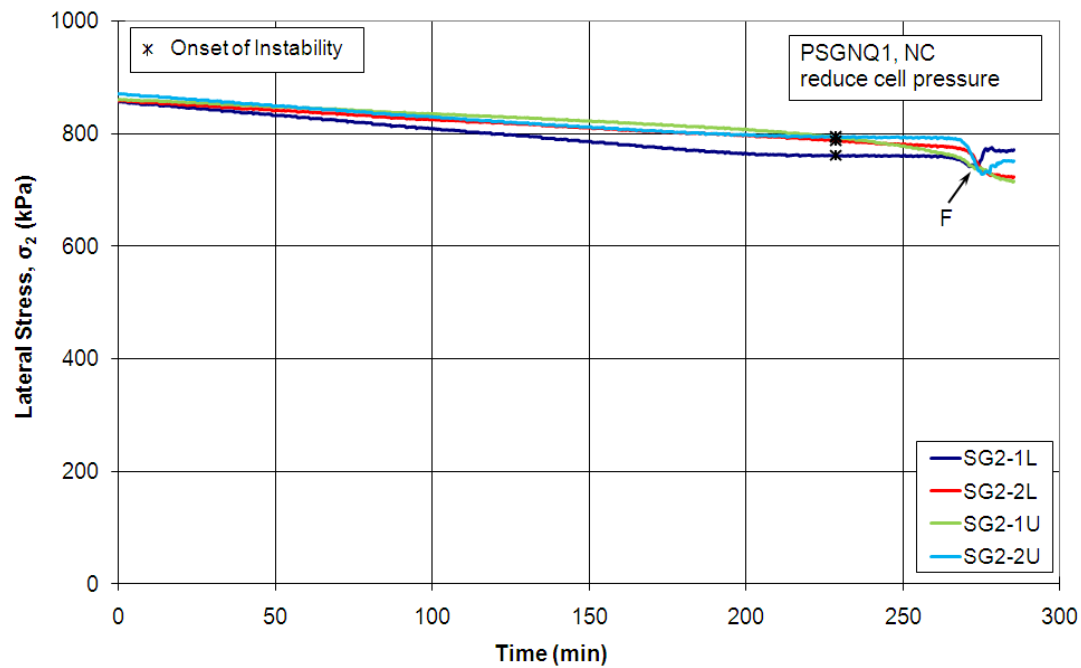


Figure 7.12 (a)

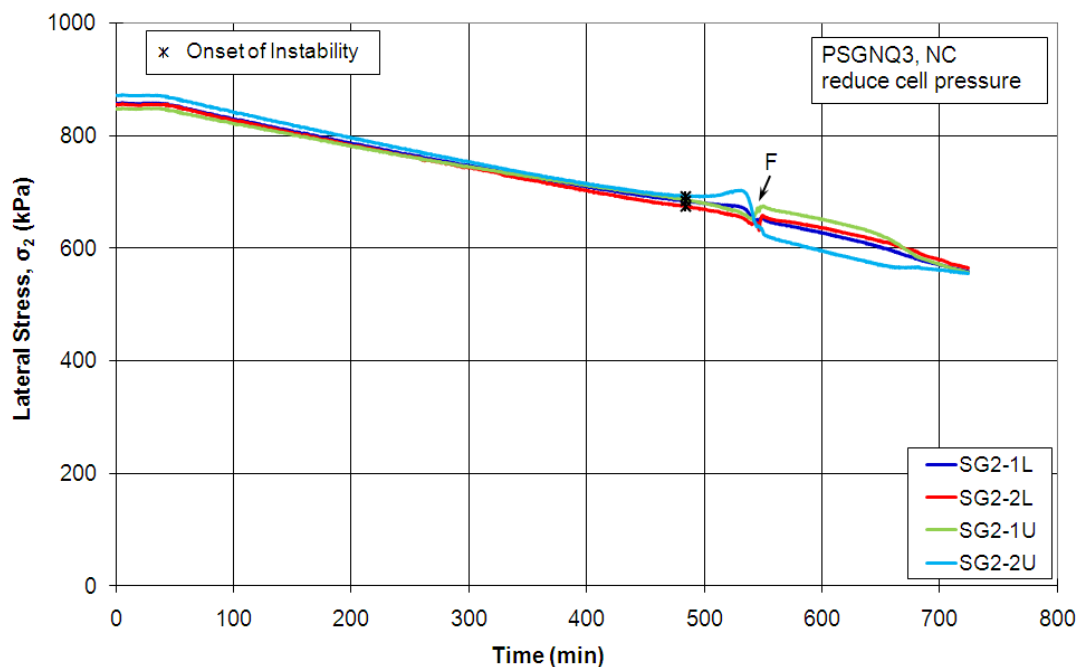
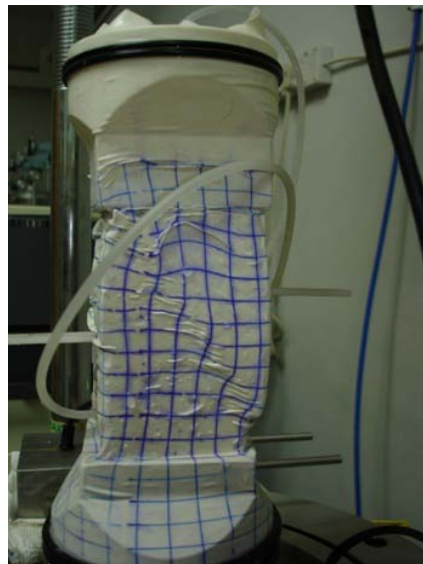


Figure 7.12 (b)

Figure 7.12 Lateral Stress σ_2 vs. Time Curves Obtained from CSD-PS Tests on the Intact Residual Soil Specimens: (a) Test PSGNQ1; (b) Test PSGNQ3



(a)



(b)

Figure 7.13 Post Test Pictures of the Intact Residual Soil: (a) Triaxial Specimen; (b) Plane-Strain Specimen

The effective stresses measured during CSD-PS tests on the intact residual soil specimens are plotted in Figure 7.14. Among the three tests conducted, only one (PSGNQ3) reached a zero effective minor stress (σ_3') towards the end of shearing (as shown in Figure 7.14(c)). It is important to point out that hydraulic fracture was initiated by shear failure at high confining pressure and differential stress, instead of tensile failure. The fracture of intact residual soil occurred while Mohr-Coulomb failure criterion was satisfied. The initiation of hydraulic fracture should be related to onset of instability and formation of shear band.

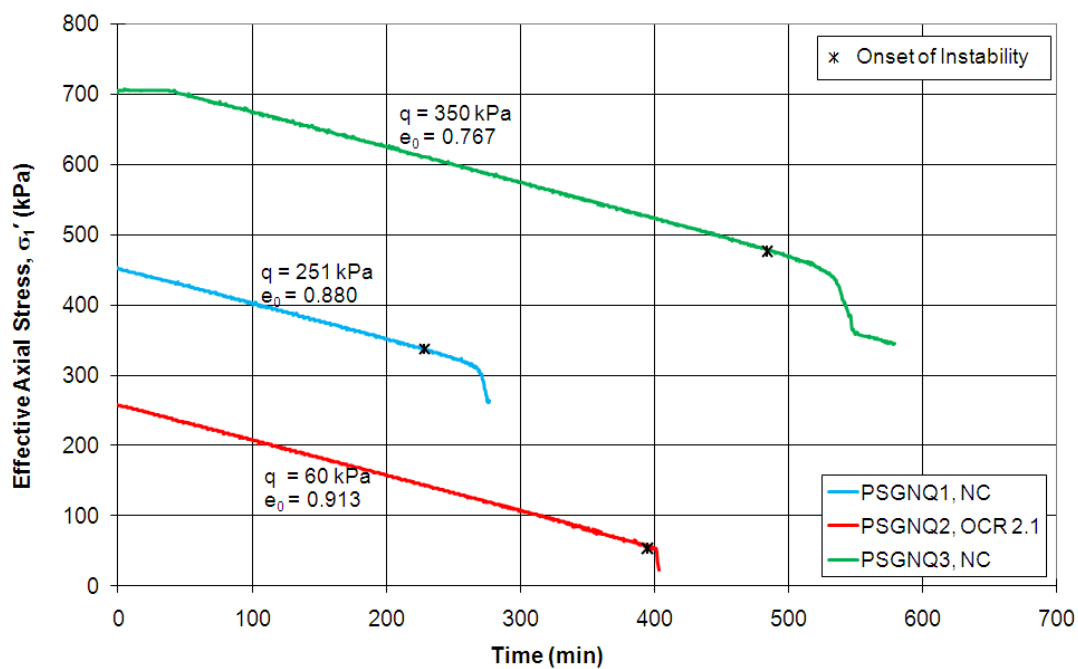


Figure 7.14(a)

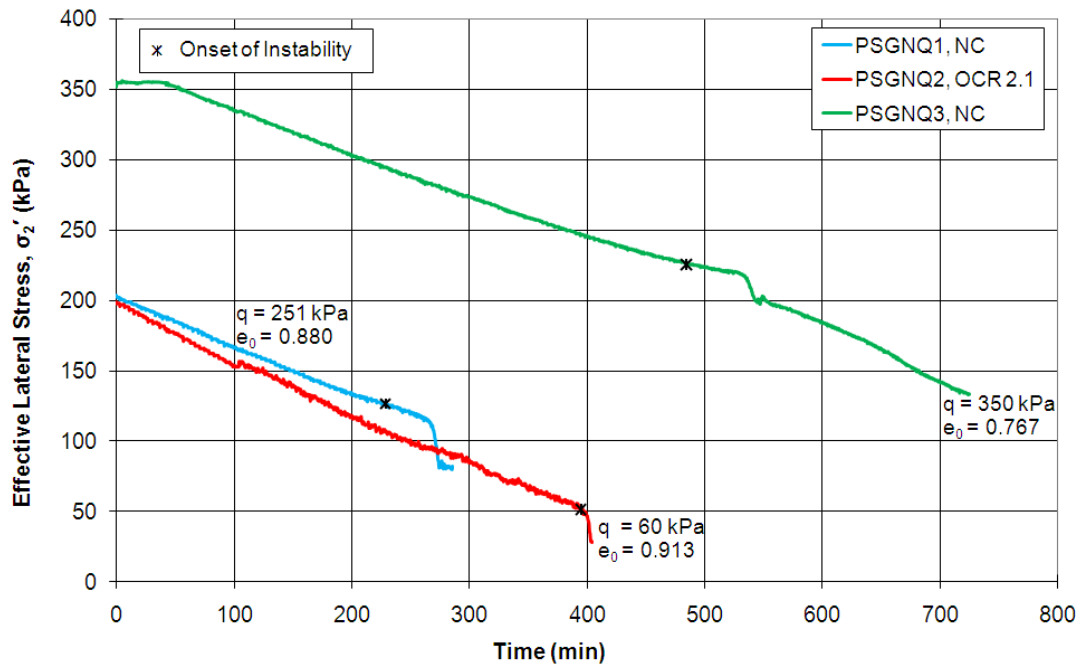


Figure 7.14(b)

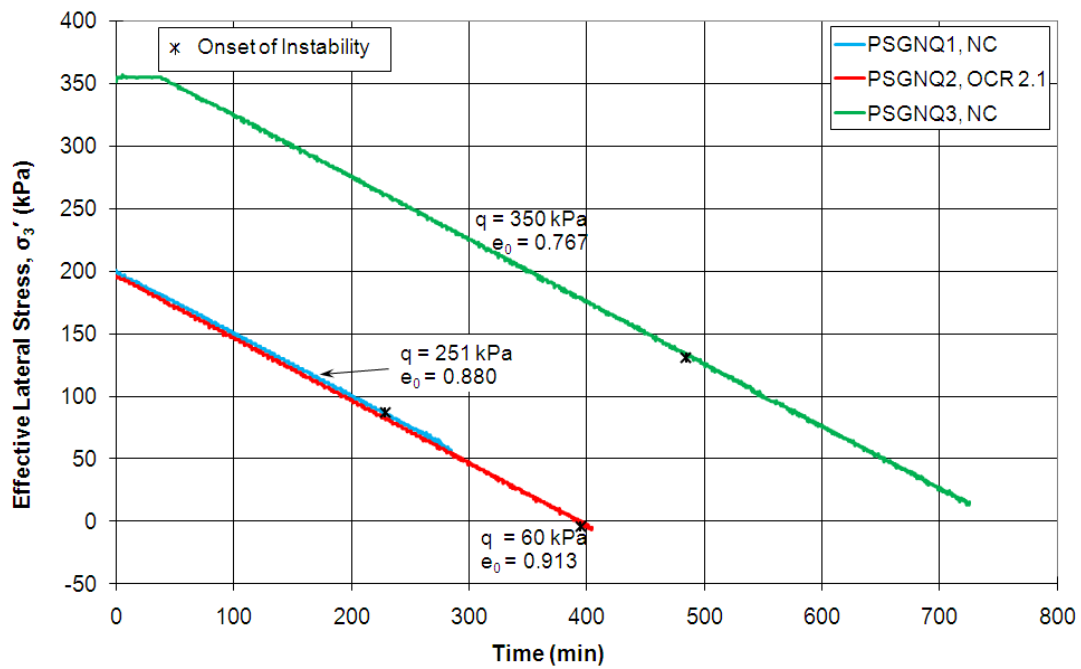


Figure 7.14(c)

Figure 7.14 Variations of Effective Stresses with Time during CSD-PS Tests on Intact Residual Soil: (a) σ_1' ; (b) σ_2' ; (c) σ_3'

7.4 CSD Tests on Artificially Cemented Soil

As mentioned in Chapter 3, the residual soil of weathered Bukit Timah granite had a true cohesion that is small enough to be ignored. Thus, it leads to the question on whether the conditions for instability and hydraulic fracture established for residual soils applicable to cemented soil. In order to study the cementation effect on the instability and hydraulic behaviour, the artificially cemented soil was also tested along a constant stress path. The summary of CSD-T tests conducted using triaxial cell is given in Table 7.5.

Table 7.5 Summary of CSD-T Tests on Artificially Cemented Soil under Triaxial Conditions

Test No.		AGN11	AGN12
Pressure for Saturation (kPa)	Cell	860	860
	Back	850	850
OCR		1.68	1.37
Testing Method		Reduce cell	Reduce cell
Shearing Rate (kPa/min)		0.5	0.5
Weight (g)	Pre-Test	276.27	283.16
	Post-Test	366.84	-
Water Content (Pre-Test) (%)	Average	0	0
Water Content (Post-Test) (%)	Average	32.43	31.41
	Top	33.22	30.2
	Middle	33.77	32.66
	Bottom	30.3	30.93
	Shear Band	-	31.86
σ'_{10} (kPa)		746.152	548.67
q_0 (kPa)		300.15	449.67
p'_0 (kPa)		546.05	248.89
e_0		0.783	0.836
$(q/p')_{\text{peak}}$		2.247	2.245
ε_1 at $(q/p')_{\text{peak}}$ (%)		0.419	0.22
$R_{\text{peak}} = (\sigma'_1/\sigma'_3)_{\text{peak}}$		9.952	9.921
Secant Friction Angle, ϕ' (°)		54.824	54.772

The effective stress paths of the artificially cemented soil are presented in Figure 7.15(a). The specimens were anisotropically consolidated to different OCRs. Failure envelope, critical state line and instability line shown in Figure 7.15(a) were determined from CK_0UC tests conducted on the artificially cement soil specimens, as presented in Chapter 6. It should be pointed out that the failure envelope of artificially cemented soil was highly depended on the OCR. The failure envelope shown in Figure 7.15(a) was determined from a specimen with an OCR value of 1.71, which was close to the value for specimen AGN11. Same as the intact and compacted residual soil, the instability behaviour of artificially cemented soil was observed. A considerable drop in the deviator stress was observed shortly after onset of instability, which led to a decrease in the effective stress ratio, as show in Figure 7.15(f). It can be seen from Figure 7.15(c) and (d), drastic change in specimen volume associated with the initiation of instability. The volumetric strain ($\varepsilon_v\%$) and strain rate ($\Delta\varepsilon_v/\Delta t$) shot up without hesitation. Comparing to the intact and compacted residual soil, the turning points of the ($\varepsilon_v\%$) and $\Delta\varepsilon_v/\Delta t$ vs. time curves were much easier to identify for artificially cemented soil. This is consistent with the large dilation of artificially cemented soil, which was more significant than that of intact and compacted residual soils, especially for the one with an initial state far away from the failure envelope (AGN12). These differences were resulted from the strong cementation of the artificially cemented soil.

The practical criteria of hydraulic fracture were achieved under CSD-T conditions, which are (1) increase in back pressure (which equivalents to reduction in cell pressure as discussed in Section 7.2.2) and (2) abrupt increase in volumetric strain rate (Bjerrum et al., 1972; Bjerrum & Andersen, 1972; Jaworski et al., 1981; Mori & Tamura, 1987; Alfaro & Wong, 2001; Ohta et al., 2007). The significant dilation observed after onset of instability indicated the tendency for artificially cemented soil to fracture. The hydraulic fracture concept is basically related to the instability and failure of soil.

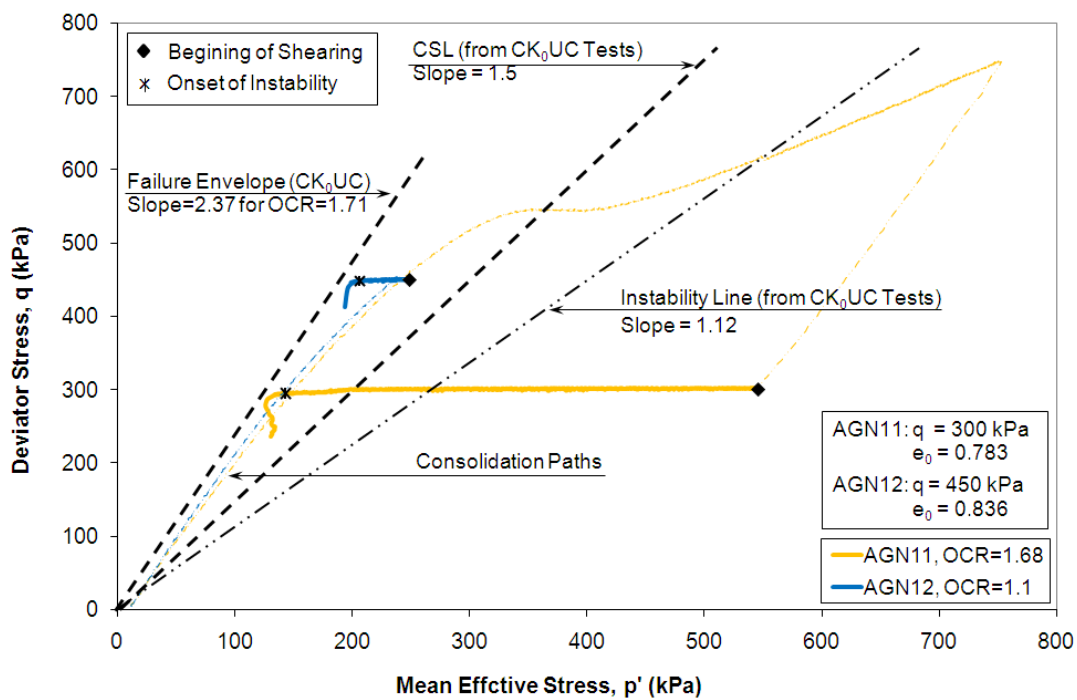


Figure 7.15(a)

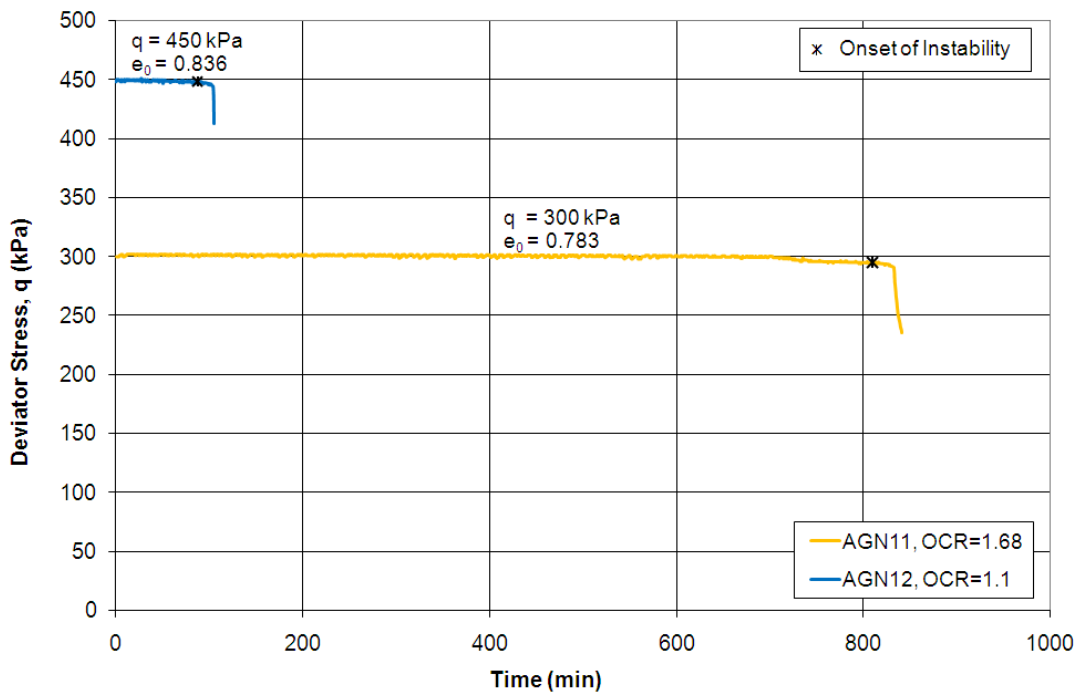


Figure 7.15(b)

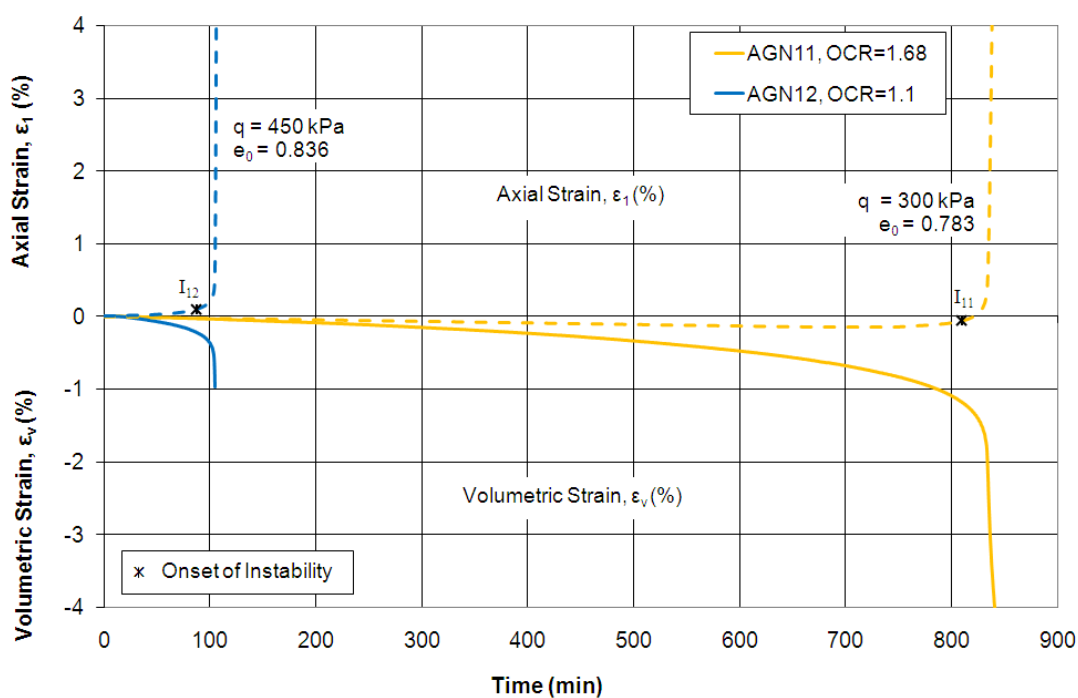


Figure 7.15(c)

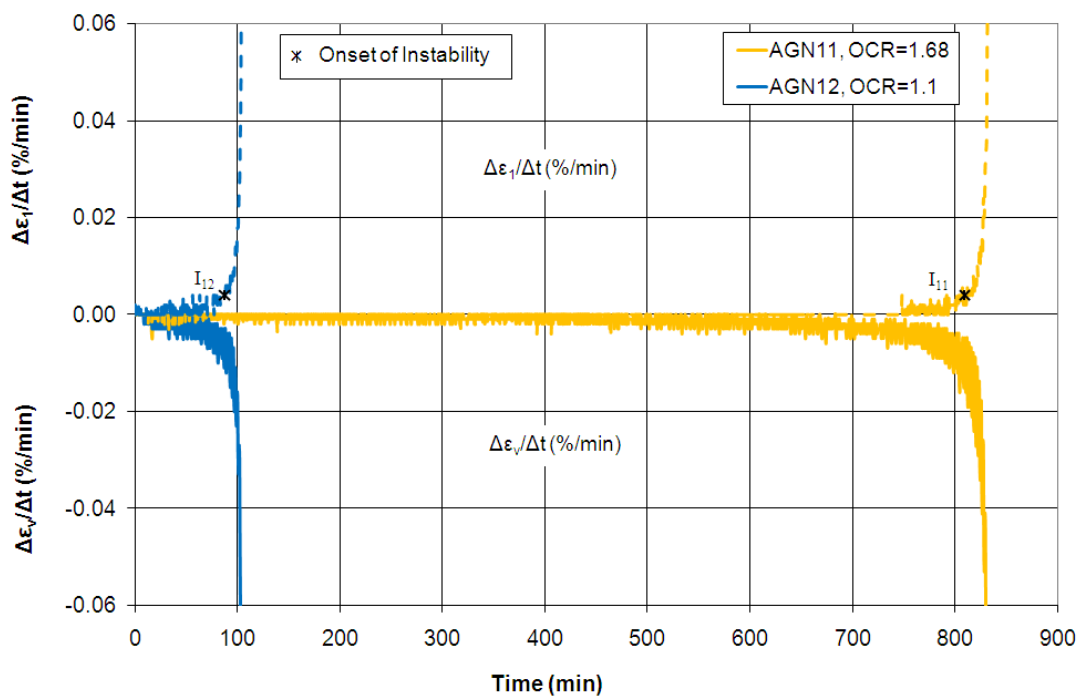


Figure 7.15 (d)

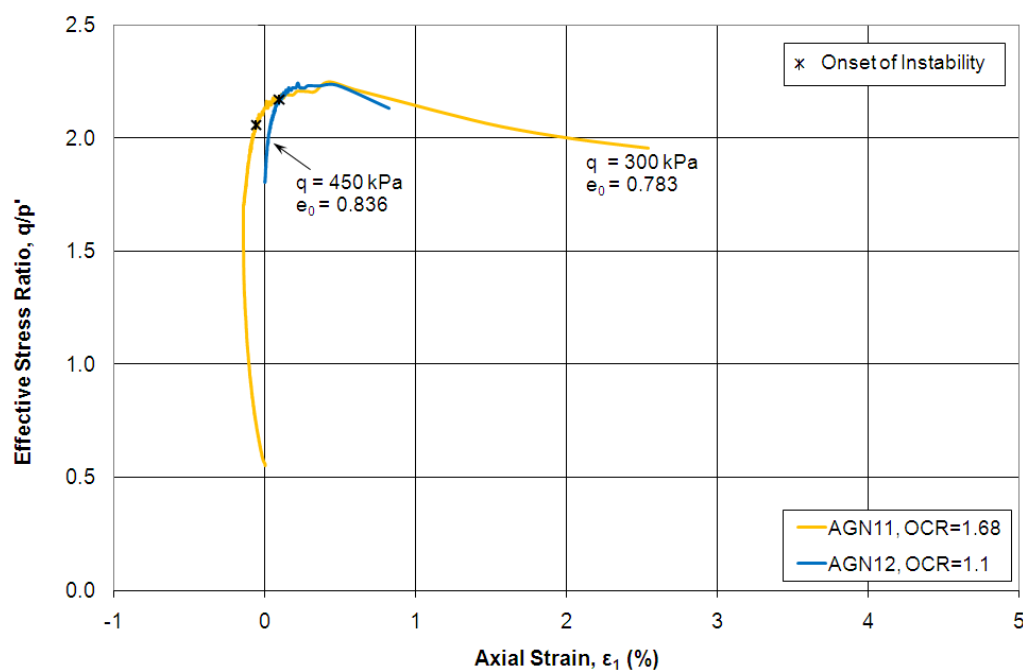


Figure 7.15(e)

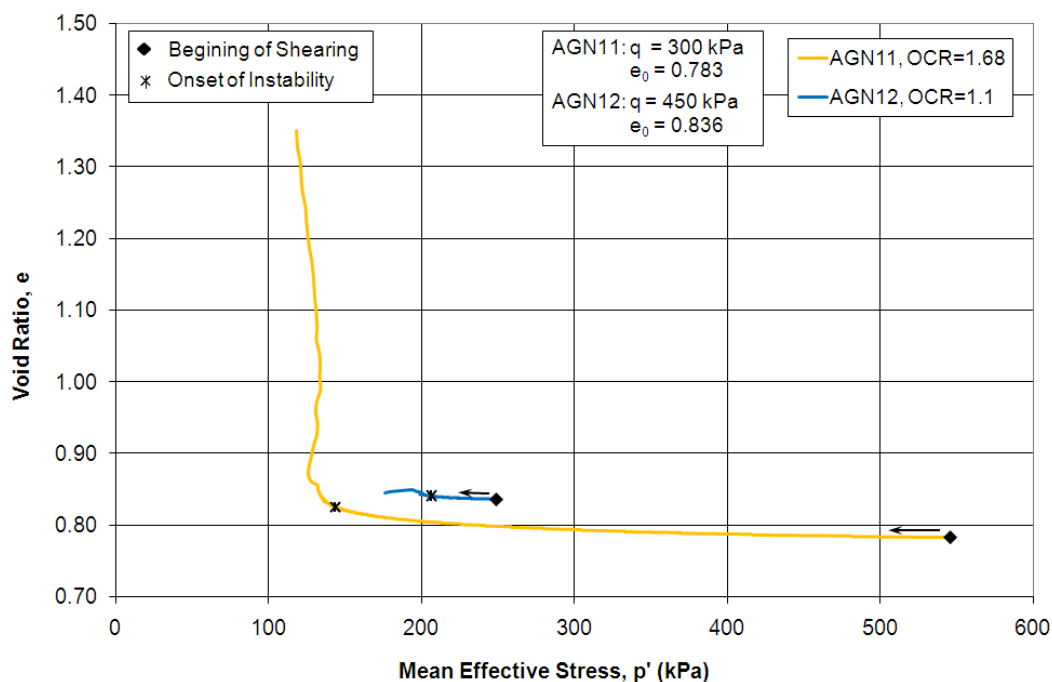


Figure 7.15 (f)

Figure 7.15 General Drained Behaviour of Artificially Cemented Soil Resulted from CSD-T Tests: (a) Effective Stress Paths; (b) q vs. Time Curves; (c) $\epsilon_v\%$ and $\epsilon_1\%$ vs. Time Curves; (d) $\Delta\epsilon_v/\Delta t$ and $\Delta\epsilon_1/\Delta t$ vs. Time Curves; (e) q/p' vs. $\epsilon_1\%$ Curves; (f) e vs. p' Curves

The effective stresses measured during CSD-T tests on the artificially cemented soil specimens are plotted in Figure 7.16. . It confirmed that hydraulic fracture was initiated by shear failure at high confining pressure and differential stress, while Mohr-Coulomb failure criterion was satisfied, rather than tensile failure. Similar observations were made for both compacted residual soil and intact residual soil.

By testing the artificially cemented soil under CSD-T conditions, it is noted that the onset of instability is more obvious for strongly cemented soils. Basically, the artificially cemented soil behaved similarly as the intact and compacted residual soil. Except the instability of the artificially cemented soil resulted in a more significant dilation (i.e. large increase in void ratio) than the soils with weak cementation did, such as intact and compacted residual soils.

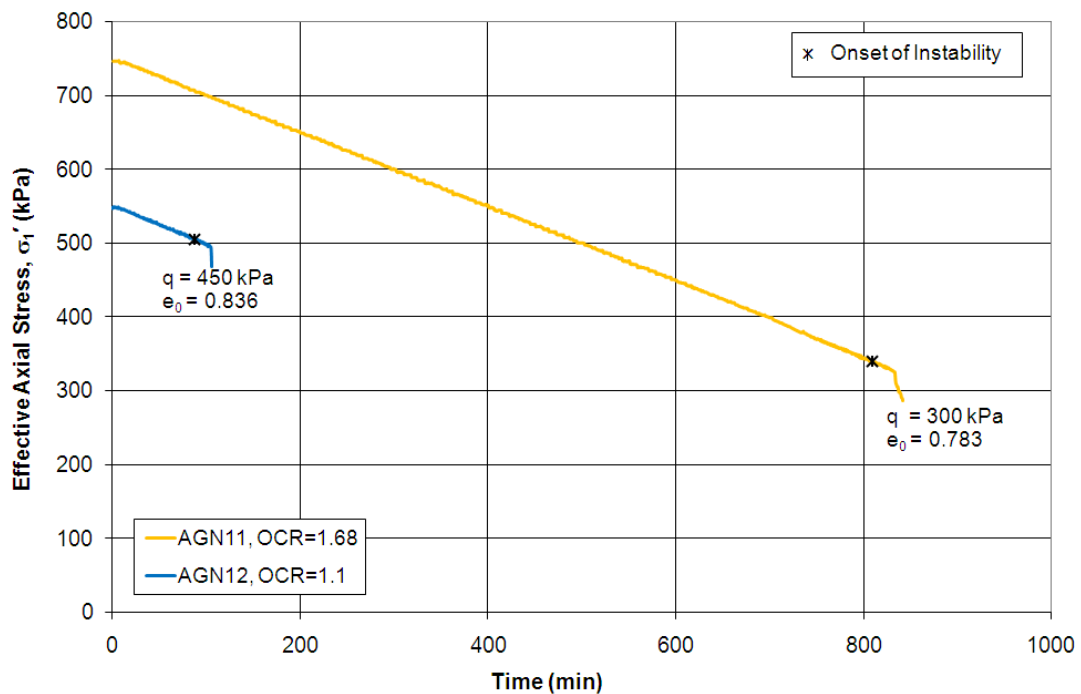


Figure 7.16(a)

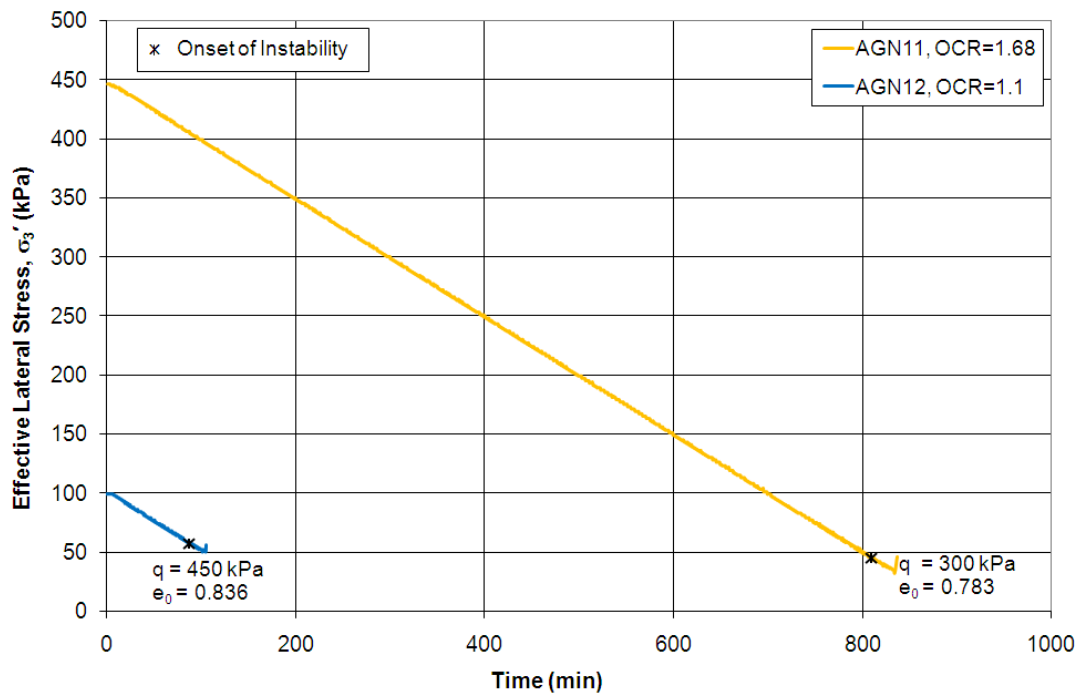


Figure 7.16(b)

Figure 7.16 Variations of Effective Stresses during CSD-T Tests on Artificially Cemented Soil under Triaxial Conditions: (a) σ_1' ; (b) σ_3'

7.5 Conclusions

A series of constant-shear-drained (CSD) tests were conducted under both triaxial and plane-strain conditions to investigate the instability behaviour of residual soils. The CSD tests were also favorable as element study to understand the mechanism of hydraulic fracture. The compacted residual soil was tested first, as the specimen was more uniform and homogenous than the intact residual soil. CSD tests were also carried out under plane-strain conditions and for intact residual soil for comparison. The artificially cemented soil was tested to investigate the cementation effects on the instability and shear band formation behaviour. Instability in the form of a sudden increase in axial strain rate was observed for both NC and OC specimens at the stress level close to the failure envelope. For compacted soil, the initiation of instability was accompanied by a dilation and shear band formation. After shear band formation in CSD tests under plane-strain condition, the effective stresses (σ_1' and σ_3') reduced drastically. These phenomena are similar to the

formation of hydraulic fracture. Therefore, the CSD tests may contribute to the understanding of the mechanism of hydraulic fracture. The condition for instability may be linked to the conditions for the occurrence of hydraulic fracture under plane-strain conditions.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

An experimental study on the strength and deformation behaviour of a residual soil in Singapore was carried out. The proposed study was performed on three types of soils: intact residual soil of Bukit Timah Formation, compacted residual soil and an artificially cemented soil. A number of important observations have been made from the study. The main conclusions and contributions of this thesis are summarized as following:

- 1) The one-dimensional consolidation behaviour of residual soil was measured using triaxial cell, as the conventional oedometer test was not suitable for the testing of the residual soil studied due to the large bedding errors and wall frictions. The pre-consolidation stress (σ_p'), as an important parameter related to the consolidation, deformation and strength behaviour of soil, was determined from the K_0 consolidation tests conducted using a triaxial cell based on the excess pore water pressure versus logarithm of vertical stress, Δu vs. $\log \sigma_1'$ curve. The procedure adopted is similar to a method proposed by DeGroot et al. (2007). By comparing the σ_p' value of each tested soil, it is noted that the σ_p' is significantly affected by soil cementation. Similar observation was also made by (Gutierrz et al., 2008). The one-dimensional consolidation behaviour was also measured using plane-strain apparatus.

The results were in good agreement with those obtained using triaxial K_0 tests

- 2) Based on the 1-D stiffness measured from the triaxial K_0 consolidation tests, two yielding stresses, σ_B' and σ_D' were identified. σ_B' represents the stress level at which the breakage of bonding starts and σ_D' represents the stress level at which most of the bonding has been broken. Therefore, the de-structuring process happens between σ_B' and σ_D' . The pre-consolidation stress, σ_p' , is between σ_B' and σ_D' and may represent the condition where the breakage of bonding has been reached a stage where large plastic strain starts to develop. The variation of K_0 value may be linked to the breakage of bonding of soil. It is also observed for intact residual soil and artificially cemented soil that the K_0 value attains a lower value first, and then increases gradually until an ultimate value is reached. The first yielding point, σ_B' , coincides with the stress level at the lowest K_0 value, which may indicate the initiation of bonding breakdown. The second yielding, σ_D' , is the stress level when K_0 start to approach a constant.
- 3) The K_0 values for NC soil measured from the K_0 tests were compared with that predicted using $K_0 = 1 - \sin\phi_{cs}'$. Good agreements were achieved for all the three soils tested, namely the intact, compacted residual soils, and artificially cemented soil.
- 4) For the intact and the artificially cemented soil, the effective stress path during K_0 consolidation was a curve and was above the K_0 line at the end of the test or the ultimate state. Similar results have also been obtained by other researchers (Maccarini, 1987; Vaughan, 1988; Leroueil & Vaughan, 1990; Toll & Malandraki, 1993; Malandraki & Toll, 2000; Leroueil & Hight, 2003; Toll et al., 2006; Lo & Hinchberger, 2007). To quantify the cementation effect, a cementation index (I_c) was proposed in this study and this index is measured by the distance between the effective stress path and the ultimate K_0 line. By comparing the I_c value for the intact residual soil and artificially

cemented soil, it is observed that the yielding stresses, σ_B' and σ_D' increased with I_c for bonded materials.

- 5) To characterize the strength and stress-strain characteristics of the intact residual soil, a series of CK₀UC, CK₀UE, CK₀UDSS and CK₀UPS tests were conducted on both vertically cut and horizontally cut specimens. The c_u/σ_{v0}' versus OCR relationships were established for each type of tests. The tests conducted on vertically and horizontally cut specimens did not show a strong inherent strength anisotropy in terms of c_u/σ_{v0}' ratio. However, pronounced strength anisotropy both in terms of effective friction angle and undrained shear strength was observed from the tests with major principal stress rotation and plane-strain tests. It is observed from the testing results that, in general, under a given OCR the c_u/σ_{v0}' obtained from the CK₀UPS tests is the highest and this is followed by the CK₀UC tests. The CK₀UE tests give the lowest one, whereas the value obtained from CK₀UDSS tests is somewhere in between. The strength anisotropy could be induced by different degree of bond breakdown due to different volumetric responses with major principal stress rotation. This is consistent with the observations of Malandraki and Toll (2000).
- 6) An artificially cemented soil was tested to study the cementation effect. The experimental results showed that the failure envelope of the artificially cemented soil is highly OCR dependent, especially under CK₀UC conditions. The slope of failure envelope can be as high as 2.52 for specimen with OCR value of 3, which led to a friction angle of 62.5°. It is also noted that at a given OCR value, the c_u/σ_{v0}' obtained from the CK₀UC test is the highest. This is followed by the CK₀UDSS tests. The c_u/σ_{v0}' resulted from extension tests is the lowest. This is consistent with the observation made earlier for the intact residual soil.
- 7) The failure envelope of artificially cemented soil is much higher than that of intact residual soil, under all tested conditions (CK₀UC, CK₀UDSS and

CK₀UE conditions). It is also observed that the artificially cemented soil gave a higher value of $c_u/\sigma_1'{}_0$. However, future studies are required to understand how cementation contributes to the increase in undrained shear strength and how $c_u/\sigma_1'{}_0$ varies under both cementation and OCR effects.

- 8) The pore pressure parameter at failure was calculated for each individual test (A_f for CK₀UC & CK₀UE tests, and a_f for CK₀UPS test) for intact residual soil and the artificially cemented soil. At the NC state, A_f is comparable to a_f . However, with increasing in OCR, the A_f or a_f value decreases significantly as shown in Figure 5.41. The OCR effect was most significant in CK₀UPS test.
- 9) The instability behaviour of residual soil was studied by conducting a series of constant-shear-drained (CSD) tests. The study was mainly carried out on compacted residual soil, as it is related to the use of residual soil as fill for as the construction of embankments or earth dams. However, some tests on intact residual soil and the artificially cemented soil were also carried out. Instability in the form of a sudden increase in axial strain rate was observed for both NC and OC specimens at the stress level close to the failure envelope. For compacted soil, the initiation of instability was accompanied by a dilation and shear band formation. After shear band formation in CSD tests under plane-strain condition, the effective stresses (σ_1' and σ_3') reduced drastically. These phenomena are similar to the formation of hydraulic fracture. Therefore, the CSD tests may contribute to the understanding of the mechanism of hydraulic fracture.
- 10) During CSD tests, a visible shear band was observed under plane-strain condition. The initiation of shear band can be identified by a significant divergence in the four lateral stress (σ_2) readings.

8.2 Recommendations

However, due to the time constraint, several important factors have not been studied. The following future research works are recommended:

- K_0 consolidated undrained plane-strain tests (CK_0 UPS) tests can be conducted under extension conditions. It would make the study on the shear strength and stress-strain characteristics of the intact residual soil a complete one. It would also be useful for the investigations on the strength anisotropy.
- The experimental study presented in this thesis was conducted within a limited range of OCR. To establish a better understanding on general behaviour of undrained shear strength, highly overconsolidated soil specimens should be tested. The inherent strength anisotropy could also be further investigated by conducting tests on both vertically and horizontally cut highly overconsolidated soil specimens.
- For further investigation on the effects of cementation on the strength and deformation behaviour of bonded materials, a comparative study could be carried out between an artificially uncemented soil and the artificially cemented soil used in this project. Different methods to introduce soil cementation could also be adopted for soil preparation, such as by simply mixing Fe-oxides with kaolin powder. The research work carried out by Dr. Susan Elizabeth Burns' Group at Georgia Institute of Technology is worth to study.
- It is observed in this study that for cemented soil, such as intact residual soil and artificially cemented soil, the specimen consolidated to a stress level below preconsolidation stress behaves very differently from those consolidated beyond preconsolidation stress. Higher normalized undrained shear strength was achieved because the cementation was not destroyed by pre-shearing consolidation. This lead to an interesting topic of the influence

of cementation breakdown on the shearing or strength behaviour of cemented soils. However, this is beyond the scope of this proposed project. Further studies on the relevant topics are recommended.

- Chemical analysis on residual soil could be of great help to understand the composition and microstructure, and also to explain the cementation index. The “composition-structure-strength” behaviour of residual soil is worth for further investigation. Future chemical analysis, such as salt concentration, types of cations, mineral contains and PH, could be carried out.
- Some parametric studies could be carried out using computer software. A constitutive model should also be developed or used to model the experimental data obtained in this study.

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APPENDIX I

$$\text{Since } \frac{\sigma_1'}{\sigma_3'} = R = \frac{1 + \sin\phi'}{1 - \sin\phi'} \quad \dots \dots (1)$$

where σ_1' and σ_3' are the major and minor effective principal stress respectively,
and ϕ' is the angle of internal friction in terms of effective stress.

$$\text{and } \frac{q}{p'} = \tan\alpha' = \frac{3R - 3}{2 + R} \quad \dots \dots (2)$$

$$\text{where } q = \sigma_1' - \sigma_3' \text{ and } p' = \frac{\sigma_1' + 2\sigma_3'}{3}$$

By substituting Equation (1) into Equation (2),

$$\begin{aligned} \frac{q}{p'} = \tan\alpha' &= \frac{3R - 3}{2 + R} = \frac{3\left(\frac{1 + \sin\phi'}{1 - \sin\phi'}\right) - 3}{2 + \frac{1 + \sin\phi'}{1 - \sin\phi'}} = \frac{3(1 + \sin\phi') - 3(1 - \sin\phi')}{2(1 - \sin\phi') + 1 + \sin\phi'} \\ &= \frac{6\sin\phi'}{3 - \sin\phi'} \end{aligned}$$

APPENDIX II

One unconsolidated-undrained (UU) test was conducted on the intact residual soil. A confining pressure of 195 kPa was applied, which was equal to the in-situ effective vertical stress. The testing details are listed in Table II-1. And the stress-strain curve is presented in Figure II-1. Undrained shear strength, c_u of 90 kPa was obtained.

Table II-1 Detail Information of UU test

Dimension of specimen (mm)		Water content (%)	Bulk density (Mg/m ³)	Dry density (Mg/m ³)
diameter	height			
38	76	35.62	1.83	1.35

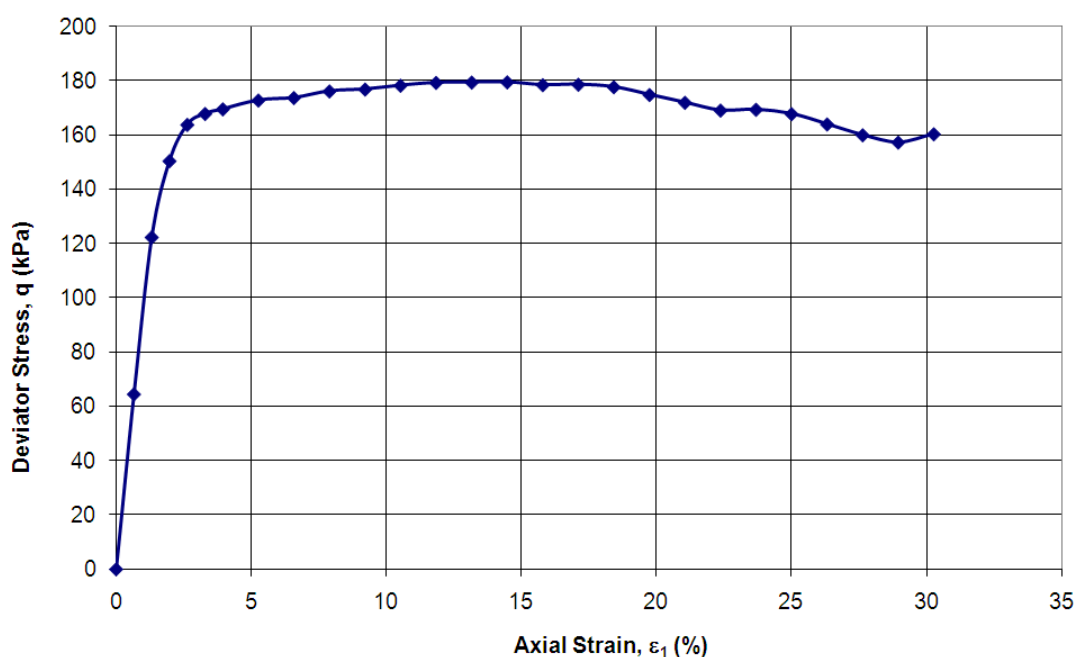


Figure II-1. Stress-Strain Curve Obtained from UU test on Intact Residual Soil