Shear strength and volume change relationship for an unsaturated soil

Trinh Minh Thu

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**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
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**A Thesis submitted to
the Nanyang Technological University
in fulfillment of the requirements for the degree of
Doctor of Philosophy**

2006

To my parents:
Trịnh Việt Miên & Mai Thị Lan
my wife:
Trần Thị Thu Hương
and
my children:
Trịnh Nữ Anna Minh Trâm & Trịnh Minh Tân

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ABSTRACT

Shear strength of unsaturated soil is commonly obtained from a consolidated drained (CD) triaxial test. However in many field situations, fill materials are compacted where the excess pore-air pressure developed during compaction will dissipate instantaneously, but the excess pore-water pressure will dissipate with time. It can be considered that the air phase is generally under a drained condition and the water phase is under an undrained condition during compaction. This condition can be simulated in a constant water content (CW) triaxial test. Comparisons between the shear strength parameters obtained from the CW and the CD triaxial tests have not been extensively investigated.

An elasto-plastic model for unsaturated soil with the incorporation of soil-water characteristic curve (SWCC) was proposed in this study. The proposed model was verified with experimental data. A series of SWCC, isotropic consolidation, the CW and CD triaxial tests were conducted on statically compacted silt specimens in a triaxial cell apparatus. The experimental results from SWCC tests under different net confining stresses showed that the air-entry value and the yield suction increased nonlinearly with the increase in net confining stress. The results of the isotropic consolidation tests indicated that the yield stress increased with the increase in matric suction. The slope of the normally consolidated line (NC), the slope of the unloading curve and the intercept of the consolidation curves at the reference stress decreased with the increase in matric suction.

The results indicated that the effective angles of internal friction, ϕ' , and the effective cohesions, c' , of the compacted silt as obtained from both the CW and CD tests were identical. The results of the CW and CD triaxial tests indicated that the effective angle of internal friction, ϕ' , and the effective cohesion, c' , of the compacted silt were 32° and zero kPa, respectively. The relationships between ϕ^b

and matric suction from the CW and CD triaxial tests on the compacted silt specimens were found to be non-linear. The ϕ^b angle was found to be the same as the effective angle of internal friction, ϕ' (i.e., 32°) at low matric suctions (i.e., matric suctions lower than the air-entry value). The ϕ^b angle decreased to a magnitude as low as 12° at high matric suctions (i.e., matric suctions higher than the residual matric suction). However, the ϕ^b angles from the CW and CD tests were different at matric suctions between the air-entry and the residual matric suction values due to the hysteretic behaviour of the soil-water characteristic curve.

The critical state lines at different matric suctions on the $(q - p)$ plane were parallel with a slope of 1.28 for both the CW and CD triaxial tests, indicating the unique relationship between the deviator stress and mean net stress. The results also indicated the unique relationship between the specific volume and mean net stress on the $(v - p)$ plane for both the CW and CD triaxial tests. The slope of the critical state lines on the $(v - p)$ plane for both the CW and CD triaxial tests decreased with the increase in matric suction.

Reasonably good agreements between the analytical simulations based on the proposed elasto-plastic model with the incorporation of SWCC and the experimental results for the shear strength, the change in pore-water pressure and the volume change during shearing tests were obtained in this study.

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LIST OF SYMBOLS

AEV	: air-entry value
B_w	: pore-water pressure parameter
CD	: consolidated drained triaxial test
CSL	: critical state line
CW	: constant water content triaxial test
D_w	: tangent pore-water pressure parameter
c'	: effective cohesion
e	: void ratio
e_o	: initial void ratio
e_f	: void ratio at failure
E	: modulus of elasticity or Young's modulus for the soil structure with respect to the change in net normal stress
E_w	: water volumetric parameter associated with a change in the net normal stress
ESP	: effective stress path
G	: shear modulus
H	: modulus of elasticity for the soil structure with respect to a change in matric suction
H_w	: water volumetric parameter associated with a change in matric suction
NCL	: normal consolidation line
k_s	: coefficient of permeability at saturated condition
k_w	: coefficient of permeability at unsaturated condition
K	: bulk modulus
R	: overconsolidation ratio
p	: mean total stress
p'	: mean net stress
p_{at}	: atmospheric pressure
p_c'	: preconsolidation stress of unsaturated soil

List of symbols

p_c^{*}	: preconsolidation stress of saturated soil
p^c	: reference stress
p_y'	: mean net stress at the initial yield point
p_f'	: mean net stress at the failure
q	: deviator stress
q_y	: deviator stress at the initial yield point
q_f	: deviator stress at the failure
LC	: loading-collapse yield curve
S	: degree of saturation
s	: matric suction
s_f	: matric suction at failure
$SWCC$	: soil-water characteristic curve
s_0	: yield suction
SI	: suction increase yield curve
TSP	: total stress path
τ	: shear stress
τ_{ff}	: shear stress on the failure plane at failure
τ_{xy}	: shear stress on x -plane in the y direction
τ_{yz}	: shear stress on y -plane in z direction
τ_{zx}	: shear stress on z -plane in x -direction
G	: shear modulus
σ	: total stress
σ_{ff}	: total stress at failure
σ_1	: major principle stress
σ_{1f}	: major principle stress at failure
σ_3	: minor principle stress
σ_{3f}	: minor principle stress at failure
σ_x	: total stress in the x -direction
σ_y	: total stress in the y -direction
σ_z	: total stress in the z -direction
σ_m	: average total normal stress
u_a	: pore-air pressure

List of symbols

u_{af}	: pore-air pressure at failure
u_w	: pore-water pressure
u_{wf}	: pore-water pressure at failure
$(u_a - u_w)$	: matric suction
$(u_a - u_w)_f$	: matric suction at failure
$(\sigma - u_a)$	: net normal stress
$(\sigma - u_a)_f$	: net normal stress at failure
Δu_w	: excess pore-water pressure
Δu_y	: excess pore-water pressure at the initial yield point
Δu_f	: excess pore-water pressure at the failure
m_1^s	: coefficient of volume change with respect to a change in net normal stress
m_2^s	: coefficient of volume change with respect to a change in matric suction
m_1^w	: coefficient of water volume change with respect to net normal stress
m_2^w	: coefficient of water volume change with respect to matric suction
V_a	: volume of air in the soil element
V_0	: initial overall volume of an unsaturated soil element
V_v	: volume of soil voids
V_w	: volume of water in the soil element
v	: specific volume ($v = 1 + e$)
v_w	: specific water volume
v_f	: specific volume at failure
w	: water content
w_{opt}	: optimum water content
w_w	: water content at the wet side
w_d	: water content at the dry side
$\gamma_{d\max}$	: maximum dry density
γ_{dw}	: dry density at the wet side
γ_{dd}	: dry density at the dry side

List of symbols

γ_{xy}	: shear strain on z -plane (i.e., $\gamma_{xy} = \gamma_{yx}$)
γ_{yz}	: shear strain on x -plane (i.e., $\gamma_{yz} = \gamma_{zy}$)
γ_{zx}	: shear strain on y -plane (i.e., $\gamma_{zx} = \gamma_{xz}$)
ε_v	: volumetric strain
ε_x	: normal strain component in x direction
ε_y	: normal strain component in y direction
ε_z	: normal strain component in z direction
$d\varepsilon_v$	: volumetric strain increment
$d\varepsilon_q$	: shear strain increment
$d\varepsilon_1, d\varepsilon_3$	: principal strain increments
$d\varepsilon_v^e$	: elastic volumetric strain increment
$d\varepsilon_v^p$	: plastic volumetric strain increment
$d\varepsilon_q^e$	: elastic shear strain increment
$d\varepsilon_q^p$	: plastic shear strain increment
$d\varepsilon_{v(p)}^e$	: elastic volumetric strain increment induced by changes in mean net stress
$d\varepsilon_{v(s)}^e$	: elastic volumetric strain increment induced by changes in matric suction
$d\varepsilon_{v(p)}^p$	: plastic volumetric strain increment induced by changes in mean net stress
$d\varepsilon_{v(s)}^p$	: plastic volumetric strain increment induced by changes in matric suction
μ	: Poisson's ratio
κ	: slope of unloading-reloading line in $(v - \ln p')$ plane
M	: slope of critical state line in $(q - p')$ plane
M_a	: slope of the critical state line with respect mean net stress
M_w	: slope of the critical state line with respect matric suction
$N(0)$	: specific volume of the normal consolidation curve of saturated soil at reference stress
$N(s)$	: specific volume of the normal consolidation curve of unsaturated soil at reference stress
$N_w(s)$	: specific volume of the normal consolidation curve with respect to water phase of unsaturated soil at reference stress
$\lambda(0)$	: stiffness parameter of the normal consolidation curve at the saturated condition

List of symbols

$\lambda(s)$	: stiffness parameter at the unsaturated condition
$\lambda_w(s)$	: stiffness parameter of the normal consolidation curve with respect to water phase
$\omega(s)$	: slope of the critical state line
$\psi_s(s)$	: Slope of the critical state line for degree of saturation
$\Gamma_s(s)$	: degree of saturation at the reference stress of the critical state line
$\Gamma(0)$	: specific volume on the critical state at the reference stress at saturated condition
$\Gamma(s)$	: specific volume on the critical state at the reference stress at unsaturated condition
Γ_{aw}	: The specific volume when both $(p - u_a)$ and $(u_a - u_w)$ equal unity
ϕ'	: effect angle of internal friction
ϕ^b	: angle indicating the rate of increasing in shear strength relative to changes in matric suction, $(u_a - u_w)_f$
θ_s	: volumetric water content at saturated condition
θ_r	: volumetric water content at residual state
θ_w	: volumetric water content at given matric suction
Θ	: normalized volumetric water content
χ	: a numerical coefficient ranging from 0 to 1

CHAPTER 1 INTRODUCTION

1.1 Background

In many field situations, fill materials are compacted where the excess pore-air pressure developed during compaction will dissipate instantaneously, but the excess pore-water pressure will dissipate with time. It can be considered that the air phase is generally under a drained condition and the water phase is under an undrained condition during compaction. This condition can be simulated in a constant water content (CW) triaxial test. The excess pore-water pressure generated during loading under the constant water content condition is an important aspect that may cause many geotechnical problems such as slope failures. However, shear strength parameters used in geotechnical designs are obtained mainly from the consolidated drained (CD) or consolidated undrained (CU) triaxial tests. In the past few decades numerous researchers (Bishop et al. 1960; Bishop and Donald 1961; Bight 1961; Satija 1978; Sivakumar 1993; Rahardjo et al. 2004) have studied the shear strength characteristics of unsaturated soils under the constant water content condition in a triaxial apparatus. The difficulty of the CW test is associated with the assurance for uniformity of the pore-water pressure during shearing. Bishop et al. (1960) studied the non-uniformities of pore-water pressure in a specimen during shearing by using a mini pore-water pressure probe. The mini pore-water pressure probe was inserted into a hole drilled inside the specimen. The maximum difference between pore-water pressure measurements at the base plate and at mid-height of the specimen was about 30 kPa as reported by Bishop et al. (1960). The characteristics of the excess pore-water pressure along the soil specimen during shearing under the constant water content condition have not

been studied in detail. In addition, comparisons between the shear strength parameters obtained from the CW and the CD triaxial tests have not been extensively investigated.

During loading, the changes in void ratio, e , and water content, w , of an unsaturated soil with respect to the two independent stress state variables, net normal stress (i.e., $(\sigma - u_a)$, where: σ = total stress; u_a = pore-air pressure), and matric suction (i.e., $(u_a - u_w)$, where: u_w = pore-water pressure), can be represented in a graphical form. The volumetric behavior can be described in a three-dimensional state surface (i.e., $\{(\sigma - u_a), V_v / V_0, (u_a - u_w)\}$ space or $\{(\sigma - u_a), V_w / V_0, (u_a - u_w)\}$ space) (where: V_v = volume of void; V_0 = initial volume; and V_w = volume of water). The V_w / V_0 term is equivalent to water content or degree of saturation. The relationship between volumetric water content and matric suction of a soil is commonly known as a soil-water characteristic curve (SWCC). Meanwhile, the V_v / V_0 term is equivalent to void ratio of a soil. The relationship between void ratio and net normal stress is commonly known as a consolidation curve. Normally the measurement of SWCC in the laboratory is conducted under a zero net confining pressure and the consolidation test is performed under a zero matric suction. Therefore, the effects of net confining stress on SWCC and the effects of matric suction on the consolidation curve need to be investigated.

Most of the theoretical development in soil mechanics has been concentrated in saturated soils in the past. As a result, geotechnical engineers are now able to predict saturated soil behaviour in the field or in the laboratory with certain degrees of success. However, the prediction of unsaturated soil behaviour is still very difficult. So far, for practical purposes the prediction of unsaturated soil behaviour is done by either ignoring the unsaturated state or by using empirical formulations. Recently, several theoretical models based on the elasto-plastic theory for predicting the unsaturated soil behaviour have been proposed. The general framework for the constitutive model of unsaturated soil was proposed by Alonso et al. (1987, 1990). The constitutive model proposed by Alonso has

been refined by other researchers (Toll 1990; Gen and Alonso 1992; Thomas and He 1994; Wheeler and Sivakumar 1995; Cui and Delage 1996; Wheeler 1996; Bolzon et al. 1996; Rampino et al. 1999; Simoni and Schrefler 2001; Tang and Graham 2002; Chiu and Ng 2003). These models were developed under the framework of independent stress state variables by using the extended concept of the critical state of saturated soil for unsaturated soils. It has been postulated that matric suction has a significant influence on soil behavior in terms of volume changes, stress-strain and shear strength (Sivakumar 1993; Fredlund et al. 1996; Vanapalli et al. 1996; Bolzon et al. 1996; Gallipoli et al. 2003; Wheeler et al. 2003). Gallipoli et al. (2003) and Wheeler et al. (2003) incorporated matric suction as a single-valued stress state variable in formulating the elasto-plastic model for unsaturated soils. However, Fredlund and Morgenstern (1977) suggested that matric suction should be treated as one of the two independent stress state variables for unsaturated soil. Fredlund and Morgenstern (1977) proposed that the constitutive behaviour of unsaturated soils be described using two independent stress state variables; namely, net normal stress, $(\sigma - u_a)$, and matric suction, $(u_a - u_w)$. SWCC relates volumetric water content to matric suction and this relationship has been found to play a significant role in controlling the behaviour of an unsaturated soil. Therefore, an elasto-plastic model that incorporates SWCC for unsaturated soil could be developed.

1.2 Objectives and Scope of the Research

The main objective of this research was to study the characteristics and the relationships between shear strength, pore-water pressure and volume change of an unsaturated soil under constant water content (CW) and consolidated drained (CD) conditions.

On the whole, the scope of the study involved two main parts, i.e., laboratory works and theoretical development. The first part of the laboratory works involved the investigation of the effects of net confining stress and initial dry density on the characteristics of SWCC. The effects of matric suction and initial dry density on the characteristics of the isotropic consolidation curves were also

investigated. The second and the most important part of the laboratory works involved the investigation of the characteristics of shear strength, volume change and excess pore-water pressure during shearing in constant water content and consolidated drained triaxial tests. The shear strength parameters that were obtained from the CW and CD triaxial tests were compared and studied in detail. The theoretical development involved the development of an elasto-plastic model with the incorporation of SWCC for describing the shear strength, excess pore-water pressure and volume change of an unsaturated soil during shearing tests. The proposed elasto-plastic model with the incorporation of SWCC was then used to simulate the experimental data obtained from the laboratory works carried out in this study and those data available from literature.

1.3 Methodology

The study mainly focused on triaxial laboratory tests and the development of the elasto-plastic model with the incorporation of SWCC. The tests for obtaining SWCC were conducted under different net confining stresses and at different dry densities in a modified triaxial apparatus. The isotropic consolidation tests were also conducted under different matric suctions and at different initial dry densities using a modified triaxial apparatus. In order to obtain the shear strength characteristics on an unsaturated soil under the constant water content and consolidated drained conditions, a triaxial test apparatus had to be modified. In a constant water content test for an unsaturated soil, the specimen was sheared under a drained condition for the air phase and an undrained condition for the water phase. Meanwhile in a consolidated drained test, the specimen was sheared under a drained condition for both the air and water phases. Reconstituted silt was used to minimise the heterogeneity of soil. Identical specimens of statically compacted silt were used in this study. The concept of axis translation technique was adopted to control matric suction in the soil specimens. Three NTU mini suction probes were installed along the soil specimen to measure pore-water pressures during the saturation, consolidation, matric suction equalization and the shearing stages. The elasto-plastic model with the incorporation of SWCC was developed and used to simulate the shear

strength, excess pore-water pressure and volume change developed during both the constant water content and the consolidated drained triaxial tests.

1.4 Outline of the Thesis

This thesis is organized into seven chapters:

Chapter 1 contains the introduction, objectives, scope, methodology of the research and the outline of the thesis.

Chapter 2 presents a brief review of unsaturated soil mechanics, examines the available literature on the shear strength, volume change and pore-water pressure characteristics and the critical state models for saturated and unsaturated soils. The review of the characteristics of SWCC and isotropic consolidation curve and the measurements of pore-water pressure during testing of saturated and unsaturated soils is also presented.

Chapter 3 presents the development of the proposed elasto-plastic model with the incorporation of SWCC.

Chapter 4 describes the modification of triaxial apparatus, preparation of specimens and procedures for the testing of basic soil properties, the CW and the CD triaxial tests. All the equipment preparations as well as calibrations of the devices are presented in this chapter. The testing programmes are also presented at the end of this chapter.

Chapter 5 presents the results of triaxial shearing under the constant water content and the consolidated drained conditions. The effects of net confining stress and initial dry densities on SWCC are presented. The effects of matric suction and initial dry densities on isotropic consolidation curve and the basic soil properties are also presented in this chapter.

Chapter 6 contains the discussions of the results presented in Chapter 5. The CW and CD triaxial test results are presented in the form of the critical state. The simulated results of soil response during shearing under the constant water

content and consolidated drained conditions as obtained using the proposed model in Chapter 3 are also presented. The simulation results of the CW and CD triaxial tests are compared with the measurement results.

Chapter 7 presents the overall conclusions and recommendations of the study. The summary of the findings of this research and some recommendations related to the soil testing and theoretical simulations as well as the potential scope for further studies are presented.

Five appendices are included in this thesis:

Appendix A provides the calibration data of the modified triaxial apparatus for obtaining SWCCs.

Appendix B presents the calibration data of the modified triaxial apparatus for obtaining isotropic consolidation curves.

Appendix C gives the calibration data of the modified triaxial apparatus, the CW and the CD triaxial test results.

Appendix D presents additional results of the simulation for the CW triaxial tests.

Appendix E presents additional results of the simulation for the CD triaxial tests.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

The literature review on the topics related to this research is presented in this chapter. A brief review on soil-water characteristics (SWCC) and isotropic consolidation tests is given first. Subsequently, volume change, shear strength and matric suction in unsaturated soils are presented in the following sections. The critical state theories for saturated and unsaturated soils are reviewed in the last section.

2.2 Stress State Variables

Bishop and Donald (1961) verified the stress state variables for unsaturated soils ($(\sigma - u_a)$ and $(u_a - u_w)$) by conducting null-type tests with respect to the shear strength of an unsaturated silt. Lambe and Whitman (1979) defined an unsaturated soil as a three-phase system, which consists of solid, water and air phases. Fredlund and Morgenstern (1977) proposed three possible pairs of stress state variables to describe the mechanical behaviour of unsaturated soils, i.e., shear strength and volume change with the assumption that contractile skin is an additional fourth phase for an unsaturated soil. Contractile skin can be described as an air-water interface with pore-air and pore-water pressures acting across it. The three pairs of stress state variables: $(\sigma - u_a)$ and $(u_a - u_w)$; $(\sigma - u_w)$ and $(u_a - u_w)$; $(\sigma - u_a)$ and $(\sigma - u_w)$ are suitable stress state variables for describing stress-strain-strength behaviour of unsaturated soils.

where:

σ = total stress,

u_a = pore-air pressure, and

u_w = pore-water pressure.

In an unsaturated condition, the term $(u_a - u_w)$ is a positive quantity and referred to as matric suction. The proposed stress state variables; namely, net normal stress, $(\sigma - u_a)$ and matric suction, $(u_a - u_w)$ for unsaturated soil were experimentally tested by Fredlund (1973) and have become well accepted in unsaturated soil mechanics.

2.3 Soil-water Characteristic Curve

SWCC relates gravimetric water content, w , or volumetric water content, θ_w , to matric suction. The volumetric water content is defined as the ratio between volume of water and total volume of the soil (i.e., $\theta_w = V_w / V_0$). Figure 2.1 shows a typical SWCC. The air-entry value (AEV) is defined as the matric suction value at which air first enters the pores of the soil. When the matric suction is increased from zero to the AEV of the soil, the volumetric water content of the soil is nearly constant, but it decreases rapidly when the matric suction becomes greater than the AEV. The volumetric water content will again decrease slowly after the residual volumetric water content is reached. The residual water content is defined as the water content at which further increases in matric suction do not result in significant decreases in the volumetric water content, and the matric suction at this point is correspondingly called residual soil suction.

Figure 2.1 A typical soil-water characteristic curve

Measurement of SWCC in the laboratory is commonly conducted using a Tempe cell or a pressure plate apparatus by following the procedure described in ASTM D2325-68 (1994) (Fredlund and Rahardjo, 1993). The apparatus uses the axis-translation technique proposed by Hilf (1956). The air pressure is applied to the soil specimen that is placed on a high air-entry ceramic disk in a pressure plate. During the test, the air-pressure, u_a , in the pressure plate is raised to a value above atmospheric pressure while the water pressure inside compartment below the high air-entry ceramic disk is kept at atmospheric (i.e., $u_w = 0$). The high-air entry disk allows the air phase to be separated from the water phase. It means that the high-air entry disk only allows water to pass through but not the air phase as long as the air pressure is less than the air-entry value of the disk. The difference between the air and water pressures is known as matric suction, $(u_a - u_w)$. When the air pressure is applied to the soil specimen, the pore-water is drained out of the specimen. When the matric suction of a soil is increased, air replaces water in soil pores and as a result, the water content of the soil decreases. At equilibrium, the soil has a water content corresponds to the controlled matric suction. Therefore, a soil-water characteristic curve is obtained by measuring water contents of the specimen at different matric suctions.

The SWCC determined from pressure plate or Tempe cell can only be performed

under a zero net confining pressure and the volume of the soil specimen is normally assumed to remain constant during the test. However, the resulting SWCC obtained under a zero net confining pressure may not always be suitable for many field conditions. The main advantage of obtaining SWCC using the Tempe cell and pressure plate is that the apparatus is simple and easy to operate.

2.4 Consolidation Tests and the Controlling Factors

The conventional oedometer test for saturated soil specimen was carried out following the test procedure given in ASTM D2435 (1985). The modified oedometer apparatus has been used for one-dimensional consolidation tests on unsaturated specimens. Barden and Sides (1970) modified Rowe cell for applying matric suction during one-dimensional consolidation tests on unsaturated soil specimens. This modified apparatus was used by other researchers including Fredlund and Morgenstern (1977) and Escario and Juca (1989). However, it was difficult to control the matric suction during the test. Recently, Alonso et al. (1990); Sivakumar (1993); Wheeler and Sivakurma (1995); Rampino et al. (1999) conducted isotropic consolidation tests on unsaturated soil specimens using triaxial apparatus under different matric suctions. The isotropic loading was applied by increasing the cell pressure under a constant rate. However, the research on the isotropic consolidation tests of unsaturated soil specimens using triaxial apparatus under different matric suctions is currently limited.

Traditionally, the preconsolidation stress of a soil, p'_c , is determined from the results of oedometer test. Various methods for determination of p'_c from the oedometer test results are available (Casagrande, 1936; Schmertmann, 1955; and Janbu et al., 1981). Most researchers (Leroueil et al., (1983); Sivakumar, (1990); Rampino et al., (2000)) have used the casagrande's construction method to determine the yield stress, p'_c , for the consolidation tests.

2.5 Volume Change of Unsaturated Soils

2.5.1 General

Many constitutive relationships have been proposed for volume change of unsaturated soil (Terzaghi, 1936; Biot, 1941; Bishop, 1959; Coleman, 1962; Bishop and Blight, 1963; Matyas and Radhakrishna, 1968; Barden et al., 1969; Aitchison et al., 1973; Fredlund and Morgenstern, 1976; Llorett and Alonso, 1985; Ho, 1988; Fredlund and Rahardjo, 1993; Wheeler and Sivakumar, 1995). The proposed constitutive relationships have been presented in the form of mathematical equations and/or presented graphically. The volumetric behaviour can be described in a three-dimensional $\{(\sigma - u_a), e, (u_a - u_w)\}$ space as state surfaces. These surfaces are able to describe the transition from swelling to collapse behaviour when the total stress increases (Matyas and Radhakrishna, 1968).

In general, two constitutive relationships are presented to describe the volume change associated with an unsaturated soil; one relationship for the soil structure (in terms of void ratio or volumetric strain) and another relationship for the water phase (in terms of degree of saturation or water content). These relationships are expressed in terms of the total stress, σ , pore-air pressure, u_a , and pore-water pressure, u_w . The constitutive relationships for unsaturated soils should be expressed using two independent stress state variables. The constitutive relationships proposed by Fredlund and Morgenstern (1976) appeared to be satisfactory to explain the behaviour of unsaturated soils.

2.5.2 Constitutive relationships

The continuity requirement for an element of unsaturated soil can be expressed as follows (Fredlund and Morgenstern, 1976):

$$\frac{\Delta V_v}{V_o} = \frac{\Delta V_w}{V_o} + \frac{\Delta V_a}{V_o} \quad (2.1)$$

where:

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- V_v = volume of soil voids,
 V_o = initial overall volume of an unsaturated soil element,
 V_w = volume of water in the element, and
 V_a = volume of air in the element.

The total volume change can be written as the sum of the normal strains:

$$\varepsilon_v = \frac{\Delta V_v}{V_o} = \varepsilon_x + \varepsilon_y + \varepsilon_z \quad (2.2)$$

where:

- ε_v = volumetric strain, and
 ε_x = normal strain component in x direction,
 ε_y = normal strain component in y direction, and
 ε_z = normal strain component in z direction.

In saturated soil mechanics, the constitutive relations for the soil structure can be formulated in accordance with the generalized Hooke's law using the effective stress variable, $(\sigma - u_w)$. For an isotropic and linearly elastic soil structure, the constitutive relations in the x -, y -, and z - directions can be expressed using the following equations:

$$\varepsilon_x = \frac{(\sigma_x - u_w)}{E} - \frac{\mu}{E}(\sigma_y - \sigma_z - 2u_w) \quad (2.3)$$

$$\varepsilon_y = \frac{(\sigma_y - u_w)}{E} - \frac{\mu}{E}(\sigma_x - \sigma_z - 2u_w) \quad (2.4)$$

$$\varepsilon_z = \frac{(\sigma_z - u_w)}{E} - \frac{\mu}{E}(\sigma_x - \sigma_y - 2u_w) \quad (2.5)$$

where:

- σ_x = total stress in the x-direction,
 σ_y = total stress in the y-direction,

σ_z = total stress in the z-direction,

E = modulus of elasticity or Young's modulus for the soil structure, and

μ = Poisson's ratio.

2.5.2.1 Soil Structure Constitutive Relationship

The constitutive relations for an unsaturated soil can be formulated as an extension of the equations used for saturated soil, using the appropriate stress state variables. The soil structure constitutive relations associated with the normal strains in the x -, y -, and z - directions (Fredlund and Morgenstem, 1976; Fredlund, 1979) are as follows:

$$\varepsilon_x = \frac{\sigma_x - u_a}{E} - \frac{\mu}{E}(\sigma_y + \sigma_z - 2u_a) + \frac{(u_a - u_w)}{H} \quad (2.6)$$

$$\varepsilon_y = \frac{\sigma_y - u_a}{E} - \frac{\mu}{E}(\sigma_x + \sigma_z - 2u_a) + \frac{(u_a - u_w)}{H} \quad (2.7)$$

$$\varepsilon_z = \frac{\sigma_z - u_a}{E} - \frac{\mu}{E}(\sigma_x + \sigma_y - 2u_a) + \frac{(u_a - u_w)}{H} \quad (2.8)$$

where:

H = modulus of elasticity for the soil structure with respect to a change in matric suction, $(u_a - u_w)$, and,

E = modulus of elasticity for the soil structure with respect to a change in the net normal stress, $(\sigma - u_a)$.

The constitutive equations associated with the shear deformation are as follows,

$$\gamma_{xy} = \frac{\tau_{xy}}{G} \quad (2.9)$$

$$\gamma_{yz} = \frac{\tau_{yz}}{G} \quad (2.10)$$

$$\gamma_{zx} = \frac{\tau_{zx}}{G} \quad (2.11)$$

where:

- γ_{xy} = shear strain on z-plane (i.e., $\gamma_{xy} = \gamma_{yx}$)
- γ_{yz} = shear strain on x-plane (i.e., $\gamma_{yz} = \gamma_{zy}$)
- γ_{zx} = shear strain on y-plane (i.e., $\gamma_{zx} = \gamma_{xz}$)
- τ_{xy} = shear stress on x-plane in the y-direction (i. e., $\tau_{xy} = \tau_{yx}$)
- τ_{yz} = shear stress on y-plane in the z-direction (i. e., $\tau_{yz} = \tau_{zy}$)
- τ_{zx} = shear stress on z-plane in the x-direction (i. e., $\tau_{zx} = \tau_{xz}$), and
- G = shear modulus.

The constitutive equations can also be applied to the situations where the stresses versus strain relationships are non-linear. An incremental procedure using small increments of stress and strain can be used to apply the linear elastic formulation to a non-linear stress versus strain curve. The non-linear stress versus strain curve is assumed to be linear within each stress and strain increment. The elastic moduli, E and H , may vary in magnitude from one increment to another. The soil structure constitutive relations associated with the normal strains can be written in an incremental form as follows:

$$d\varepsilon_x = \frac{d(\sigma_x - u_a)}{E} - \frac{\mu}{E} d(\sigma_y + \sigma_z - 2u_a) + \frac{d(u_a - u_w)}{H} \quad (2.12)$$

$$d\varepsilon_y = \frac{d(\sigma_y - u_a)}{E} - \frac{\mu}{E} d(\sigma_x + \sigma_z - 2u_a) + \frac{d(u_a - u_w)}{H} \quad (2.13)$$

$$d\varepsilon_z = \frac{d(\sigma_z - u_a)}{E} - \frac{\mu}{E} d(\sigma_x + \sigma_y - 2u_a) + \frac{d(u_a - u_w)}{H} \quad (2.14)$$

A change in the volumetric strain of the soil for each increment, $d\varepsilon_v$, can be obtained by summing the changes in normal strains in the x-, y-, and z-directions:

$$d\varepsilon_v = d\varepsilon_x + d\varepsilon_y + d\varepsilon_z \quad (2.15)$$

where:

$d\varepsilon_v$ = change in volumetric strain for each increment.

Substituting Equations (2.12), (2.13), and (2.14) into Equation (2.15) gives the volumetric strain for a particular loading increment:

$$d\varepsilon_v = 3\left(\frac{1-2\mu}{E}\right)d\left(\frac{\sigma_x + \sigma_y + \sigma_z}{3} - u_a\right) + \frac{3}{H}d(u_a - u_w) \quad (2.16)$$

Equation (2.16) can be simplified to the following form:

$$d\varepsilon_v = 3\left(\frac{1-2\mu}{E}\right)d(\sigma_m - u_a) + \frac{3}{H}d(u_a - u_w) \quad (2.17)$$

where:

σ_m = average total normal stress (i.e., $(\sigma_x + \sigma_y + \sigma_z)/3$).

Fredlund and Rahardjo (1993) proposed the constitutive equation for soil structure in a compressibility form for the three-dimensional loading conditions as follows:

$$d\varepsilon_v = m_1^s d(\sigma_m - u_a) + m_2^s d(u_a - u_w) \quad (2.18)$$

where:

m_1^s = coefficient of volume change with respect to a change in net normal stress (i.e., $m_1^s = \frac{3(1-2\mu)}{E}$), and

m_2^s = coefficient of volume change with respect to a change in matric suction (i.e., $m_2^s = \frac{3}{H}$).

These constitutive equations have been experimentally tested for uniqueness (Fredlund and Morgenstern, 1976; Matyas and Radhakrishna, 1968; Barden et al., 1969). The results indicate uniqueness as long as the deformation conditions are monotonic. The loading constitutive relationship for soil structure is presented

graphically in the form of constitutive surfaces in Figure 2.2.

Figure 2.2 Constitutive surfaces for an unsaturated soil (a) Soil structure constitutive surface; (b) water phase constitutive surface (after Fredlund and Rahardjo, 1993)

2.5.2.2 Water Phase Constitutive Relationship

The water phase constitutive relationship describes the change in the volume of water present in soil element under various stress conditions. Fredlund and Rahardjo (1993) formulated the water phase constitutive relationship using a semi-empirical approach based on a linear combination of the stress state variables. In an elasticity form, the constitutive equation can be written as follows:

$$\frac{dV_w}{V_o} = \frac{3}{E_w} d(\sigma_m - u_a) + \frac{1}{H_w} d(u_a - u_w) \quad (2.19)$$

where:

E_w = water volumetric parameter associated with a change in the net normal stress (i.e., $(\sigma - u_a)$), and

H_w = water volumetric parameter associated with a change in matric suction (i.e., $(u_a - u_w)$).

The compressibility form for the water constitutive relationship under a general loading is as follows:

$$\frac{dV_w}{V_o} = m_1^w d(\sigma_m - u_a) + m_2^w d(u_a - u_w) \quad (2.20)$$

where:

m_1^w = coefficient of water volume change with respect to net normal stress, and

m_2^w = coefficient of water volume change with respect to matric suction.

2.6 Shear Strength of Unsaturated Soils

2.6.1 Shear Strength Equation

Bishop (1959) proposed shear strength equation for unsaturated soil as follows:

$$\tau = c' + [(\sigma - u_a) + \chi(u_a - u_w)] \tan \phi' \quad (2.21)$$

where:

c' = effective cohesion of saturated soil,

ϕ' = the effective of friction angle of saturated soil,

σ = total normal stress,

χ = a numerical coefficient ranging from 0 to 1,

u_a = pore-air pressure, and

u_w = pore-water pressure.

Fredlund et al. (1978) proposed a shear strength equation using two stress state variables (i.e., net normal stress, $(\sigma - u_a)$ and matric suction, $(u_a - u_w)$) as follows:

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan \phi' + (u_a - u_w)_f \tan \phi^b \quad (2.22)$$

where:

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- τ_{ff} = shear stress on the failure plane at failure,
- c' = intercept of the “extended” Mohr-Coulomb failure envelope on the shear stress axis when the net normal stress and the matric suction at failure are equal to zero. It is also referred to as the “effective cohesion”,
- $(\sigma_f - u_a)_f$ = net normal stress on the failure plane at failure,
- σ_f = total normal stress on the failure plane at failure,
- u_a = pore-air pressure at failure,
- ϕ' = angle of internal friction associated with the net normal stress state variable, $(\sigma_f - u_a)$,
- $(u_a - u_w)_f$ = matric suction at failure,
- u_w = pore-water pressure at failure, and
- ϕ^b = angle indicating the rate of change in shear strength relative to changes in matric suction, $(u_a - u_w)_f$.

Equation (2.22) can be considered as an extension of the shear strength equation for a saturated soil. For an unsaturated soil u_w is smaller than u_a . At saturation u_w approaches u_a , causing matric suction to approach zero (i.e., $(u_a - u_w)$). At saturation condition (i.e., $(u_a - u_w) = 0$) Equation (2.22) reduces to the familiar shear strength of a saturated soil as follows:

$$\tau_{ff} = c' + (\sigma_f - u_w)_f \tan \phi' \quad (2.23)$$

The Mohr-Coulomb envelope for saturated soils is commonly drawn as a two dimensional graph of shear stress versus effective normal stress (Figure 2.3). The

failure envelope for unsaturated soils can be described in a three-dimensional plot. The three dimensional plot of failure envelope for unsaturated soils is an extension of Figure 2.3 into the third dimension for the matric suction, $(u_a - u_w)$, axis which is illustrated in Figure 2.4. The angle relating the shear strength with respect to the matric suction (i.e., $(u_a - u_w)$) is characterized by ϕ^b . The angle of internal friction with respect to the net normal stress, $(\sigma - u_a)$, is defined by the angle ϕ' .

Figure 2.3 Mohr-Coulomb failure envelope for saturated soils (after Fredlund and Rahardjo, 1993)

Figure 2.4 Extended Mohr-Coulomb failure envelope for unsaturated soils (after Fredlund and Rahardjo, 1993)

Results of several shear strength tests on unsaturated soils have indicated some non-linearity of ϕ^b over a large range of matric suction (Escario and Saez, 1986; Gan, 1986; Fredlund et al., 1987; Gan et al., 1988). Generally, the tangent of ϕ^b is less than or equal to ϕ' . However, some other researchers (Gan, 1986; Escario and Saez, 1986) found that ϕ^b can be greater than ϕ' angle.

Gan (1986) conducted a multistage direct shear tests on an unsaturated glacial till. The result showed non-linearity of the failure envelope on the shear stress versus matric suction plot as shown in Figure 2.5. The angle of the failure envelope, ϕ^b , is 25.5° at low matric suctions and decreases to $5^\circ - 7^\circ$ at high matric suctions.

Figure 2.5 Failure envelope for unsaturated soil glacial till specimens. (a) Failure envelope on the, τ , against $(u_a - u_w)$ plane; (b) ϕ^b values versus matric suction (after Gan, 1986)

Fredlund et al. (1987) reanalyzed the shear strength data presented by Satija (1978). A re-evaluation was performed by assuming a curved failure envelope with respect to matric suction as shown in Figure 2.6 and Figure 2.7. The curved failure envelope have a slope angle, ϕ^b , equal to, ϕ' , starting at zero matric suction. The ϕ^b angle begins to decrease significantly at matric suction values greater than 50 kPa.

Figure 2.6 Non-linearity in the failure envelope for compacted Dhanauri clay at low-density. (a) The stress strength, τ , plotted against $(u_a - u_w)$; (b) ϕ^b values for various $(u_a - u_w)$ (after Satija, 1978)

Figure 2.7 Nonlinearity in the failure envelope with respect to suction compacted Dhanauri clay at high-density. (a) The shear strength, τ , plotted against $(u_a - u_w)$; (b) nonlinear relationship between ϕ^b and matric suction, $(u_a - u_w)$ (after Satija, 1978).

2.6.2 Constant Water Content Triaxial Tests

The conditions for a constant water content triaxial test are similar to the conditions for an undrained triaxial test for unsaturated soils except that the air phase is drained. The specimen is first consolidated under an isotropic confining stress of σ_3 while the pore-air and pore-water pressures are controlled at the desired values of u_a and u_w , respectively. During the shearing process, the air valve is opened while the water valve is closed. Figure 2.8 shows that the net confining stress, $(\sigma_3 - u_a)$, remains constant but the matric suction, $(u_a - u_w)$, changes during shearing.

Figure 2.9 shows the stress path during a constant water content triaxial test. Point A represents the stress state at the end of consolidation. When the soil is compressed during shearing, the stress point is assumed to move from point A to point B along stress path AB. Stress point B represents the stress state at failure. Figure 2.9 shows that the net confining pressure remains constant at $(\sigma_3 - u_a)$, the same as the net confining pressure at the end of consolidation. The pore-water pressure will increase or matric suction will decrease during shearing, $[(u_a - u_w)_f < (u_a - u_w)_i]$.

Stages	Total stress	Pore-air pressure	Pore-water pressure	$(\sigma - u_a)$	$(u_a - u_w)$
Equilibrium at the end of consolidation	σ_3 	u_a 	u_w 	$(\sigma_3 - u_a)$ 	$(u_a - u_w)$
Axial compression	$(\sigma_1 - \sigma_3)$ σ_3 	Drained and controlled u_a 	Undrained and measured $u_w + \Delta u_w$ 	$(\sigma_1 - u_a)$ $(\sigma_3 - u_a)$ 	$(u_a - u_w) - \Delta u_w$ $(u_a - u_w) - \Delta u_w$
At failure	$(\sigma_1 - \sigma_3)_f$ σ_3 	u_a 	$u_{wf} = u_w + \Delta u_{wf}$ 	$(\sigma_1 - u_a)_f = \sigma_{1f} - u_a$ $(\sigma_3 - u_a)_f = \sigma_3 - u_a$ 	$(u_a - u_w)_f$ $(u_a - u_w)_f = (u_a - u_w) - \Delta u_{wf}$

Figure 2.8 Stress condition during a constant water content triaxial compression test (after Fredlund and Rahardjo, 1993)

Figure 2.9 Stress path of the CW triaxial tests performed at various matric suctions under a net confining pressure (after Fredlund and Rahardjo, 1993)

The failure envelope is tangent to the Mohr circles at failure with an angle of ϕ' with respect to the net normal stress. The failure envelope intersects the shear strength versus matric suction plane at a cohesion intercept, c . The ϕ^b angle can be obtained by joining the cohesion intercepts at various matric suctions.

Typical stress versus strain curves for a constant water content test was presented by Satija (1978) as shown in Figure 2.10a. Figure 2.10b presents the changes in matric suction during shearing at the same initial matric suction but different net confining pressures. The results show that the higher the net confining stress was, the larger the change in matric suction that would occur during shearing. Figure 2.10c shows that soil specimens undergo compression until the maximum deviator stress is reached. Meanwhile the specimen with the lower net confining pressure dilates after reaching the maximum deviator stress.

Figure 2.10 Constant water content triaxial tests on Dahaunari clay (a) Stress versus strain curve; (b) matric suction change versus strain; (c) soil volume change versus strain (after Satija, 1978)

Rahardjo et al. (2004) conducted a series of consolidated drained and constant water content tests on compacted residual soils from the Jurong Sedimentary and Bukit Timah Granite Formations of Singapore. The results of the study indicate that at high matric suctions relative to the net confining stress, the soils exhibit characteristics of an overconsolidated soil with post-softening and dilatancy behaviour. The shear strengths obtained from the consolidated drained and constant water content tests are the same when the initial matric suction of the soil is less than the air-entry value of the soil.

The use of the CW triaxial test is beneficial because it is less time consuming as compared with the CD triaxial test on unsaturated soil specimens. Another advantage of the CW triaxial test is that the stress path of the CW triaxial shearing test can be used to simulate closely many stress paths in field situations for unsaturated soils.

In the past few decades many researchers (Bishop et al., 1960; Bishop and Donald, 1961; Bight, 1961; Satija, 1978; Sivakumar, 1993; Rahardjo et al., 2004) have studied the shear strength characteristics of unsaturated soils under the constant water content condition in a triaxial apparatus. The difficulty associated with the CW test is the assurance for uniformity of the pore-water pressure

during shearing. Bishop et al. (1960) studied the non-uniformities of pore-water pressure in a specimen during shearing by using a mini pore-water pressure probe. The mini pore-water pressure probe was inserted into a hole drilled inside the specimen. The maximum difference between pore-water pressure measurements at the base plate and at mid-height of the specimen was about 30 kPa as reported by Bishop et al. (1960). The excess pore-water pressure along the soil specimen during shearing under the constant water content condition has not been studied in detail.

2.6.3 Consolidated Drained (CD) Triaxial Tests

A CD triaxial test can be performed to obtain the shear strength parameters for unsaturated soil. The CD triaxial test refers to a test condition where a specimen is sheared under drained conditions for both the pore-air and pore-water phases. Figure 2.11 shows the CD triaxial test conditions for unsaturated soil. The specimen first consolidated under an isotropic stress of σ_3 , with pore-water, u_w , and pore-air, u_a , pressures being controlled at a desired value. At the end of the consolidation stage, the soil specimen has a net confining stress of $(\sigma_3 - u_a)$ and a matric suction of $(u_a - u_w)$.

Figure 2.11 Stress conditions during a consolidated drained triaxial compression test (after Fredlund and Rahardjo, 1993)

Figure 2.12 shows the stress paths followed during the CD triaxial tests conducted at different net confining stresses under a given matric suction. Stress point A represents the stress state of a soil specimen at the end of consolidation and matric suction equalization with a particular net confining stress. During shearing, an axial load is applied to the soil specimen at a slow rate to ensure dissipation of pore pressures. The net confining stress, $(\sigma_3 - u_a)$, and matric suction, $(u_a - u_w)$, are kept constant while the deviator stress increases throughout shearing stage until failure condition is reached. Meanwhile, stress point B represents the stress state at failure condition. The stress path AB represents the stress path followed by the soil specimen from the starting spoint of shearing until failure is reached. The net confining stress and matric suction remain constant throughout the stress path AB. As the net confining stress increases, the Mohr circle diameter at failure increases. Tangential point to the Mohr circle, C, C₁, and C₂ represent the failure conditions at various net confining stresses under a constant matric suction. The slope of tangential line to the Mohr circle is internal friction angle, ϕ' . The failure envelope intersects the shear strength versus matric plane at an effective cohesion intercept, c' .

Figure 2.12 Stress paths followed during a consolidated drained test at various net confining pressures under a constant matric suction (after Fredlund and Rahardjo, 1993)

Figure 2.13 shows the stress paths followed during the CD triaxial tests carried out at various matric suctions under a constant net confining stress. Similarly, the stress points C_1 , C_2 and C_3 are tangent to the failure envelope. However, those stress points do not lie at the same net confining stress. The line joining these cohesion intercepts will give the ϕ^b angle.

The advantage of the CD triaxial tests is that the matric suction can be controlled during the triaxial shearing test. The CD triaxial test can be performed under any matric suction value. However, the CD triaxial test is time consuming. The other difficulty associated with the use of CD triaxial test is that the stress path followed during the CD triaxial test may not simulate the stress path that occurred in the many field situations.

Figure 2.13 Stress paths followed during consolidated drained tests at various matric suctions under a constant net confining stress (after Fredlund and Rahardjo, 1993)

2.6.4 The Measurements of Matric Suction

Bishop et al. (1960) studied the non-uniformities in pore-water pressure in a specimen during shearing by using the mini pore-water pressure probe. The mini pore-water pressure probe was inserted into a hole drilled inside the specimen. The test result is shown in Figure 2.14 illustrating the difference in pore-water pressure measurements at the base plate and mid-height of the specimen. The maximum difference between pore-water pressure measurements at the base plate and mid-height of the specimen was about 30 kPa. The important conclusion drawn from this study is that the measurement of pore-water pressure at the base plate gives too high a value.

Figure 2.14 Pore-water pressure measurement at the base plate and mid-height of the sample on compacted shale with strain rate of 20% in 8 hours (after Bishop et al., 1960)

Blight (1961) used four miniature pore pressure probes between the base and midheight of a single triaxial specimen of 100mm in diameter by 200mm in height. Barden and McDermott, (1965) conducted a series of triaxial tests on a saturated clay. The pore-water pressures at the base plate and the mid-height of the specimen were measured during shearing. Figure 2.15 shows the main features of the triaxial

apparatus. The study pointed out that the pore-water pressure measured by a probe at the mid-height of the specimen may be more relevant than the conventional one measured at the base plate. However, the measurements of pore-water pressure by a probe at the mid-height of the specimen cannot be assumed to be representative of the entire specimen.

Blight (1965) re-examined the relationship between the pore-water pressure parameter A and the soil stress history from undrained triaxial tests on saturated clays. The values of the pore-water pressure parameter A corresponding to the principal stresses nominally imposed on the soil specimens cannot be measured correctly unless the end restraint is eliminated. Two methods of achieving this were proposed: (1) to compress the soil between parallel lubricated surfaces (i.e., Figure 2.16 a) and (2) to isolate the central part of the test specimen from the zones in contact with the loading platens and then to measure only the pore-water pressure established in the central zone (Figure 2.16b). The results show that the conventional triaxial test overestimates the pore-water pressure parameter A for normally consolidated clays and conversely underestimates the A value for overconsolidated clays.

Figure 2.15 Set-up of triaxial apparatus with mid-height mini pore pressure probe (after Barden and McDermott, 1965)

Hight (1982) used miniature silicon diaphragm pressure transducer to measure pore-water pressure at the mid-height of Lower Cromer Till saturated specimens. The probe was installed in specimens of 38 mm in diameter. Table 2.1 presents the characteristics of the miniature silicon diaphragm pressure transducer. The pressure transducer was mounted at the mid-height of the specimen with its porous ceramic face being flushed with the cylindrical surface of the specimen. The combination allows minimum interference between the transducer and the specimen.

Figure 2.16 Schematic test arrangement (after Blight, 1965)

Table 2.1 Characteristics of the miniature silicon diaphragm pressure transducer (after Hight, 1982)

Overall length	11.4 mm
Overall diameter	6.3 mm
Weight (excluding cable)	~ 2g
Transduction principal	4 arm silicon strain gauge
Non-linearity	± 0.15 % of full scale output
Pressure range	0-600 kPa
Resolution	0.1 kPa

Toll (1988) measured the pore-water pressure at mid-height of unsaturated soil specimen by using a pore-water pressure probe. The pore-water pressure probe was installed into a hole of 12.5 mm in diameter, drilled in the soil specimen. The probe used a 500 kPa high air-entry hollow ceramic cup produced by Soilmoisture Equipment Corp. The high air-entry hollow ceramic cup was mounted in a brass housing epoxy resin. However, the response of the pore-water pressure probe used by Toll (1988) was slow. Figure 2.17 shows the response time of the pore-water pressure measurement using the mini probe and at the base plate when the cell pressure was increased suddenly. The response time of the pore-water pressure measurement at the base plate is even faster than the pore-water pressure probe at the mid-height of the specimen.

Ridley and Burland (1993) used a stainless steel miniature pore-water pressure transducer, Entran EPX-500, for direct measurement of negative pore-water pressure. The transducer can measure pore-water pressure down to -3500 kPa. The miniature pore-water pressure transducer consisted of a stainless steel sheath and a 15 bar ceramic porous stone mounted on top of the sheath (Figure 2.18). The sheath was designed with a small gap between the high air-entry ceramic disk and the pressure probe, to give a water reservoir of 3 mm³. Negative pore-water pressure was measured using the probe that was mounted in a hole at the centre of a horizontal plate. The transducer was able to measure the negative pore-water pressure directly down to -1200 kPa with a response time of less than 5 minutes.

Figure 2.17 The response of pore-water pressure probe and pore-water pressure at base plate due to increase cell pressure (after Toll, 1988)

Muraleetharan and Granger (1999) evaluated the use of miniature pore-water pressure transducer (PDCR 81) to measure matric suction in unsaturated soils. The transducer was manufactured by Druck limited. The schematic diagram of the transducer is shown in Figure 2.19. A high air-entry ceramic disk was mounted at the tip of the transducer. The air-entry value of the disk was measured by using the drying method. At first, the transducer was saturated in a 90 kPa vacuum chamber three – quarter filled with water. Then, the transducer was exposed to atmospheric pressure for drying. The result showed that the air-entry value of the disk was about 132 kPa. Any air bubbles trapped in the gap between the porous stone and the diaphragm can be removed only by dissolving air into water with the application of high pressure.

Figure 2.18 Matric suction measurement probe (from Ridley and Burland, 1993)

Figure 2.19 Schematic diagram of the miniature pore pressure probe PDCR 81 (after Kutter et al., 1990)

Wong (2000) used mini pore-water pressure probes PDCR 81 to measure the pore-water pressures along the specimen. Three mini pore-water pressure probes PDCR 81 were installed at the top, middle and bottom (i.e., the 1/2, 1/4 and 3/4 heights) of a soil specimen in the triaxial testing. The working range of the transducer is 1500 kPa. However, the transducers were only able to measure matric suction up to 31 hours and matric suction about 60 kPa as shown in Figure 2.20.

The mid-height pore-water pressure measurements in unsaturated triaxial testing have limitations in terms of response time (Toll, 1988) and cavitations in the measurement at high suction range (Wong, 2000).

Meilani et al. (2002) designed and fabricated a new ceramic disk for mini pore-water pressure probe PDCR 81 (NTU mini suction probe). The original ceramic disk of the mini pore-water pressure probe PDCR 81 was removed and replaced by a new 5 - bar high air-entry ceramic disk. The ceramic disk was designed with a T-shape as shown in Figure 2.21. The ceramic disk was glued to the steel tube from outside of the steel tube.

Figure 2.20 Pore-water pressure measurements from miniature pressure transducers (PDCR 81) during the equalisation stage under 100 kPa net confining pressure and 75 kPa matric suction (after Wong, 2000).

Figure 2.21 High air-entry ceramic disk for NTU mini suction probe (from Meilani et al., 2002)

Three NTU mini suction probes have been tested by a drying method. Figure 2.22 shows the response of the NTU mini suction probes to the application of negative pore-water pressure during a drying process. The 'top', 'middle' and 'bottom' NTU mini suction probe measurements dropped quickly to -426 kPa, -400 kPa and -495 kPa, respectively. The newly designed high air-entry ceramic disk for PDCR 81 was quite successful and the NTU mini suction probe can measure negative pore-water pressure down to -400 kPa. The NTU mini suction probe PDCR 81 modified by Meilani et al. (2002) can then be used to measure matric suction in a triaxial specimen.

Figure 2.22 Response of the mini suction probes during a drying process (from Meilani et al., 2002)

2.6.5 Volume Change Measurements

The measurement of specimen volume change during an unsaturated soil triaxial shearing has been a difficult problem. Three methods have been used to measure volume change of an unsaturated soil specimen during shearing: (1) measurement of cell fluid, (2) measurement of water and air volumes separately, and (3) direct measurement of the soil specimen, (Geiser et al., 2000).

Bishop and Donald (1961) developed a modified triaxial cell to minimize the effects of expansion-compression of the cell and volume change in the cell fluid. The

modified triaxial cell included an additional inner cylindrical cell sealed to the outer cell base. Mercury was used as the cell fluid in the lower part of the inner cell. The pressure was kept constant in the inner cell and the outer cell. A stainless steel ball was floated on the mercury used in the inner cell to monitor the movement of the inner cell fluid during testing. The movement of the stainless steel ball was converted to the total volume change of the specimen.

Sivakumar (1993) used a double-wall triaxial cell for measuring the volume change in unsaturated soils. The specific volume was calibrated at an accuracy of ± 0.011 . However, there was some difficulty in accurately measuring the deviator load with the particular design of double-walled cell, because the load cell was positioned external to the triaxial cell.

Cui and Delage (1996) used a double wall triaxial cell similar to Bishop and Donald (1961). Lightly coloured water was used in the inner cell instead of mercury. A thin layer of silicon oil was added on top of the water to avoid the dissolution of air into water and to reduce evaporation. Volume changes were optically monitored by the changes in the level of the oil-water interface with a cathetometer.

Geiser et al. (2000) used methods of direct measurement of air and water volume changes. However, it is generally accepted that accurate measurement of air volume change is difficult because volume of air is very sensitive to variations in ambient temperature, and undetectable air leakage through the tubes and air diffusion through rubber membrane.

Ng and Chiu (2001) measured the local vertical and radial deformations of the soil specimens. However, the method is best suited to small deformation of the specimens and requires a fairly rigid specimen.

Ng et al. (2002) developed a new system for measuring total volume change of saturated and unsaturated soil specimens in triaxial tests. The aluminium inner cell with a bottle-shape was sealed onto a pedestal. The specimen volume change was measured by recording the change in the differential pressure due to changes in the water level inside an open-ended, bottle-shaped inner cell. The error of the total

volume change of unsaturated soil was 0.04%. However, this design may not be suitable for specimens that incorporate with mini suction probes.

Non-contacting methods have been used to measure the soil volume change in triaxial tests. Romero et al. (1997) proposed the use of a laser scanner and Macari et al. (1997) reported the use of a miniature camera to assist the measurement of volume change of unsaturated soils in triaxial tests. However, these methods require sophisticated installation procedures and may not be suitable for small deformations in unsaturated specimen (Ng et al., 2002).

Agus et al. (2003) used a flexible wall permeameter for measurements of the water and air coefficients of permeability of residual soils. The total volume change of the soil specimen was measured by monitoring the volume of water entering or leaving the permeameter cell using digital water pressure-volume controller. The observed reading of volume change fluctuated with the ambient room temperature. The coefficient of the volume change expansion, α , of the confined system was used to calculate the volume change of the confined system. The coefficient of the volume change expansion of the confined system was obtained by a dummy test using a stainless steel specimen in the system.

2.7 Review the Elasto-plastic Model for Saturated Soils

2.7.1 Basic Concept of Critical State Model for Saturated Soil

Critical state soil mechanics can be used for estimating the shear strength, excess pore-water pressure and volume change in response to changes in loading conditions. The critical state model developed for saturated soils is defined in terms of three state variables, mean effective stress, p' , deviator stress, q , and specific volume, v , (Schofield and Wroth, 1968). For the simplified conditions of the triaxial test, ($\sigma_2 = \sigma_3$), these state variables are defined as follows:

$$q = \sigma_1 - \sigma_3 \quad (2.24)$$

$$p' = \left(\frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \right) - u_w \quad (2.25)$$

$$v = 1 + e \quad (2.26)$$

where:

q = deviator stress,

p' = mean effective stress of a saturated soil,

u_w = pore-water pressure,

v = specific volume, and

e = initial void ratio.

If a saturated soil is sheared, it will ultimately reach a critical state where shear strain can continue without any further change in deviator stress, q , or specific volume, v . At a critical state the soil behaves like a frictional fluid (Schofield and Wroth, 1968).

Casagrande (1936) identified a critical state density at which soil continues to deform at a constant shear strength. Rendulic (1936) identified that for a given soil at given conditions of initial stress and stress history, there was a unique relationship between void ratio and changes in stress. Rendulic supported this concept by conducting undrained and drained tests on saturated soils and found that constant void ratio contours plotted from the drained test data on axes of q and p' coincided with the contours plotted from the undrained test data.

2.7.1.1 Yield Surface

Figure 2.23 shows the typical yield surfaces in stress space that separates stress states which produce elastic response from those which produce plastic response. The yield surface is assumed to be an ellipse and its initial size on the major axis is determined by the preconsolidation stress, p_c^* . A higher preconsolidation stress corresponds to a larger initial ellipse. All combinations of q and p' that lie within the yield surface (e.g., point A in Figure 2.23), will cause the soil to respond elastically. If the stress state of the soil lies on the initial surface (e.g., point B in

Figure 2.23) the soil is yielding. Any tendency of a stress combination to move outside the current initial yield surface causes the yield surface to expand to a larger yield surface corresponding to a new preconsolidation stress (p_{c1}^* in Figure 2.23).

The equation for an ellipsoidal yield surface as proposed by Schofield and Wroth (1968) and Roscoe and Burland (1968) is as follows:

$$\left(p'\right)^2 - p' p_c^* + \frac{q^2}{M^2} = 0 \quad (2.27)$$

where:

p_c^* = preconsolidation stress of saturated soil, and

M = slope of the critical state line.

Figure 2.23 Expansion of the yield surface (after Budhu, 2000)

The critical state line (CSL) intersects the yield surface at $p_c^*/2$. The failure line in $(q - p')$ space can be defined by the following equation (Budhu, 2000):

$$q_f = M p_f' \quad (2.28)$$

where:

q_f = deviator stress at failure, and

p'_f = mean effective stress at failure.

2.7.1.2 Critical State Parameters

The specific volume, v , of soil in an isotropic normally consolidated state is linearly related to the logarithm of mean effective stress (Scholfield and Wroth, 1968; Wood, 1990; Budhu, 2000) as shown in Figure 2.24 (c).

Figure 2.24 Critical state lines and these parameters. (a) Yield surface; (b) CSL on $(v - p')$ space; (c) CSL on $(v - \ln p')$ space (after Budhu, 2000)

The deviator stress, q , will be zero when the soil is under an isotropic stress condition. Therefore, the isotropic normal compression line (i.e., NCL) is defined by 2 equations:

$$q = 0 \quad (2.29)$$

$$v_c = N(0) - \lambda(0) \ln p_c^{*'} \quad (2.30)$$

where:

- $N(0)$ = the specific volume of normal compression line at reference stress,
- $\lambda(0)$ = the stiffness of the saturated soil, which is obtained from an isotropic or a one dimensional consolidation test,
- κ = unloading/reloading index or stiffness, which is obtained from an isotropic or one dimensional consolidation test, and
- v_c = the specific volume on the normal consolidated line at the preconsolidation stress, p_c^{*} .

The reference stress is used to make the equation dimensionally consistent and to minimize the error in the evaluation of intercept, $N(0)$. The value of $N(0)$ depends on the units chosen for the p' scale. In soils mechanics the unit of kPa is usually used for the stress and matric suction.

When a soil is sheared it will ultimately reach a critical state. These critical states were found to lie on a unique line in $(q - v - p')$ space: the critical state line (i.e., CSL in Figure 2.24). The critical state line is defined by the following two equations:

$$q = Mp' \quad (2.31)$$

$$v_f = \Gamma(0) - \lambda(0) \ln p_f' \quad (2.32)$$

where:

- $\Gamma(0)$ = specific volume on the critical state line for saturated soil at reference stress,
- v_f = specific volume at failure, and
- p_f' = mean effective stress at failure.

Assuming that a soil is isotropically consolidated to a mean effective stress, p'_c , and then isotropically unloaded to a mean effective stress, p'_o , (Figure 2.24). Let point F be the intersection of the unloading/reloading line (i.e., line AF) with the critical state line (Figure 2.24)). The mean effective stress at F is $\frac{p_c^{*'}}{2}$ and then the equation for the unloading/reloading line can be obtained as Budhu (2000):

$$v_f = v_o + \kappa \ln \left(\frac{p'_o}{p_c^{*'} / 2} \right) \quad (2.33)$$

where

v_o = the initial specific volume.

The equation for the critical state line, CSL in Figure 2.24(c) is as follows:

$$v_f = \Gamma(0) - \lambda(0) \ln \frac{p_c^{*'}}{2} \quad (2.34)$$

The specific volume of the critical state line at reference stress can be calculated by equating Equations (2.33) and (2.34) as follows:

$$\Gamma(0) = v_o + \left[\lambda(0) - \kappa \right] \ln \frac{p_c^{*'}}{2} + \kappa \ln p'_o \quad (2.35)$$

The specific volume of the critical state line at reference stress (i.e., Equation (2.35)) will later be used to calculate excess pore-water pressure at failure.

2.7.2 Prediction of the Excess Pore-water Pressure in Normally Consolidated and Lightly Overconsolidated Saturated Soils under an Undrained Condition

Figure 2.25 shows the stress paths of triaxial compression on lightly overconsolidated. If a soil is sheared under undrained loading conditions from an initial stress state (point K, Figure 2.25), the stress path will be vertical until the soil

reaches the initial yield surface at point Y (Figure 2.25). From point Y the soil behaves as an elasto-plastic material and the stress path will continue to move toward the critical state line (CSL) until the critical state line is reached at point F (Figure 2.25).

In the undrained test for saturated soil volume remains constant, that is $\Delta v = 0$, and effective stress path (ESP) inside the yield curve is vertical (i.e., KY), that is the change in mean effective stress, $\Delta p'$, is zero for linearly elastic soils (i. e., initial mean effective stress, p'_o , equal to mean effective stress at yield, p'_y , (i.e., $p'_o = p'_y$), as shown in Figure 2.25). Since the ESP is vertical within the initial yield surface, then the yield stress can be found from the intersection point of ESP and yield surface.

Figure 2.25 Undrained stress path of triaxial compression test on lightly overconsolidated soil

The equation to determine q_y for an isotropically consolidated soil can be obtained by rearranging Equation (2.27) as follows:

$$q_y = Mp_o' \sqrt{\frac{p_c^{*'} }{p_o'} - 1} \quad (2.36)$$

$$\frac{p_c^*}{p_o} = R \quad (2.37)$$

where:

R = overconsolidation ratio.

Substituting Equation (2.37) into Equation (2.36) gives,

$$q_y = Mp_o' \sqrt{R-1} \quad (2.38)$$

- ***Excess Pore-water Pressure at Yield Point***

In a triaxial shearing test, the total stress path can be calculated in $(q - p)$ plane as follows:

Since $\Delta\sigma_2 = \Delta\sigma_3 = 0$, $\Delta\sigma_1 > 0$

Then $\Delta q = \Delta\sigma_1 - \Delta\sigma_3 = \Delta\sigma_1$ (2.39)

$$\Delta p = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3} = \frac{\Delta\sigma_1}{3} \quad (2.40)$$

The slope of the total stress path (TSP) can be obtained by dividing Equation (2.39) with Equation (2.40):

$$\frac{\Delta q}{\Delta p} = 3 \quad (2.41)$$

The difference in mean stress between the total stress path and the effective stress path gives the change in pore-water pressure. The change in pore-water pressure at yield point, Δu_y , (Figure 2.25) can be calculated as follows:

$$\Delta u_y = p_y - p_y' = p_y - p_o' \quad (2.42)$$

where:

Δu_y = change in pore-water pressure at the yield point,

p_y = total stress at the yield point, and

p'_y = effective stress at the yield point.

From Figure 2.25, the mean total stress at the yield point can be calculated as follows:

$$p_y = p'_o + \frac{1}{3}q_y \quad (2.43)$$

Substituting Equations (2.43) and (2.38) into Equation (2.42), the change in pore-water pressure at the yield point can be obtained:

$$\Delta u_y = \frac{1}{3}Mp'_o\sqrt{R-1} \quad (2.44)$$

- ***Excess Pore-water Pressure at Failure:***

The stress path moves toward the critical state line upon further shearing after reaching the initial yield point (i.e., point Y in Figure 2.25). The stress at failure is given the intersection of the ESP after the yield point and the critical state line (i.e., point F in Figure 2.25). The excess pore-water pressure at failure can be determined from the difference between the mean effective stress and the mean total stress at failure. Therefore, the excess pore-water pressure at failure (i.e., Δu_f) can be calculated as follows (Figure 2.25):

$$\Delta u_f = p'_o - p'_f + \frac{1}{3}q_f \quad (2.45)$$

Substituting Equation (2.28) into Equation (2.45) gives:

$$\Delta u_f = p'_o - p'_f + \frac{1}{3}Mp'_f \quad (2.46)$$

Equation (2.46) contains only one unknown parameter, p'_f . Therefore, the mean effective stress, p'_f , should be calculated.

No volume change occurs in an undrained test, meaning that $\Delta v = 0$ and therefore, Equation (2.32) can be rewritten as:

$$v_f = v_o = \Gamma(0) - \lambda(0) \ln p'_f \quad (2.47)$$

The mean effective stress at failure can be calculated by rearranging Equation (2.47)

$$p'_f = \exp \left(\frac{\Gamma(0) - v_o}{\lambda(0)} \right) \quad (2.48)$$

The change in pore-water pressure at failure can be calculated by substituting Equations (2.35) and (2.48) into Equation (2.46) as follows:

$$\Delta u_f = p'_o + \left(\frac{M}{3} - 1 \right) \exp \frac{[\lambda(0) - \kappa] \ln \frac{p_c^*}{2} + \kappa \ln p'_o}{\lambda(0)} \quad (2.49)$$

2.7.3 Prediction of the Excess Pore-water Pressure of Heavily Overconsolidated Soils

- ***Excess Pore-water Pressure at the Initial Yield Point***

Figure 2.26 shows the idealized stress paths associated with an undrained triaxial test on heavily overconsolidated soil. Heavily overconsolidated soils have ESP that lies to the left of the intersection point (i.e., point F) between the critical state line and the yield surface. Critical state soil mechanics models the mechanical behaviour of heavily consolidated soils as an elastic material up to the peak shear stress and thereafter as an elasto-plastic material when the imposed loading causes the soil to strain soften toward the critical state line.

In the undrained test for a saturated soil, the soil volume remains constant, that is $\Delta v = 0$, and the effective stress path (ESP) inside the yield curve is vertical (i.e., KY in Figure 2.26), that is the change in the mean effective stress, $\Delta p'$, is zero for a linearly elastic soil (i. e., $p'_o = p'_y$ in Figure 2.26).

For a saturated soil, the pore-water pressure is equal to the change in the total mean stress. In other words, the difference between the total mean stress and the effective mean stress gives the change in pore-water pressure. Therefore, the excess pore-water pressure at the yield point (i.e., Δu_y) can be calculated as follows:

$$\Delta u_y = p_y - p'_y = p_y - p'_o \quad (2.50)$$

Figure 2.26 Idealized undrained stress path triaxial compression test on heavily overconsolidated soil

From the TSP the total stress at the yield point can be calculated as follows:

$$p_y = p'_o + \frac{1}{3} q_y \quad (2.51)$$

The excess pore-water pressure at the yield point can be obtained by substituting Equation (2.51) into Equation (2.50) and replacing p_y with p'_o as follows:

$$\Delta u_y = p'_o + \frac{q_p}{3} - p'_o \text{ or } \Delta u_y = \frac{q_p}{3} \quad (2.52)$$

- ***Excess Pore-water Pressure at Failure:***

Figure 2.26 shows that after the soil has reached the yield point (i.e., point Y) the mean effective stress increases. As a result, the excess pore-water pressure will decrease. The excess pore-water pressure at failure can be negative because p'_f is greater than p_f . The excess pore-water pressure at failure can be computed as follows

$$\Delta u_f = p'_f - p'_o - \frac{1}{3} q_f \quad (2.53)$$

The excess pore-water pressure at failure can then be formulated by substituting Equations (2.28), (2.35) and (2.48) into Equation (2.53) as follows:

$$\Delta u_f = \left(1 - \frac{M}{3}\right) \exp \left\{ \frac{[\lambda(0) - \kappa] \ln \frac{p_c^*}{2} + \kappa \ln p'_o}{\lambda(0)} \right\} - p'_o \quad (2.54)$$

2.8 Review the Elasto Plastic Model for Unsaturated Soils

In the past few decades numerous researchers have studied the role of matric suction or degree of saturation in engineering properties of unsaturated soils. The influence of matric suction on the shear strength, volume change and excess pore-water pressure have been investigated (Bishop and Donald, 1961; Fredlund and Morgenstern, 1977; Fredlund et al., 1978; Alonso et al., 1987; Fredlund and Rahardjo, 1993; Wheeler and Karube, 1995; Graham et al., 1995; Delage and Graham, 1995; Wheeler, 1996; Cui and Delage, 1996; Wheeler and Sivakumar, 1995; Rampino et al., 1999; Tang and Graham, 2002 and Chiu and Ng, 2003).

The general framework for the constitutive model for unsaturated soil proposed by Alonso et al. (1987, 1990) has been refined by other researchers (Toll, 1990; Thomas and He, 1994; Wheeler and Sivakumar, 1995; Cui and Delage, 1996; Wheeler, 1996; Bolzon et al., 1996; Rampino et al., 1999; Simoni and Schrefler, 2001; Tang and Graham, 2002; Chiu and Ng, 2003). These models were developed

under the framework of independent stress state variables and using the extended concept for unsaturated soils. However, different assumptions and constitutive relationships have been used in these models.

Alonso et al. (1990) formulated an elasto-plastic model with four state variables: mean net stress p , deviator stress q , matric suction s and specific volume v . The plastic strains start when the state boundary hypersurface is reached. The model includes the yield loci, flow rules, hardening laws, and elastic and plastic strains. Wheeler and Sivakumar (1995) proposed an elasto-plastic model that is similar to the Alonso et al. (1990) model. Wheeler et al. (2003) and Gallipoli et al. (2003) incorporated suction into the elasto-plastic model for unsaturated soils. Wheeler et al. (2003) and Gallipoli et al. (2003) incorporated matric suction as a single stress state variable in formulating the elasto-plastic model for unsaturated soils. However, Fredlund and Morgenstern (1977) suggested that matric suction should be treated as independently. Fredlund and Morgenstern (1977) proposed that the constitutive behaviour of unsaturated soils be described using two independent stress state variables; namely, net normal stress, $(\sigma - u_a)$, and matric suction, $(u_a - u_w)$.

Toll (1990) conducted a series of triaxial tests on compacted Kiunyu gravel. Toll (1990) tested on statically compacted specimens at different water contents. The degree of saturation was considered as a factor affecting the slopes of the critical state line on the $(q - p)$ and $(v - p)$ planes (i.e., M and λ). Most of the tests were conducted by shearing at constant water content condition. Toll (1990) proposed equations for the deviator stress, q and specific volume, v , at the critical state.

$$q = M_a (p - u_a) + M_w (u_a - u_w) \quad (2.55)$$

$$v = \Gamma_{aw} - \lambda_a \ln(p - u_a) - \lambda_w (u_a - u_w) \quad (2.56)$$

where:

M_a = slope of the critical state with respect to mean net stress on $(p - q)$

	plane,
$(p - u_a)$	= mean net stress,
M_w	= slope of the critical state with respect to matric suction on $(q - (u_a - u_w))$ plane,
v	= specific volume,
Γ_{aw}	= the specific volume when both $(p - u_a)$ and $(u_a - u_w)$ equal unity,
λ_a	= slope of the critical state with respect to mean net stress on $(v - p)$ plane, and
λ_w	= slope of the critical state with respect to matric suction on $(v - (u_a - u_w))$ plane.

The parameters M_a , M_w , λ_a , λ_w and Γ_{aw} were factors which changed with degree of saturation, S . The parameters M_a and M_w can be calculated as follows:

$$M_a = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (2.57)$$

$$M_w = \frac{6 \cos \phi' \tan \phi'}{3 - \sin \phi'} \quad (2.58)$$

Toll (1990) back – calculated values of M_a , M_w , λ_a and λ_w by conducting a regression analysis and assuming that each parameter was a function of S only. The value of M_a increased from the saturated value M as the degree of saturation decreased. This behaviour can be perhaps be explained by the fact that ϕ' is not a constant for gravel soil and it changes with dry density. In addition, the strength parameters of a soil are not the same under wet and dry consitions (Skinner, 1975). The parameter M_w decreased from the saturated value M as the degree of saturation decreased.

Fredlund, Rahardjo and Gan (1987) suggested that unique critical state relationships for q and v (in terms of $(p - u_a)$ and $(u_a - u_w)$) might be obtained if the initial structure was destroyed during shearing to produce similar structures at critical

state. Fredlund, Rahardjo and Gan (1987) argued that if different initial structures were created in a number of different compacted soil specimens then those specimens should be considered as different soils. Fredlund et al. (1987) suggested that specimens with similar initial structures must be used when establishing any constitutive relationship, to avoid any influence of initial structure in the model.

Wheeler (1991) re-analysed some of the results obtained by Toll (1990). Wheeler described the difficulties involved in applying Equations (2.55) and (2.56). As Toll's (1990) equations depend on degree of saturation, it would be very difficult to predict the critical state value of q and v in a field problem or a laboratory test because there would be no way of predicting the variation of degree of saturation in advance.

Wheeler (1991) defined a critical state volumetric relationship in terms of mean net stress and matric suction.

$$v_w = \Gamma(s) - \lambda(s) \ln(p - u_a) + f(u_a - u_w) \quad (2.59)$$

where:

v_w = specific water volume,

$\Gamma(s)$ = specific volume at reference stress, and

$\lambda(s)$ = slope of the normal compression curve.

Rampino et al. (2000) proposed critical state for the degree of saturation, S , instead of specific water volume as proposed by Wheeler (1991).

$$S = \Gamma_s(s) - \psi_s(s) \ln \left[\frac{(p - u_a)}{(p - u_a)_{ref}} \right] \quad (2.60)$$

where:

$\Gamma_s(s)$ = degree of saturation at the reference stress of the critical state line,

$$\begin{aligned}\psi_s(s) &= \text{slope of the critical state for the degree of saturation, and} \\ (p - u_a)_{ref} &= \text{mean net stress at reference stress.}\end{aligned}$$

These models were developed under the framework of independent stress state variables using the extended concept of saturated soil mechanics for unsaturated soils. It has been postulated that matric suction has a significant influence on soil behavior in terms of volume changes, stress-strain and shear strength (Sivakumar, 1993; Fredlund et al., 1996; Vanapalli et al., 1996; Bolzon et al., 1996). The relationship between volumetric water content and matric suction is known as soil-water characteristic curve. Vanapalli et al. (1996) stated that the shear strength of unsaturated soil should bear a relationship to the soil-water characteristic curve. Vanapalli et al. (1996) used the soil-water characteristic curve to predict the shear strength of unsaturated soils. However, the existing elasto-plastic models for unsaturated soil do not incorporate soil-water characteristic curve. Therefore, it would be appropriate to incorporate SWCC in the elasto-plastic model for unsaturated soils.

CHAPTER 3 THEORY

3.1 Introduction

This chapter presents the theoretical development of shear strength, excess pore-water pressure and volume change during triaxial shearing. The theory of shear strength, pore-water pressure and volume change development are based on elasto-plastic theory. The approach used in deriving the theory for the shear strength, excess pore pressure and volume change development is based on the Modified Cam Clay model and critical state concept for unsaturated soils. The existing elasto-plastic models for unsaturated soil do not incorporate soil-water characteristic curve (SWCC) in obtaining soil parameters required for the elasto-plastic model. An elasto-plastic model that incorporates SWCC for unsaturated soil is proposed in this chapter. The SWCC is shown to govern the rate of change in the soil parameters for the elasto-plastic model with respect to matric suction.

3.2 Theoretical background for Elasto-plastic Theory for Unsaturated soil

The general framework for the constitutive model for unsaturated soil has been proposed by Alonso et al. (1987, 1990). The constitutive model proposed by Alonso has been refined by other researchers (Toll 1990; Thomas and He, 1998; Wheeler and Sivakumar, 1995; Cui et al., 1995; Wheeler, 1996; Bolzon et al., 1996; Rampino et al., 1999; Simoni and Schrefler, 2001; Tang and Graham, 2002 and Chiu and Ng, 2003). These models were developed under the framework of

independent stress state variables and using the extended concept for unsaturated soils (Alonso et al., 1990; Thomas and He, 1998; Wheeler and Sivakumar, 1995; Cui et al., 1995; Wheeler, 1996; Bolzon et al., 1996; Rampino et al., 1999; Simoni and Schrefler, 2001). The difference lies in the assumptions and constitutive relationships that have been used in these models.

In the past few decades numerous researchers have studied the role of matric suction or degree of saturation in engineering properties of unsaturated soils. The influence of matric suction on the shear strength, volume change and excess pore-water pressure have been investigated (Bishop and Donald, 1961; Fredlund and Morgenstern, 1977; Fredlund et al., 1978; Alonso et al., 1987; Fredlund and Rahardjo, 1993; Wheeler and Karube, 1995; Graham et al., 1995; Delage and Graham, 1995; Wheeler, 1996; Cui and Delage, 1993; Wheeler and Sivakumar, 1993; Rampino et al., 1999, Tang and Graham, 2002 and Chiu and Ng, 2003).

Alonso et al. (1990) formulated an elasto-plastic model with four state variables: mean net stress p , deviator stress q , matric suction s and specific volume v . The plastic strains start when the state boundary hypersurface is reached. The model includes the yield loci, flow rules, hardening laws, and elastic and plastic strains. Wheeler and Sivakumar (1995) proposed an elasto-plastic model that is similar to the Alonso et al. (1990) model. However, the Wheeler and Sivakumar (1995) model proposed specific water volume as a fifth state variable.

It has been postulated that matric suction has a significant influence on soil behaviour in terms of volume changes, stress-strain and shear strength (Sivakumar, 1993; Fredlund et al., 1996; Vanapalli et al., 1996; Bolzon et al., 1996). Vanapalli et al. (1996) used the soil-water characteristic curve to predict the shear strength of unsaturated soils.

The behaviour of unsaturated soil can be described using four state variables, which are mean net stress, p , deviator stress, q , matric suction, s , and specific volume, v , (Alonso et al., 1990; Josa et al., 1992; Wheeler and Sivakumar, 1995). However, Wheeler (1996); Rampino et al. (2000); Wang et al. (2002) and Chiu and Ng (2003) stated that the specific water volume or degree of saturation should be taken into

account in the elasto-plastic constitutive framework models. In other words, the behaviour of unsaturated soil can be described using five state variables, which are mean net stress, p , deviator stress, q , matric suction, $s = (u_a - u_w)$, specific volume, v , and specific water volume, v_w , with the following definitions:

$$p = \left(\frac{\sigma_1 + \sigma_2 + \sigma_3}{3} \right) - u_a \quad (3.1)$$

$$q = \sigma_1 - \sigma_3 \quad (3.2)$$

$$s = u_a - u_w \quad (3.3)$$

$$v = 1 + e \quad (3.4)$$

$$v_w = 1 + Se \quad (3.5)$$

where:

- p = mean net stress,
- $\sigma_1, \sigma_2, \sigma_3$ = total principal normal stresses,
- u_a = pore-air pressure,
- q = deviator stress,
- s = matric suction of the soil,
- u_w = pore-water pressure,
- v = specific volume,
- e = void ratio,
- v_w = specific water volume, and
- S = degree of saturation.

The volumetric strain increment ($d\epsilon_v$) and shear strain increment ($d\epsilon_q$) can be calculated as,

$$d\epsilon_v = d\epsilon_1 + 2d\epsilon_3 \quad (3.6)$$

$$d\epsilon_q = \frac{2}{3}(d\epsilon_1 - d\epsilon_3) \quad (3.7)$$

where:

$d\epsilon_1$ and $d\epsilon_3$ are the principal strain increments.

The total strain increment ($d\epsilon$) is the sum of the elastic strain increment ($d\epsilon^e$) and the plastic strain increment ($d\epsilon^p$). The volumetric strain and the shear strain can be calculated as follows:

$$d\epsilon_v = d\epsilon_v^e + d\epsilon_v^p \quad (3.8)$$

$$d\epsilon_q = d\epsilon_q^e + d\epsilon_q^p \quad (3.9)$$

where:

$d\epsilon_v^e$ = elastic volumetric strain increment,

$d\epsilon_v^p$ = plastic volumetric strain increment,

$d\epsilon_q^e$ = elastic shear strain increment, and

$d\epsilon_q^p$ = plastic shear strain increment.

$$d\epsilon_v^e = d\epsilon_{v(p)}^e + d\epsilon_{v(s)}^e \quad (3.10)$$

$$d\epsilon_v^p = d\epsilon_{v(p)}^p + d\epsilon_{v(s)}^p \quad (3.11)$$

where:

$d\epsilon_{v(p)}^e$ = elastic volumetric strain increment induced by changes in mean net stress,

$d\epsilon_{v(s)}^e$ = elastic volumetric strain increment induced by changes in matric suction,

$d\epsilon_{v(p)}^p$ = plastic volumetric strain increment induced by changes in mean net stress, and

$d\epsilon_{v(s)}^p$ = plastic volumetric strain increment induced by changes in matric suction.

3.2.1 Elastic strains

A yield surface is the boundary of the elastic region. The changes in stresses within the yield surface are accompanied by purely elastic and recoverable deformations.

The elastic volumetric strain increment induced by changes in mean net stress can be calculated as follows (Alonso et al. 1990):

$$d\epsilon_{v(p)}^e = \frac{\kappa(s)}{v} \frac{dp}{p} \quad (3.12)$$

where:

$d\epsilon_{v(p)}^e$ = elastic volumetric strain increment with respect to mean net stress,

$\kappa(s)$ = elastic stiffness parameter from isotropic consolidation curve under a given matric suction,

dp = increment of mean net stress, and

p = mean net stress.

The elastic volumetric strain can also be calculated as follows:

$$d\epsilon_{v(p)}^e = \frac{dp}{K} \quad (3.13)$$

where:

K = bulk modulus, ($K = \frac{vp}{\kappa(s)}$).

The elastic shear strain increment can be calculated as follows:

$$d\epsilon_q^e = \frac{1}{3G} dq \quad (3.14)$$

where

G = shear modulus, and

dq = increment of deviator stress.

The shear modulus can be calculated from bulk modulus K and Poisson's ratio, ν :

$$G = \frac{3}{2} K \frac{(1-2\nu)}{(1+\nu)} \quad (3.15)$$

The elastic volumetric strain increment induced by the changes in matric suction can be calculated as follows:

$$d\epsilon_{v(s)}^e = \frac{\kappa_s}{\nu} \frac{ds}{(s + p_{at})} \quad (3.16)$$

where:

κ_s = elastic stiffness parameter from SWCC under a given net confining stress, and

p_{at} = atmospheric pressure.

3.2.2 Plastic strains

When the mean net stress reaches the yield surface, a change of stress will cause the soil to yield. The total volumetric strain can be expressed as follows (Alonso, et al. 1990):

$$d\epsilon_{v(p)} = \frac{\lambda(s)}{\nu} \frac{dp}{p} \quad (3.17)$$

where:

$\lambda(s)$ = stiffness of the normal consolidation line under a given matric suction, and

dp = increment of the mean net stress.

The plastic volumetric strain induced by the changes in mean net stress can be formulated as follows:

$$d\epsilon_{v(p)}^p = \frac{\lambda(s) - \kappa(s)}{v} \frac{dp}{p} \quad (3.18)$$

When the matric suction reaches the maximum past matric suction in the soil, the total volumetric strain induced by matric suction can be calculated as:

$$d\epsilon_{v(s)} = \frac{\lambda_s}{v} \frac{ds_o}{(s + p_{at})} \quad (3.19)$$

where:

λ_s = plastic stiffness parameter from SWCC under given net confining stress,
and

ds_o = increment of matric suction.

The plastic volumetric strain induced by matric suction changes can be formulated as follows:

$$d\epsilon_{v(s)}^p = \frac{\lambda_s - \kappa_s}{v} \frac{ds_o}{(s_o + p_{at})} \quad (3.20)$$

3.2.3 Loading – collapse (LC) yield curve

Figure 3-1 shows a typical isotropic consolidation test result in $(v - s - \ln p)$ space. The projection of the yield points at a given matric suction on the $(s - p)$ plane defines the loading-collapse (LC) yield curve. Once the LC yield curve is reached, a further increase of mean net stress or reduction of matric suction leads to an

expansion of the yield curve and irrecoverable plastic compression. Wheeler and Sivakumar (1995) proposed that the isotropic normal compression curve at a given matric suction can be calculated as:

$$v = N(s) - \lambda(s) \ln \left(\frac{p_c}{p_{at}} \right) \quad (3.21)$$

where:

$N(s)$ = specific volume of the normal consolidation line under a given matric suction at atmospheric pressure,

$\lambda(s)$ = stiffness parameter for the normal consolidation line under a given matric suction,

p_c = yield stress of unsaturated soil, and

p_{at} = atmospheric pressure.

Figure 3-1 Idealized isotropic consolidation tests at different matric suctions

The compressibility of a soil decreases with increasing matric suctions and the yield stress also increases with increasing matric suctions. The typical isotropic consolidation test result can be plotted in $(v - \ln p)$ space (Figure 3-2). Figure 3-3 shows the shape of the LC yield curve in $(s - \ln p)$ plane. When a stress state moves from point A (at the saturated condition) to a new point C by following the elastic path ABC (Figure 3-3), the corresponding change in specific volume, Δv , can be related to the stress changes by two elastic swelling indices $\kappa(s)$ and κ_s as follows (Alonso et al. 1990):

$$\Delta v = -\kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) - \kappa(s) \ln \left(\frac{p_c}{p_c^*} \right) \quad (3.22)$$

when:

Δv = change in specific volume,

$\kappa(s)$ = elastic stiffness parameter from isotropic consolidation tests under given matric suction, and

p_c^* = yield stress at saturated condition.

Considering an elastic stress path ABC joining two points A and C (Figure 3-3) on the same yield curve at different matric suctions, Wheeler and Sivakumar (1995) presented the following expression for the normal consolidation line:

Figure 3-2 Idealized isotropic consolidation tests in the $(v - \ln p)$ plane

Figure 3-3 Stress paths in the elastic zone in the $(s - \ln p)$ plane (Wheeler, 1996)

$$\Delta v = N(s) - \lambda(s) \ln \left(\frac{p_c}{p_{at}} \right) - N(0) + \lambda(0) \ln \left(\frac{p_c^*}{p_{at}} \right) \quad (3.23)$$

where:

$N(0)$ = specific volume of the normal consolidation line under zero matric suction at reference stress, and

$\lambda(0)$ = stiffness parameter for the normal consolidation line under zero matric

suction.

The LC yield curve can be expressed by equating Equations (3.22) and (3.23):

$$[\lambda(s) - \kappa(s)] \ln\left(\frac{p_c}{p_{at}}\right) = [\lambda(0) - \kappa(0)] \ln\left(\frac{p_c^*}{p_{at}}\right) + \kappa_s \ln\left(\frac{s + p_{at}}{p_{at}}\right) + N(s) - N(0) \quad (3.24)$$

where:

$\kappa(0)$ = elastic stiffness parameter from isotropic consolidation tests at zero matric suction.

The elastic volumetric strain can be computed as follows (Alonso et al. 1990):

$$d\epsilon_v^e = \frac{\kappa(s)}{v} \frac{dp}{p} + \frac{\kappa_s}{v} \frac{ds}{(s + p_{at})} \quad (3.25)$$

where:

$d\epsilon_v^e$ = increment of elastic volumetric strain,

$\kappa(s)$ = Elastic stiffness parameter from isotropic consolidation test under given matric suction,

v = specific volume,

p = mean net stress,

κ_s = elastic stiffness parameter from SWCC under net confining stress,

s = matric suction,

p_{at} = Atmospheric pressure.

Alonso et al. (1990) postulated that the evolution of the yield surfaces was controlled by the hardening parameters, p_c^* and s_0 . The hardening parameters can

be calculated based on the total plastic volumetric strain:

$$\frac{dp_c^*}{p_c^*} = \frac{vd\epsilon_v^p}{\lambda(0) - \kappa(0)} \quad (3.26)$$

where:

p_c^* = yield stress of saturated soil,

$d\epsilon_v^p$ = increment of plastic volumetric strain,

$\lambda(0)$ = stiffness parameter for the normal consolidated test under zero matric suction, and

$\kappa(s)$ = elastic stiffness parameter from isotropic consolidation test under zero matric suction.

$$\frac{ds_0}{(s_0 + p_{at})} = \frac{vd\epsilon_v^p}{(\lambda_s - \kappa_s)} \quad (3.27)$$

where:

s_0 = matric suction at yield point,

λ_s = plastic stiffness parameter from SWCC under given net confining stress, and

κ_s = elastic stiffness parameter from SWCC under given net confining stress.

Sivakumar (1993) developed the isotropic yield stress, p_c , equation at the current value of mean net stress, p , and specific volume, v . The elastic stiffness parameter, $\kappa(s)$, was assumed to be independent of matric suction. However, the elastic stiffness parameter is made to be function of matric suction as follows:

$$\ln p_c = \frac{N(s) - v - \kappa(s) \ln p + \lambda(s) \ln p_{at}}{[\lambda(s) - \kappa(s)]} \quad (3.28)$$

3.2.4 Flow rules

Ohmaki (1982) modified the expression for the associated flow rule by introducing a parameter α as follows:

$$d\epsilon_q^p = \frac{2 \alpha q}{M^2 (2p + p_s - p_c)} d\epsilon_v^p \quad (3.29)$$

where:

p_s = tensile strength on p axial and

α = soil parameter and can be calculated as follows:

$$\alpha = \frac{M(M-9)(M-3)}{9(6-M)} \left(\frac{1}{1 - \kappa(0)/\lambda(0)} \right) \quad (3.30)$$

3.2.5 Determination of the Mean Net Stress and the Deviator Stress at the Initial Yield Point

Determination of the mean net stress and the deviator stress at the initial yield point is an essential part for predicting volumetric strain, shear strength and excess pore-water pressure at the initial point.

The shape of the yield curve for an unsaturated soil can be computed using the Modified Cam Clay model proposed by Roscoe and Burland (1968). Figure 3-4 shows the yield curves for the saturated condition (i. e., zero matric suction plane) and the unsaturated condition (i.e., non-zero matric suction plane) on the $(q - p)$ plane.

For a zero matric suction condition, the yield curve must pass through the yield stress at saturated condition, p_c^* , and the original axis. If the increase in shear strength with respect to matric suction follows a linear relationship, the ellipse on a non-zero matric suction plane will intersect the p axis at a yield stress, p_c , and $-p_s$, which can be defined as follows (Alonso et al. 1990):

$$p = -p_s = -ks \quad (3.31)$$

where:

k = a constant.

Figure 3-4 Yield curves at different suction planes (after Alonso et al. 1990)

Alonso et al. (1990) assumed that the yield surface on a constant matric suction plane would be elliptical in shape with a constant aspect ratio M , to be consistent with the modified Cam clay model for a saturated soil.

The equation of the elliptical yield curve on the non-zero matric suction plane is as follows (Alonso et al. 1990):

$$q^2 = M^2 (p + p_s)(p_c - p) \quad (3.32)$$

where:

M = slope of the critical state line on the $(q - p)$ plane.

Figure 3-5 shows the idealized net stress path for a triaxial loading test on $(q - p)$ plane. The net stress path (i.e., line KY in Figure 3-5) approaches the total stress path (TSP). The equation representing the net stress path for a triaxial test can then be expressed as follows:

$$q = 3(p - p_o) \quad (3.33)$$

where:

p_o = mean net stress at the starting point of shearing.

Figure 3-5 Idealized stress paths for a triaxial compression test on $(q-p)$ plane

The mean net stress, p_y , and the deviator stress, q_y , at the initial yield point (i.e., point Y in Figure 3-5) can be determined from the intersection between the net stress path (line KY) and the initial yield surface. Hence, Equation (3.32) can be rewritten by replacing q with q_y and p with p_y as follows:

$$q_y^2 = M^2 (p_y + p_s)(p_c - p_y) \quad (3.34)$$

Meanwhile, the intersection point between the net stress path (line KY) and the initial yield surface also lies on the net stress path. Therefore, Equation (3.33) can be rewritten by replacing q with q_y and p with p_y :

$$q_y = 3(p_y - p_o) \quad (3.35)$$

The mean net stress and the deviator stress at the initial yield point can be obtained by substituting Equation (3.35) into Equation (3.34):

$$\left[3(p_y - p_o) \right]^2 = M^2 (p_s + p_y)(p_c - p_y) \quad (3.36)$$

$$\left[9 + M^2\right] p_y^2 - \left[18p_o + M^2(p_c - p_s)\right] p_y + \left[9p_o^2 - M^2 p_s p_c\right] = 0 \quad (3.37)$$

Equation (3.37) is a quadratic equation in terms of “ p_y ”. To solve Equation (3.37), the following relationship needs to be satisfied:

$$\left[18p_o + M^2(p_c - p_s)\right]^2 - 4\left[9 + M^2\right]\left[9p_o^2 - M^2 p_s p_c\right] \geq 0 \quad (3.38)$$

Equation (3.38) can be rewritten as follows:

$$M^2 \left[36(p_s + p_o)(p_c - p_o) + M^2(p_c + p_s)^2 \right] \geq 0 \quad (3.39)$$

Since $-p_s < p_o < p_c$, the term inside the bracket in Equation (3.39) is always positive. Therefore, the two solutions for Equation (3.37) can be obtained as follows:

$$p_{y1} = \frac{18p_o + M^2(p_c - p_s) + M\sqrt{\delta}}{18 + 2M^2} \quad (3.40)$$

and

$$p_{y2} = \frac{18p_o + M^2(p_c - p_s) - M\sqrt{\delta}}{18 + 2M^2} \quad (3.41)$$

where:

$$\delta = 36(p_s + p_o)(p_c - p_o) + M^2(p_c + p_s)^2$$

Only the first solution (i.e., p_{y1}) is feasible as $p_y \geq p_o$. Therefore, the mean net stress, p_y , at the initial yield point is as follows:

$$p_y = \frac{18p_o + M^2(p_c - p_s) + M\sqrt{\delta}}{18 + 2M^2} \quad (3.42)$$

The deviator stress at the initial yield point (i.e., q_y) can be obtained by replacing

p_y in Equation (3.42) into Equation (3.35):

$$q_y = 3 \left(\frac{18p_o + M^2(p_c - p_s) + M\sqrt{\delta}}{18 + 2M^2} - p_o \right) \quad (3.43)$$

The mean net stress and the deviator stress at the initial yield point will be used to calculate the volumetric strain, shear strength and excess pore-water pressure during shearing test.

3.3 Proposed Equations for Determination of the Model Parameters

One of the most important steps in the critical state framework is to compute the soil parameters under a given matric suction. This section proposed the equations for the calculation of the soil parameters in the elasto-plastic model for unsaturated soil by incorporating the SWCC in terms of normalized volumetric water content.

Figure 2.1 shows a typical soil-water characteristics curve (see Chapter 2). There are two distinct points of the soil-water characteristic curve: air-entry value (AEV) and residual volumetric water content (θ_r). The soil-water characteristic curve can also be used in the form of normalized volumetric water content (Θ). Figure 3-6 shows the normalized volumetric water content as generated in accordance with Figure 2.1. The normalized volumetric water content is defined as:

$$\Theta = \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \quad (3.44)$$

where:

θ_w = volumetric water content at a given matric suction,

θ_r = volumetric water content at residual state, and

θ_s = volumetric water content at saturated condition.

Figure 3-6 A typical normalized soil-water characteristic curve

Alonso et al. (1990) proposed the following exponential curve to compute the slope of the isotropic normal compression line for a constant suction loading ($\lambda(s)$). The $\lambda(s)$ was assumed to decrease monotonically with increasing matric suctions from the saturated value $\lambda(0)$ to an asymptotic value of $\lambda(s)$ at high suctions.

$$\lambda(s) = \lambda(0)[(1-r)\exp(-\beta s) + r] \quad (3.45)$$

where:

- r = constant related to the maximum stiffness of the soil, and
- β = parameter which controls the rate of increase of soil stiffness with suction.

Rampino et al. (2000) and Chiu and Ng (2003) used Equation (3.45) to obtain the slopes of the normal compression line (i.e., $\lambda(s)$) and the unloading line (i.e., $\kappa(s)$) at various matric suctions for unsaturated soils.

Parameter β in Alonso et al. (1990)'s equation (i.e., Equation (3.45)) controls the rate of change in $\lambda(s)$ with respect to a change in matric suction. In other words, parameter β defines the shape of $\lambda(s)$ parameter with respect to matric suction.

When parameter β increases, the rate of change in $\lambda(s)$ increases and when parameter β decreases the rate of change in $\lambda(s)$ also decreases. Parameter $\lambda(s)$ exhibits the largest change in the low range of matric suction. Parameter r is the ratio between the lowest value of any soil parameter and the value at the saturated condition. The influence of matric suction on $\lambda(s)$ decreases when parameter r increases and the influence of matric suction on $\lambda(s)$ increases when parameter r decreases.

Equation (3.45) does not incorporate SWCC in the determination of the soil parameters for the elasto-plastic model for unsaturated soil. Equation (3.45) would require at least three $\lambda(s)$ values in order to determine the parameters for the elasto-plastic model. Meanwhile, equation (3.45) would require at least two material parameters (i.e., β and r) and this can be demanding for identification and application purposes. One of the material parameters, β , can be eliminated by incorporating SWCC for the determination of soil parameters for the elasto-plastic model for unsaturated soil. The SWCC in the proposed equations governs the rate of change of $\lambda(s)$ with respect to matric suction. The incorporation of SWCC for determining soil parameters for the elasto-plastic model for unsaturated soil is expected to result in a more realistic model. The slope of the normal compression curve on the $(v - \ln p)$ plane can be determined as follows:

$$\lambda(s) = \lambda(0) - \frac{(1 - \Theta)}{m_\lambda} \quad (3.46)$$

where:

m_λ = soil parameter ($m_\lambda \neq 0$).

The form of Equation (3.46) can also be used to determine the slope of the unloading $\kappa(s)$, the intercept $N(s)$ of the normal compression line at reference stress and the yield stress with respect to matric suction as follows:

For the slope of the unloading curve:

$$\kappa(s) = \kappa(0) - \frac{(1-\Theta)}{m_\kappa} \quad (3.47)$$

where:

m_κ = soil parameter ($m_\kappa \neq 0$).

For the intercept of the normal compression line at reference stress:

$$N(s) = N(0) - \frac{(1-\Theta)}{m_N} \quad (3.48)$$

m_N = soil parameter ($m_N \neq 0$).

The initial LC yield curve can also be defined in a similar manner as follows:

$$p_c = p_c^* + \frac{(1-\Theta)}{n} \quad (3.49)$$

where:

n = parameter which controls the rate of change of soil stiffness with matric suction, kPa^{-1}

The incorporation of SWCC for the determination of the soil parameters in the elasto-plastic model can also be used for fitting the parameters $\alpha(s)$ and $A(s)$ as follows:

$$\alpha(s) = \frac{(1-\Theta)}{m_\alpha} \quad (3.50)$$

where:

m_α = soil parameter ($m_\alpha \neq 0$).

and

$$A(s) = \frac{(1 - \Theta)}{m_A} \quad (3.51)$$

m_A = soil parameter ($m_A \neq 0$).

3.4 Critical State

Figure 3-7 shows the idealized plot of the elliptical yield curve on a constant matric suction plane.

Figure 3-7 Idealized of the elliptical yield curve on a constant matric suction plane

Wheeler and Sivakumar (1995) proposed the critical states at various values of matric suctions by the critical state line as follows:

$$q = M(s)p + \mu(s) \quad (3.52)$$

$$v = \Gamma(s) - \omega(s) \ln \left[\frac{p_c}{p_{at}} \right] \quad (3.53)$$

where:

$M(s)$ = the gradient of the critical state line in the $(q - p)$ plane under given matric suction,

$\mu(s)$ = intercept of the critical state line at $p = 0$ under given matric suction,

$\Gamma(s)$ = intercept of the critical state line in the $(v - \ln p)$ plane at reference stress, and

$\omega(s)$ = the gradient of the critical state line in the $(v - \ln p)$ plane at given matric suction.

3.5 Prediction of the Change in Matric Suction during CW Test

In order to predict the variation of matric suctions during shearing under the constant water content condition, the elasto-plastic variation of specific water volume, v_w , proposed by Wheeler (1996) is adopted. However, the elastic stiffness parameter $\kappa(s)$ is made to be a function of matric suction. The specific air volume, v_a , can be calculated:

$$v_a = A(s) - \alpha(s) \ln \frac{p_c^*}{p_{at}} \quad (3.54)$$

where $A(s)$ and $\alpha(s)$ are functions of matric suction. The specific water volume, v_w , can be calculated by subtracting the specific volume of air, v_a , from the specific volume, v .

$$v_w = v - v_a \quad (3.55)$$

$$v_w = v - \left[A(s) - \alpha(s) \ln \left(\frac{p_c^*}{p_{at}} \right) \right] \quad (3.56)$$

The specific volume, v , at any point within the elastic zone can be calculated as follows:

$$v = N(0) - \lambda(0) \ln \left(\frac{p_c^*}{p_{at}} \right) - \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) - \kappa(s) \ln \left(\frac{p}{p_c^*} \right) \quad (3.57)$$

Substituting Equation (3.57) into Equation (3.56) gives:

$$v_w = N(0) - \lambda(0) \ln \left(\frac{p_c^*}{p_{at}} \right) - \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) - \kappa(s) \ln \left(\frac{p}{p_c^*} \right) - A(s) + \alpha(s) \ln \left(\frac{p_c^*}{p_{at}} \right) \quad (3.58)$$

The increment of specific water volume can be obtained by differentiating Equation (3.58) with respect to p_c^* as follows:

$$dv_w = [\alpha(s) - \lambda(0) + \kappa(s)] \frac{dp_c^*}{p_c^*} - \left[\frac{dA(s)}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d\alpha(s)}{ds} \ln \left(\frac{p_c^*}{p_{at}} \right) \right] ds - \kappa(s) \frac{dp}{p} \quad (3.59)$$

The elastic variation of the specific water volume can be determined by differentiating Equation (3.59) with respect to p_c^* . However, within the elastic region, p_c^* remains constant, therefore dp_c^* equals to zero:

$$dv_w^e = - \left\{ \frac{dA(s)}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d\alpha(s)}{ds} \ln \left[\frac{p_c^*}{p_{at}} \right] \right\} ds - \frac{\kappa(s) dp}{p} \quad (3.60)$$

where:

dv_w^e = elastic increment of specific water volume.

The plastic component of the change in specific water volume can be determined by subtracting Equation (3.60) from Equation (3.59):

$$dv_w^p = dv_w - dv_w^e \quad (3.61)$$

where:

dv_w^p = plastic increment of specific water volume.

$$dv_w^p = [\alpha(s) - \lambda(0) + \kappa(s)] \frac{dp_c^*}{p_c^*} - \left[\frac{dA(s)}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d\alpha(s)}{ds} \ln \left(\frac{p_c^*}{p_{at}} \right) \right] ds - \kappa(s) \frac{dp}{p} + \frac{\kappa(s) dp}{p} + \left[\frac{dA(s)}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d\alpha(s)}{ds} \ln \left(\frac{p_c^*}{p_{at}} \right) \right] ds \quad (3.62)$$

$$dv_w^p = [\alpha(s) - \lambda(0) + \kappa(s)] \frac{dp_c^*}{p_c^*} \quad (3.63)$$

During shearing under the constant water content condition, the specific water volume remains constant. Therefore, the increment of matric suction (ds) caused by shearing under the constant water content condition can be estimated by setting the total increment of specific water volume (i.e., Equation (3.59)) to zero:

$$dv_w = [\alpha(s) - \lambda(0) + \kappa(s)] \frac{dp_c^*}{p_c^*} - \left[\frac{dA(s)}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d\alpha(s)}{ds} \ln \left(\frac{p_c^*}{p_{at}} \right) \right] ds - \frac{\kappa(s) dp}{p} = 0 \quad (3.64)$$

$$\Rightarrow ds = \frac{[\alpha(s) - \lambda(0) + \kappa(s)] \frac{dp_c^*}{p_c^*} - \frac{\kappa(s) dp}{p}}{\frac{d(A(s))}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d(\alpha(s))}{ds} \ln \left(\frac{p_c^*}{p_{at}} \right)} \quad (3.65)$$

The parameters $\alpha(s)$ and $A(s)$ in Equation (3.65) can be obtained from the measured data of isotropic consolidation tests. The specific water volume can be calculated from Equation (3.56) as follows:

$$v_w = N(s) - \lambda(s) \ln \left[\frac{p_c}{p_{at}} \right] - A(s) + \alpha(s) \ln \left(\frac{p_c^*}{p_{at}} \right) \quad (3.66)$$

Substituting p_c^* from Equation (3.24) into Equation (3.66):

$$v_w = N(s) - \lambda(s) \ln \left(\frac{p_c}{p_{at}} \right) - A(s) - \frac{\alpha(s)}{[\lambda(0) - \kappa(0)]} \left\{ N(s) - N(0) - \lambda(s) \ln \left(\frac{p_c}{p_{at}} \right) + \kappa(s) \ln \left(\frac{p_c}{p_{at}} \right) + \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) \right\} \quad (3.67)$$

$$v_w = N(s) - A(s) - \frac{\alpha(s)}{[\lambda(0) - \kappa(0)]} \left\{ N(s) - N(0) + \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) \right\} - \left\{ \lambda(s) - \frac{\alpha(s)[\lambda(s) - \kappa(s)]}{\lambda(0) - \kappa(0)} \right\} \ln \left(\frac{p_c}{p_{at}} \right) \quad (3.68)$$

Equation (3.68) can be rewritten as follows:

$$v_w = N_w(s) - \lambda_w(s) \ln \left(\frac{p_c}{p_{at}} \right) \quad (3.69)$$

where:

$$N_w(s) = N(s) - A(s) - \frac{\alpha(s)}{\lambda(0) - \kappa(0)} \left[N(s) - N(0) + \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) \right] \quad (3.70)$$

$$\lambda_w(s) = \lambda(s) - \frac{\alpha(s)[\lambda(s) - \kappa(s)]}{\lambda(0) - \kappa(0)} \quad (3.71)$$

where:

$N_w(s)$ = specific water volume of the normal consolidation line under a given matric suction at atmospheric pressure, and

$\lambda_w(s)$ = slope of the normal consolidation for water phase under a given matric suction.

The values $\alpha(s)$ and $A(s)$ can be computed by rearranging Equations (3.70) and (3.71), respectively:

$$\alpha(s) = \frac{[\lambda(s) - \lambda_w(s)][\lambda(0) - \kappa(0)]}{[\lambda(s) - \kappa(s)]} \quad (3.72)$$

$$A(s) = N(s) - N_w(s) - \frac{\alpha(s)}{\lambda(0) - \kappa(0)} \left[N(s) - N(0) + \kappa_s \ln \left(\frac{s + p_{at}}{p_{at}} \right) \right] \quad (3.73)$$

The stress paths within the initial yield surface correspond to a constant p_c^* .

Therefore, dp_c^* is equal to zero (3.70) and Equation (3.65) can be simplified as follows:

$$ds = \frac{-\kappa(s)dp}{\frac{d(A(s))}{ds} + \frac{\kappa_s}{s + p_{at}} - \frac{d(\alpha(s))}{ds} \ln\left(\frac{p_c^*}{p_{at}}\right)} \quad (3.74)$$

Soil parameters involved in the constitutive models and the tests for determination of soil parameters, as well as typical values of each parameter are presented in Table 3.1.

Table 3.1 Soil parameters involved in the constitutive models and typical values of each parameter.

Parameters	Physical meaning	Type of tests	Typical value
$M(0)$	Slope of critical state line in the $(q - p')$ plane under zero matric suction	CD and CW triaxial tests under zero matric suction	0.8 – 1.5
$M(s)$	Slope of critical state line in the $(q - p')$ plane under a given matric suction	CD and CW triaxial tests under a given matric suction	0.8 – 1.5
$\lambda(0)$	Plastic stiffness parameter of the normal consolidation curve under zero matric suction	Isotropic consolidation test at saturated condition	0.05 – 0.15
$\lambda(s)$	Plastic stiffness parameter of the normal consolidation curve under given matric suction	Isotropic consolidation test at unsaturated condition for the CW tests	0.06 – 0.10
$\kappa(0)$	Elastic stiffness parameter of the unloading curve under zero matric suction	Isotropic consolidation test under zero matric suction	0.01-0.05
$\kappa(s)$	Elastic stiffness parameter of the unloading curve under given matric suction	Isotropic consolidation test under a given matric suction	0.01-0.05
$N(0)$	Specific volume of the normal consolidation curve under zero matric suction at reference stress	Isotropic consolidation test under zero matric suction	1.5 – 2.5
$N(s)$	Specific volume of the normal consolidation curve under given matric suction at reference stress	Isotropic consolidation test under a given matric suction	1.5 – 2.5
p_c^*	Preconsolidation stress of the saturated soil	Isotropic consolidation test under zero matric suction	>10
p'_c	Preconsolidation stress of the unsaturated soil	Isotropic consolidation test under a given matric suction	>10
λ_s	Plastic stiffness parameter from SWCC after air-entry value	Soil-water characteristic curve	0.01 – 0.1
κ_s	Elastic stiffness parameter from SWCC	Soil-water characteristic curve	0.001 – 0.005

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$\lambda_w(s)$	Plastic stiffness parameter for water phase under a given matric suction	Isotropic consolidation test under a given matric suction	0.001 – 0.1
$N_w(s)$	Specific volume of the normal consolidation curve for water phase under given matric suction at reference stress	Isotropic consolidation test under a given matric suction	0.1 – 2.0
$\alpha(s)$	Plastic stiffness for air phase under a given matric suction	Calculated from Equation (3.72)	0 – 0.10
$A(s)$	Specific volume of the critical state in the $(v - \ln p')$ plane under zero matric suction	Calculated from Equation (3.73)	0 – 0.90
$\Gamma(0)$	Specific volume of the critical state in the $(v - \ln p')$ plane under zero matric suction	CD and CW triaxial tests under zero matric suction	1.2 – 2.0
$\Gamma(s)$	Specific volume of the critical state in the $(v - \ln p')$ plane under a given matric suction	CD and CW triaxial tests under a given matric suction	1.2 -2.0

CHAPTER 4 RESEARCH PROGRAMME

4.1 Introduction

The research programme mainly focused on laboratory tests. This chapter describes the methods and procedures for conducting tests to obtain the basic soil properties, soil-water characteristic curve (SWCC), the isotropic consolidation test, the constant water content (CW) and the consolidated drained (CD) triaxial tests. The simulation of the triaxial response during shearing of the CW and CD tests was also carried out using the proposed theory given in Chapter 3.

4.2 Outline of Research Programme

The research programme consisted of six main parts:

1. Basic soil properties:

The index properties tests and tests for obtaining soil-water characteristic curves on the pressure plate were conducted on the selected soil.

2. Modification of triaxial apparatus and incorporating the NTU mini suction probes in the measurement pore-water pressure in soil specimen during triaxial tests.
3. Conducting a series of the isotropic consolidation tests under different matric suctions and at different initial dry densities on compacted specimens. Conducting a series of tests for obtaining soil-water characteristic curves under different net confining stresses and at

different initial dry densities on compacted specimens using the modified triaxial apparatus.

4. Conducting a series of the CW triaxial tests to study the shear strength, volume change and excess pore-water pressure characteristics during shearing of an unsaturated soil under a constant water content condition.
5. Conducting a series of the CD triaxial tests to study the shear strength, total volume change and water volume change characteristics during shearing of an unsaturated soil under a drained condition. To compare the shear strength parameters that were obtained from the CD triaxial tests with those data obtained from the CW triaxial tests.
6. Simulation of the shear strength, volume change and excess pore-water pressure generated during shearing under the constant water content and consolidated drained conditions using the theory developed in Chapter 3.

4.3 Preparation of the Compacted Specimens and Basic Soil Properties

The preparation of compacted specimen is an important part of the laboratory works.

4.3.1 Criteria for Preparing the Specimen

The soil specimen used in this study should meet the following conditions:

- 1) the soil specimens must be homogenous and uniform in initial dry density and initial water content in order to study the shear strength, pore-water pressure and volume change characteristics;
- 2) the unsaturated coefficient of permeability with respect to water phase should not be too low at high matric suctions in order for both air and water phases to be continuous during the tests and also for the tests not to be too time consuming; and

- 3) the soil particles should not be too large in order to avoid difficulties associated with specimen preparation and poor contact between the NTU mini suction probe and soil specimen.

Kaolin, which is essentially silt, was selected for the experimental program in this study since it meets all of the above conditions. The use of an homogeneous material, such as Kaolin, to verify the proposed critical state theory for unsaturated soil is crucial for reproducibility of identical specimens used in this testing programme, although Kaolin may not be truly representative of natural soils. The Kaolin used in this study was produced by Kaolin Malaysia SDN BHD.

4.3.2 Basic Soil Properties

The Atterberg limits, grain size analysis, specific gravity and permeability tests were conducted to determine the index properties of the compacted silt. Basic soil properties tests were conducted following ASTM (1997) standards.

4.3.2.1 Compaction Test

The standard compaction test procedure as given in ASTM No D698 was adopted. Identical statically compacted specimens with the same maximum dry density and optimum water content (i.e., $\rho_{d\max} = 1.35 \text{ Mg/m}^3$ and $w_{opt} = 22 \%$) were used in this experimental programme in order to minimize the heterogeneity of soil specimen. Figure 4.1 shows idealized of the compaction curve. In order to investigate the effects of the initial dry density and initial water content on the characteristics of SWCC and isotropic consolidation curve, the compacted specimens on the dry side (i.e., $\rho_{dd} = 1.30 \text{ Mg/m}^3$ and $w_d = 13 \%$) and the on wet side (i.e., $\rho_{dw} = 1.25 \text{ Mg/m}^3$ and $w_w = 36 \%$) of the optimum water content were also tested (see Figure 4.1).

A uniform density through the length of the specimen is difficult to obtain from dynamic compaction test. Therefore, static compaction method was used for obtaining identical specimens for this research. The details of the static

compaction method are discussed in the following sections.

Figure 4.1 Idealized of the compaction curve

4.3.3 Static Compaction Mould

Static compaction was conducted on silt specimens. The static compaction method described by Ong (1999) was adopted. Figure 4.2 shows the static compaction mould and stainless steel plugs used in this study. The static compaction mould consisting of a stainless steel and two stainless steel plugs are also shown in Figure 4.2. The stainless steel mould with a dimension of 70 mm x 70 mm x 200 mm and an internal diameter of 50 mm was used to compact the specimens. The mould body consists of two symmetrical parts that are connected together on two sides.

The stainless steel plug comprises a number of removable disks with a thickness of 10 mm. The disks can be attached together with a screw at the centre of the disks. The length of the plug can be changed by either removing or adding disks. Figure 4.3b shows the layout of removing or attaching the plugs. In order to reduce the friction between soil particles and the wall of the mould during compaction, the disk was made in T-shape. The base of the disk has a diameter of 50 mm and the top part of the disk has a diameter of 40 mm.

Figure 4.2 Static compaction mould and stainless steel plugs

Figure 4.3 Equipment for static compaction of specimens, (a) Connection between two adjacent disks; (b) removable disk; (c) small plug (after Ong 1999)

The thickness of each part of the disk is 5 mm as shown in Figure 4.3c. For attaching a pair of disks together, each of the disks is made such that one threaded screw protrudes out from one face of the disk, while a threaded hole is drilled into another face of the same disk. The screw is connected to the hole of the adjacent disk as shown in Figure 4.3b and 4.3c. All disks can be connected

in the same manner to form a complete plug.

4.3.4 Static Compaction Process

The silt mass and water mass were calculated to achieve the desired dry density and water content. The specimen was statically compacted in 10 layers of 10 mm thickness. The height and diameter of the specimen were 100 mm and 50 mm, respectively. In the first layer of compaction, the disks were placed from the bottom of the mould up to 100 mm into the compaction mould of 200 mm high. The required amount of the silt mixture for the first layer was placed over the disk from the top. The top plug with an assemblage of 9 disks fitted together with a total length of 90 mm was placed on top of the soil mixture inside the mould. The entire compaction mould with plugs and silt mixture was inserted into the triaxial frame as shown in Figure 4.4.

Figure 4.4 Compaction machine for static compaction specimens

In order to achieve a uniform density throughout the specimen and to avoid the building up of high excess pore-water pressure, the silt was compressed at a constant rate of 1 mm per minute. An axial load was applied onto the top plug until the soil layer was compressed to a thickness of 10 mm. In the second

layer, the mould was turned over so that the top plug would be at the bottom. The height of the plug at the top was then reduced to 90 mm by removing one disk in order to give space for the second layer. Before adding the soil for the second layer, the compacted soil surface was lightly scratched to ensure good bonding between the adjacent layers. The top plug was inserted into the mould and the axial load was applied with the same rate until a thickness of 20 mm was reached. This process, which included turning over the mould, removal of one disk to reduce the length of the top plug for compacting the second layer, scratching the surface and filling the new soil mixture, was repeated until a full specimen of 100 mm in height and 50 mm in diameter was achieved. The specimen was extruded using the same compression machine as shown in Figure 4.5.

Figure 4.5 Extrusion of the compacted silt specimen

4.3.5 Tests for Obtaining SWCC using Pressure Plate

The pressure plate was used for obtaining the SWCC. The statically compacted soil specimens used were 20 mm in thickness and 50 mm in diameter. The

specimens used for obtaining SWCC were prepared in the same manner as the soil specimens for triaxial tests. Figure 4.6 shows the set up of the pressure plate with a 5 bar high air-entry ceramic disk and a rubber membrane beneath the disk. The space between the disk and the rubber membrane serves as a water compartment. The water compartment is connected to a burette line that is opened to atmospheric pressure. The number of specimens that can be tested in a pressure plate depends on the available disk space. The ceramic disk was saturated prior to test. The saturation was done by pouring the de-aired distilled water on top of the disk and applying a high air pressure of 500 kPa while opening the valve of the burette line for about 1 hour. Due to the high pressure in the chamber, the distilled de-aired water infiltrated through the ceramic plate.

Figure 4.6 Set up of pressure plate extractor (after Agus, 2001)

The saturation of specimens was done by spraying de-aired distilled water on top of the specimens and left overnight in the extractor. The specimens were taken out and weighed the next day. The specimens were put back into the pressure plate and a matric suction of 20 kPa was then applied to the specimen by increasing air pressure in the pressure extractor to 20 kPa and leaving the burette line opened to atmospheric pressure (i.e., $u_a = 20$ kPa and $u_w = 0$ kPa). Due to the application of matric suction to the soil specimens, the pore-water in the specimens would be drained out through the ceramic disk. The mass of each specimen was recorded at the same time intervals as those used in the

consolidation test until the specimens reached equilibrium. The same procedure was repeated in order to increase the matric suction to 20 kPa, 50 kPa, 100 kPa, 200 kPa, 400 kPa and 700 kPa incrementally.

4.4 Triaxial Set up and Its Development

The modified triaxial apparatus used in this study for obtaining SWCCs under different net confining stresses, the isotropic consolidation curves under different matric suctions, the CW and CD triaxial tests were similar to the modified triaxial apparatus described by Fredlund and Rahardjo (1993). This section provides the test procedures and the modified triaxial apparatus for obtaining SWCCs, the isotropic consolidation tests, the CW and CD triaxial tests.

4.4.1 Modified Triaxial Apparatus for the Soil-water Characteristic Curve Tests (SWCC)

The modified triaxial apparatus allows the control of both pore-air, u_a and pore-water pressures, u_w , using the axis-translation technique (Hilf, 1956) and consequently the matric suction, $(u_a - u_w)$, of the specimen. Figure 4.7 shows the layout of the modified triaxial cell for unsaturated soils testing. The NTU mini suction probes were also installed along the specimen in order to measure the pore-water pressure during testing.

Figure 4.8 shows the schematic diagram of plumbing of the modified triaxial apparatus for obtaining SWCC. During testing, the confining pressure, pore-air pressure and pore-water pressure were applied by opening valves D, A and C, respectively. During flushing, valves B and C, were opened so that the air bubbles accumulated in the water compartment could be removed by water pressure gradient.

Figure 4.7 Modified triaxial cell for unsaturated soils testing (Modified from Fredlund and Rahardjo, 1993).

Figure 4.9 shows the modified triaxial set up and its accessories. The modified triaxial apparatus for obtaining SWCCs consists of a triaxial compression cell; a digital pressure and volume controller (DPVC-2) for back pressure; a digital pressure and volume controller (DPVC-1) for cell pressure; diffused-air volume indicator (DAVI); three pore pressure transducers, data acquisition unit and personal computer. The NTU mini suction probes for pore-water pressure measurements in an unsaturated soil specimen were incorporated in the modified triaxial apparatus during testing.

The cell pressure, σ_3 , was controlled by the DPVC-1. The cell volume change recorded by the DPVC-1 was also used to measure the volume change of the specimen during testing. The air-pressure, u_a , is controlled by the air pressure system. The water-pressure at the base plate, u_w , was controlled by the DPVC-2. All of the transducers and NTU mini suction probes were calibrated prior to test and for every 6-month period during the testing programme. The calibration

data are presented in Appendix A.

Figure 4.8 Schematic diagram of plumbing for the modified triaxial apparatus for obtaining SWCC

Figure 4.9 Assemblage of the modified triaxial apparatus for obtaining SWCC

4.4.1.1 Triaxial cell

In order to incorporate three NTU mini suction probes, a large triaxial cell was required for the triaxial test. The triaxial cell dimension is 201 mm in diameter, 380 mm in height and 16 mm in thickness. The maximum capacity of the cell pressure is 1700 kPa.

4.4.1.2 Special Adapters at Triaxial Base

Conventional triaxial apparatus requires modifications prior to its use in testing unsaturated specimens. Figure 4.10 shows a circular grooved water compartment in the pedestal head with the high air-entry ceramic disk being removed. A pedestal head with water compartment and a 5-bar high air-entry ceramic disk was attached to the triaxial base. To control the pore-water pressure, a high air-entry ceramic disk was sealed onto the base pedestal of the cell. The 5-bar high air-entry ceramic disk of 50 mm in diameter and 6.35 mm in thickness was glued into the water compartment by applying the slow setting epoxy resin around its circumference.

The coefficient of permeability of the high – air entry ceramic disk with respect to water phase is 2.36×10^{-10} m/s). There are two ports at the pedestal base connected to two small holes in the water compartment which can be seen in Figure 4.10. The ports are connected with valves. One of the ports is attached with a pressure transducer for the measurement of pore-water pressure. Another one is connected to the DPVC for controlling the water pressure and measuring the water volume change during testing.

Figure 4.10 A circular grooved water compartment in the pedestal head with the high air entry disk removed

Cables for the three NTU mini suction probes were assembled through the extension ring sited on the triaxial base as shown in Figure 4.11. The NTU mini suction probes in the triaxial cell were then connected externally with a wire passing through the extension ring as shown Figure 4.11. The O-ring would be squeezed by the washer as the adapter was tightened to prevent the water leakage. NTU Mini suction probes were then connected to the data logger and to a personal computer. The calibration data and graphs for the NTU mini suction probes used in this research programme are shown in Appendix A.

Figure 4.11 A typical wire of NTU mini suction probe to pass through the extension ring at the triaxial base

4.4.1.3 Pore-water Pressure Control System

Water will cavitate under a pressure below -100 kPa. When the cavitation occurs, the water compartment will be partially filled with air. Therefore, erroneous measurements of pore-water pressure will occur. In order to prevent the occurrence of cavitation in the measuring system, the axis-translation technique proposed by Hilf (1956) was adopted for controlling matric suction in the soil specimen. The pore-air and pore-water pressures were controlled to reach the targeted matric suction. In this way, the applied air and water pressures are greater than zero and the problems associated with cavitation could be prevented.

A soil specimen was placed directly on top of the high air-entry disk to permit a continuous water column between the pore-water in the soil specimen and the controlling / measuring system. Therefore, the pore-water pressure in the soil

specimen could be controlled by controlling the water pressure in the water compartment through DPVC. The water pressure in the water compartment can be measured by the pore-water pressure transducer that was connected to the water compartment at the base plate.

4.4.1.4 Pore-air pressure Control System

The modified triaxial cell allowed the control of both pore-air and pore-water pressures. A filter paper, a coarse corundum disk and a perspex loading cap were placed on top of the soil specimen in this order. A small hole was drilled in the loading cap and it was connected to a port which was connected to an air pressure line. The air pressure was in contact with the specimen through the coarse corundum disk. A filter paper was placed below the coarse corundum disk. The filter paper together with the corundum disk was placed in between the soil specimen and the loading cap. The disk was connected to the pore-air pressure control line through the drilled hole in the loading cap. The corundum disk, which has a very low air-entry value, would allow air to go through when the air-entry value was exceeded. As a result, continuity between air voids in the specimen and the pore-air pressure control system could be established for an independent control of air.

4.4.1.5 Flushing System

To remove air bubbles which could be trapped in the water compartment during unsaturated soil testing, a flushing system was required. Air diffused through water in the ceramic disk and accumulated in the water compartment as air bubbles. If not removed, these air bubbles could impede the flow of water from and into the specimen and also cause incorrect measurements of pore-water pressures and volume changes (Ho and Fredlund, 1982). The flushing system is made up of an extra pore-water drainage line with a control valve connected to the base of the triaxial apparatus. When flushing, the control valve was momentarily opened to allow the accumulated air bubbles to be flushed out through the additional drainage line.

4.4.1.6 Measurement of Matric Suction in the Soil Specimen

NTU mini suction probes developed by Meilani et al, (2002) were used to measure pore-water pressures in the soil specimen. The NTU mini suction probe uses a pore-water pressure transducer PDCR 81, manufactured by Druck Ltd. The original ceramic disk of the mini pore-water pressure probe PDCR 81 was removed and replaced by a new 5 Bar high air-entry ceramic disk as described in Chapter 2. Figure 4.12 shows the NTU mini suction probe for measuring pore-water pressures in soil specimens.

Figure 4.12 NTU mini suction probe

In order to obtain the pore-water pressure changes in a soil specimen during the tests, three NTU mini suction probes were installed along the soil specimen. Figure 4.13 shows the locations of the NTU mini suction probes along the soil specimen. The probes were installed at $3/4$ (i.e. “top”), $1/2$ (i.e. “middle”) and $1/4$ (i.e. “bottom”) heights of the specimen from its base and at 120 degrees apart in the lateral direction.

Figure 4.13 illustrates the details of an NTU mini suction probe. A silicon rubber grommet was needed to hold NTU mini suction probes and to prevent cell water from entering soil specimen (Figure 4.14). The internal hole of the

grommet must fit well to the external diameter of NTU mini suction probe for easy instalment of the probe to the soil specimen. After pushing the probe into the rubber grommet and ensuring that the face of the NTU mini suction probe was in good contact with soil specimen, two O-rings were assembled to give additional protection from leakage (Figure 4.14).

Figure 4.13 Installation details for NTU mini suction probes

A three-part split membrane stretcher was used so that the silicone rubber grommets could be installed through the rubber membrane and the membrane stretcher could be removed from the specimen. The original membrane stretcher designed by Wong (2000) and modified by Meilani et al. (2002) was adopted. Figure 4.15 shows the three parts of the membrane stretcher with three rubber holders at different heights (i.e., 1/4, 1/2 and 3/4 heights of the specimen). Figure 4.16 shows the full assemblage of the membrane stretcher with rubber holders. These rubber holders are the places where the silicone rubber grommets were attached to.

Figure 4.14 Details of NTU mini suction probe on silt specimen

Figure 4.15 Three split parts of the membrane stretcher with rubber holders

Figure 4.16 Full assemblage of the membrane stretcher

4.4.1.7 Data Acquisition System

A data acquisition program Triax 4.0 developed by Toll (1999) was used as part of the data acquisition system. The data acquisition system provides channels for 3 pressure transducers and three NTU mini suction probes. The pore-air pressure was controlled at a desired value by an air pressure system. The pore-water pressure at the base plate was controlled by the DPVC-2. As a result, the matric suction as a difference between the pore-air pressure and the pore-water pressure (i.e., $(u_a - u_w)$) could be controlled. The water volume change of the specimen during the tests was measured by recording the volume change on DPVC-2 which indicated the amount of water that flowed into or out of the specimen. The DPVC-1 was used to control cell pressure and measure volume change of the specimen during testing. All recorded data were transferred to a personal computer using Triax 4.0 and they could be displayed on the PC monitor and then opened as an Excel file. The data could be copied to floppy disk without disturbing or stopping the test.

4.4.2 Modified Triaxial Apparatus for Isotropic Consolidation Tests

In order to investigate the effects of matric suction on the characteristics of the isotropic consolidation curves, a series of isotropic consolidation tests was conducted on compacted silt specimens under different matric suctions. Meanwhile, the effects of the initial dry density and initial water content on the characteristics of the isotropic consolidation curve were also investigated. The isotropic consolidation tests were conducted using the modified triaxial apparatus. The schematic diagram of the plumbing for the modified triaxial apparatus for the isotropic consolidation tests is the same as that for the modified triaxial apparatus for obtaining SWCC. The assemblage of the modified triaxial apparatus for the isotropic consolidation tests is shown in Figure 4.17.

Figure 4.17 Assemblage of modified triaxial apparatus for isotropic consolidation test

The modified triaxial apparatus for the isotropic consolidation test is the same as the modified triaxial apparatus for obtaining SWCC and it was described in

previous section except for some differences as follows. A small triaxial cell was utilised. The cell is made of perspex wall with 102.6 mm in inner diameter and 2488 mm in height and 16 mm in thickness. The cell can also sustain an applied cell pressure of 1700 kPa. The cell can accommodate a soil specimen with a maximum diameter and height of 50 mm and 100 mm, respectively. The DPVCs were used to measure the water volume and total volume changes of a specimen during saturation, matric suction equalization and isotropic consolidation stages.

4.4.3 Modified Triaxial Apparatus for the CW and CD Triaxial Tests

Basically, the modified triaxial apparatus for the CW and CD tests was the same with the one used for obtaining SWCCs and isotropic consolidation curves except that the air pressure was controlled by DPVC (i.e., valve A in Figure 4.8). The modified triaxial apparatus for the CW and CD tests consists of a compression machine, a triaxial cell, a load cell, linear variable differential transformer (LVDT), two DPVCs for water, a DPVC for air, a diffused air volume indicator (DAVI), 3 pore pressure transducers, three NTU mini suction probes, data acquisition and personal computer. The pore pressure transducers, NTU mini suction probes, load cell and LVDT were calibrated prior to test and for every 6 months period during the testing programme. The calibration data are presented in Appendix C.

During the CW shearing stage, the confining pressure and pore-air pressure were applied by opening valves D, and A, respectively. During the CD shearing stage, the confining pressure, pore-air pressure and pore-water pressure were applied by opening valves D, A and C, respectively. Meanwhile, during flushing, valves B and C were opened so that the air bubbles that accumulated in the water compartment could be removed by water pressure gradient. The assemblage of the modified triaxial apparatus for the CW and CD triaxial tests is shown in Figure 4.18. The digital pressure and volume controllers were used to measure water volume changes in a specimen and cell volume changes during the tests.

Figure 4.18 Assemblage of modified triaxial apparatus for the CW and CD tests

4.5 Testing Procedure

The testing procedure for triaxial tests on saturated soils as given by Head (1986) was adopted. The procedure for triaxial tests on unsaturated soils follows the procedure described in Fredlund and Rahardjo (1993). The initial matric suctions of the specimens were established using the axis-translation technique.

4.5.1 Testing Procedure for SWCC Tests

4.5.1.1 Saturation Stage

The soil specimen was initially saturated for all the tests in order to obtain consistent initial water content and degree of saturation. In this stage, the air pressure line was replaced by water pressure line connected to a digital pressure and volume controller to inject water into the specimen from the top. During the

saturation stage, the specimen was saturated by applying a cell pressure, σ_3 , and a back pressure, u_w , under an effective confining pressure of 10 kPa until the pore-water pressure parameter, B , was close to 1. Full saturation was assumed to have been achieved when the B value was greater than 0.97 (Head 1986). Each specimen required about four days for the saturation stage to be completed.

4.5.1.2 Consolidation Stage

After saturation, the soil specimen was isotropically consolidated to the desired net confining stress, $(\sigma_3 - u_w)$. During the consolidation stage, the specimen was allowed to consolidate under an isotropic confining stress, σ_3 , and a pore-water pressure, u_w . The magnitudes of the pressures for the consolidation stage were selected based on the desired values of the net confining stress, $(\sigma_3 - u_a)$. During the isotropic consolidation, the water valve was left opened and the pressures were controlled at the desired values. The amount of water that drained out from the soil specimen during the isotropic consolidation was recorded by the DPVC and the data were transferred to the personal computer. The total volume change of the specimen during the test was calculated from the volume change on the DPVC which indicated the amount of water that flowed into or out of the triaxial cell. The consolidation was assumed to have been completed when the water volume change levelled off and the excess pore-water pressure had fully dissipated. Each specimen required about 1 hour for the consolidation stage to complete. After the isotropic consolidation had been completed, matric suction was applied to the soil specimen.

4.5.1.3 Drying and Wetting Stages (i.e., Matric suction equalization stages)

Once the consolidation stage had been completed, the line connected to the top loading cap was disconnected from the DPVC and reconnected to the air pressure system. The DPVC was subsequently connected to the pore-water pressure at the base plate (i.e., u_w). To control the pore-water pressure, a high air-entry ceramic disk was sealed onto the base pedestal of the cell (as discussed in previous section). The water pressure in the water compartment was

measured by the pore-water pressure transducer that was connected to the water compartment at the base plate. During matric suction equalization stages, the net confining stress was maintained at the desired value by controlling the cell pressure and air pressure. During the drying stage, matric suction was increased by decreasing the pore-water pressure at the base plate while maintaining the pore-air pressure. Meanwhile for the wetting stage, matric suction was decreased by increasing the pore-water pressure at the base plate.

The amount of water that drained out from the soil specimen and the total volume change of the specimen during drying and wetting were recorded by the DPVCs (i.e., DPVC for cell and DPVC for back pressure) and stored in a personal computer. The equalization was assumed to have been achieved when the excess pore-water pressure had fully dissipated and water volume change reduced to 0.04 % per day as suggested by Sivakumar (1993). All of recorded data were transferred to a personal computer by using Triax 4.0 (Toll 1999) and could be displayed on the PC monitor and then opened as an Excel file.

4.5.2 Testing Procedure for Isotropic Consolidation Tests

4.5.2.1 Saturation and Matric Suction Equalization Stages

The saturation stage was conducted in the same manner as the tests for obtaining SWCCs. Once the saturation stage had been completed, the matric suction equalization stage was carried out subsequently. The net confining stress at the end of the saturation stage was maintained at the value of 10 kPa, by controlling the cell pressure and air pressure. During the matric suction equalization stage the cell pressure and air pressures were kept at the same values as at the end of the saturation stage. Meanwhile, the pore-water pressure at the base was set to the desired value. The water and air valves were opened during the matric suction equalization stage. The amount of water drained out from the soil specimen and the total volume change of the specimen during the matric suction equalization stage was recorded by the DPVCs and the data were transferred to the personal computer. The matric suction equalization stage was deemed to be completed when there was negligible water drainage from the soil specimen or

the pore-water pressures along the soil specimen had come to equilibrium.

4.5.2.2 Isotropic Consolidation Stage

Once the matric suction equalization had been completed, the isotropic consolidation tests were conducted at a constant matric suction. The air-pressure and pore-water pressures were kept constant throughout the consolidation stage at the same values as controlled during the matric suction equalization stage. The net confining stress, $(\sigma_3 - u_a)$, was increased during the isotropic consolidation by increasing the cell pressure using the DPVC. The isotropic consolidation stage was conducted under a constant rate of stress loading – unloading of 6 kPa / hour.

The amount of water that drained out from the soil specimen during the isotropic consolidation was recorded by the DPVC and the data were transferred to the personal computer. The total volume change of the specimen during the test was also monitored using the DPVC. The isotropic consolidation stage was deemed to be completed when there was negligible water flow either into or out of the soil specimen.

4.5.3 Testing Procedure for Constant Water Content Tests

4.5.3.1 Saturation, Consolidation and Matric Suction Equalization Stages

The saturation, consolidation and matric suction equalization stages in the CW triaxial tests were carried out in the same manner as in the tests for obtaining SWCCs. Full saturation was assumed to have been achieved when the B value was greater than 0.97.

4.5.3.2 Shearing Stage

When an equilibrium condition had been achieved under the applied net confining stress, $(\sigma_3 - u_a)$, and matric suction, $s = (u_a - u_w)$, the specimen was then sheared by applying an axial load at a constant strain rate. Rahardjo et al. (2004) conducted a pilot triaxial test to investigate the effect of strain rate on

matric suction and shear strength of compacted specimens of the residual soil from the Jurong Sedimentary Formation of Singapore. The pilot test was conducted under a constant water content condition with different strain rates varying from 0.009 to 0.081mm/min. The results showed that the change of strain rate from 0.009mm/min to 0.081mm/min did not produce a significant difference in the shear strength and matric suction during shearing. Rahardjo et al. (2004) used the strain rate of 0.009mm/min for the compacted residual soil from the Jurong Sedimentary Formation. In this study, the strain rate of 0.009 mm/min was also used because the silt had similar properties (i.e., plasticity index, coefficient of permeability) with the properties of the residual soil from the Jurong Sedimentary Formation of Singapore. The soil specimen was sheared under drained conditions for the pore-air phase and undrained conditions for the pore-water phase. This means that during shearing the valve for the air pressure line was opened and controlled at a required pressure while the valve for the water pressure line was closed. In this shearing stage, the pore-air pressure, u_a , was maintained at the same magnitude as that obtained at the end of the equalization stage. The pore-water pressure, u_w , in the soil specimen would increase or decrease depending on the volume change of the soil specimen and this pore-water pressure was measured using NTU mini suction probes. The difference between the pore-air pressure, u_a , and the pore-water pressure, u_w , gives the matric suction, $(u_a - u_w)$. Shearing was terminated when the deviator stress, $q = (\sigma_1 - \sigma_3)$, reached a constant value or a distinct shear plane in the specimen had been observed. The maximum axial strain was set to 20%. Each specimen required about one to three days for the shearing stage to be completed.

4.5.4 Testing Procedure for the CD Triaxial Tests

4.5.4.1 Saturation, Consolidation and Matric Suction Equalization Stages

The saturation, consolidation and matric suction equalization stages were conducted in the same manner as those performed in the CW triaxial tests. Once the matric suction equalization had been completed, the specimen was sheared

under a consolidation drained condition.

4.5.4.2 Shearing Stage

Rahardjo et al. (2004) used the strain rate of 0.0009 mm/min for the compacted residual soil from the Singapore Jurong Sedimentary Formation. In this study, the strain rate of 0.0009 mm/min was also used because the silt had similar properties (i.e., plasticity index, coefficient of permeability) with the properties of the residual soil from the Singapore Jurong Sedimentary Formation. The soil specimen was sheared under drained conditions for the pore-air and pore-water phases. This means that during shearing the valves for the air pressure and water pressure lines were opened and controlled at the required pressures. In this shearing stage, the pore-air pressure, u_a , and pore-water pressure, u_w , were maintained at the same magnitudes as those obtained at the end of the equalization stage. Shearing was terminated when the deviator stress, $q = (\sigma_1 - \sigma_3)$, reached a constant value or a distinct shear plane in the specimen had been observed. The maximum axial strain was set to 20%.

4.5.5 Final Measurement

At the end of each test (i.e., SWCC, isotropic consolidation test, CW and CD triaxial tests), the pore-air pressure applied to the specimen was first reduced to zero. The cell pressure was then reduced to zero and the water in the cell was drained out. The NTU mini suction probes were then removed carefully from the specimen. The specimen was then dismantled from the pedestal and the membrane was removed from the specimen. The final mass of the specimen was measured immediately after removing the membrane from the specimen. The specimen was divided into ten layers and water content was determined for each layer.

4.6 Testing Programme

4.6.1 SWCC Tests under Different Net Confining Stresses

In the series of SWCC tests under different net confining stresses, the net confining

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stress for each SWCC tests was selected as the same with the net confining stress used in the constant water content triaxial tests (i. e., $(\sigma_3 - u_a) = 10 \text{ kPa}, 50 \text{ kPa}, 100 \text{ kPa}, 150 \text{ kPa}, 200 \text{ kPa}, 250 \text{ kPa}$ and 300 kPa). Meanwhile, the SWCC tests were conducted at different initial dry densities and initial water contents in order to study the effect of initial dry density and initial water content on the characteristics of the SWCC. Table 4.1 shows the testing programme for the SWCC under different net confining stresses and dry densities. A naming convention is adopted for each test as SWCCm – n where the term m refers to soil specimens that were compacted at the maximum dry density (i.e., opt), the dry side (i.e., d) and the wet side (i.e., w) of the optimum water content. The term n means that the test was conducted under a net confining pressure of n kPa. Figure 4.19 shows the idealized changes in specific volume during the SWCC tests under different net confining stresses. The changes in water content during the SWCC tests under different net confining stresses can also be idealized as illustrated in Figure 4.20.

Table 4.1 Programme for the SWCC under different net confining stresses

Net confining stresses, $(\sigma_3 - u_a)$ (kPa)						
10	50	100	150	200	250	300
SWCCopt-10	SWCCopt-50	SWCCopt-100	SWCCopt-150	SWCCopt-200	SWCCopt-250	SWCCopt-300
SWCCd-10		SWCCd-100		SWCCd-200		SWCCd-300
SWCCw-10		SWCCw-100		SWCCw-200		SWCCw-300

Figure 4.19 Idealized specific volume versus matric suction from SWCC tests under constant net confining stress

Figure 4.20 Idealized water content versus matric suction from SWCC tests under constant net confining stress

The stress conditions that were used in the SWCC tests are tabulated in Table 4.2. Figure 4.21 shows the stress paths followed during the soil-water characteristic curve tests.

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Table 4.2 Stress conditions that were used in SWCC tests under different net confining stresses

SWCC tests under net confining stress, $(\sigma_3 - u_a) = 10$ kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20
SWCC tests under net confining stress = 50 kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20
SWCC tests under net confining stress, $(\sigma_3 - u_a) = 100$ kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20
SWCC tests under net confining stress, $(\sigma_3 - u_a) = 150$ kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20
SWCC tests under net confining stress, $(\sigma_3 - u_a) = 200$ kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20
SWCC tests under net confining stress, $(\sigma_3 - u_a) = 250$ kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20
SWCC tests under net confining stress, $(\sigma_3 - u_a) = 300$ kPa												
Loading step	1	2	3	4	5	6	7	8	9	10	11	12
$(u_a - u_w)$, (kPa)	0	20	40	80	160	320	480	320	160	80	40	20

Figure 4.21 Stress path for soil-water characteristic curve tests

4.6.2 Testing Programme for Isotropic Consolidation Tests

The matric suction for each isotropic consolidation test was also chosen to be the same as the initial matric suction for the CW and the matric suction in the CD triaxial tests. Figure 4.22 shows the idealized plot of the changes in specific volume with respect to the net confining stress during the isotropic consolidation tests under constant matric suction. The change in water content during the isotropic consolidation tests under constant matric suctions can also be idealized as shown in Figure 4.23. Meanwhile, in order to investigate the effect of the initial dry density and initial water content on the characteristics of the isotropic consolidation curve, a series of the isotropic consolidation curve at different initial dry densities and initial water content was also conducted on the compacted silt specimens. The testing programme for the isotropic consolidation tests is shown in Table 4.3. A naming convention for the tested soil specimens was adopted using a designation of ISCT-u. The terms t refer to soil specimens at the maximum dry density (i.e., opt), dry side (i.e., d) and wet side (i.e., w) of the optimum water content. The term u in ISCT-u means that the test was conducted under a matric suction of u kPa. Figure 4.24 shows the stress paths that were used in this research programme. Table 4.4 shows the stress conditions that were used in the isotropic consolidation tests.

Figure 4.22 Idealized specific volume versus net confining stress from isotropic consolidation tests under constant matric suction

Figure 4.23 Idealized water content versus net confining stress from isotropic consolidation tests under constant matric suction

Table 4.3 Programme for the isotropic consolidation tests in the triaxial apparatus under different matric suctions

Saturated specimen	Unsaturated specimens				
Matric suction, $(u_a - u_w)$ (kPa)					
0	50	100	150	200	300
ISCOpt-0	ISCOpt-50	ISCOpt-100	ISCOpt-150	ISCOpt -200	ISCOpt-300
ISCd-0	ISCd-50	ISCd-100	ISCd-150	ISCd -200	ISCd-300
ISCw-0	ISCw-50	ISCw-100	ISCw-150	ISCw -200	ISCw-300

Figure 4.24 Stress path for isotropic consolidation tests

Table 4.4 Initial stresses conditions that were used in the isotropic consolidation tests under different matric suctions

Matric suction ($u_a - u_w$) (kPa)	σ_3 (kPa)	u_a (kPa)	u_w (kPa)	($\sigma_3 - u_a$) (kPa)
0	300	290	290	10
50	300	290	240	10
100	300	290	190	10
150	300	290	140	10
200	300	290	90	10
300	310	300	0	10

4.6.3 Testing Programme for Constant Water Content Tests

The CW triaxial test was carried out using a single stage triaxial test. In order to investigate the failure envelope for an unsaturated soil, the tests were conducted at different initial matric suctions and under different net confining stresses. To prevent the NTU mini suction probes from cavitation at high matric suctions during shearing, the maximum initial matric suction of 300 kPa was used in this study. The initial matric suctions of 0 kPa, 50 kPa, 100 kPa, 150 kPa, 200 kPa and 300 kPa were chosen for the testing programme. The testing programme for the constant water content test is shown in Table 4.5. A naming convention for the CW tests was adopted using a designation of CWx-y. The term x-y in CWx-y means that the test was conducted under a net confining pressure of x kPa and at an initial matric suction of y kPa.

Table 4.5 Programme for the constant water content triaxial tests

Net Confining Stress, ($\sigma_3 - u_a$) (kPa)	Matric Suction, ($u_a - u_w$) (kPa)				
	0	100	150	200	300
25	-	CW25-100	-	-	-
50	CW50-0	CW 50-100	CW50-150	CW50-200	CW50-300
100	CW100-0	CW100-100	CW100-150	CW100-200	CW100-300
150	CW150-0	CW150-100	CW150-150	CW150-200	CW150-300
200	CW200-0	CW200-100	CW200-150	CW200-200	CW200-300
250	CW250-0	CW250-100	CW250-150	CW250-200	CW250-300
300	CW300-0	CW300-100	CW300-150	CW300-200	CW300-300
350	-	CW350-100	CW350-150	-	-

4.6.4 Testing Programme for the Consolidated Drained Tests

The CD triaxial tests were also carried out using single stage tests. The CD triaxial tests were conducted at constant matric suctions of 0 kPa, 100 kPa, 200 kPa and 300 kPa. The testing programme for the CD triaxial tests is shown in Table 4.6. A naming convention for the CD tests was also adopted using a designation of CDa-b. The term a-b in CDa-b means that the test was conducted under a net confining pressure of a kPa and a matric suction of b kPa.

Table 4.6 Programme for the consolidated drained triaxial tests

Net Confining Stress, ($\sigma_3 - u_a$) (kPa)	Matric Suction, ($u_a - u_w$) (kPa)			
	0	100	200	300
100	CD100-0	CD100-100	CD100-200	CD100-300
200	CD200-0	CD200-100	CD200-200	CD200-300
300	CD300-0	CD300-100	CD300-200	CD300-300

4.7 Theoretical simulation of the Shear Strength, Excess Pore-water Pressure and Volume Change during Shearing under the CW and CD Triaxial Tests

The shear strength, excess pore-water pressure and volume change developed during shearing under the constant water content triaxial tests conducted in this research programme were simulated. Meanwhile, the shear strength and volume change during shearing under the consolidated drained condition were simulated. The basic soil parameters for the numerical simulations were obtained from the isotropic consolidation tests under different matric suctions and the SWCC under different net confining stresses. The simulations were carried out using the proposed elasto-plastic with the incorporation of SWCC presented in Chapter 3. The procedures for simulating the excess pore-water pressure based on the proposed elasto-plastic model with the incorporation of SWCC are as follows:

4.7.1 Shear strength, Excess Pore-water Pressure and Volume Change at the Initial Yield Surface

To predict the shear strength, excess pore-water pressure and volume change during shearing, it is necessary to predict the stress state at the initial yield surface. The stress state within the initial yield surface was determined by calculating the mean net stress, p_y , and the deviator stress, q_y , at the initial yield surface. The mean net stress at a point on the initial yield surface could be calculated from Equation (3.42) and the deviator stress at a point on the initial yield surface could be determined from Equation (3.43) given in Chapter 3.

4.7.2 Shear Strength, Excess Pore-water Pressure and Volume Change beyond the Initial Yield Surface

To predict the shear strength, excess pore-water pressure and volume change during shearing beyond the initial yield surface, it is also necessary to predict the stress state. The deviator stress, q_i , could then be calculated from Equation

(3.32). The volume change and excess pore-water pressure can be calculated at each step stress increment.

CHAPTER 5 PRESENTATION OF RESULTS

5.1 Introduction

This chapter presents the results of the basic soil property tests. The SWCC tests under different net confining stresses and at different initial dry densities and water contents are presented. The results of the isotropic consolidation curves conducted under different matric suctions and at different initial dry densities are also presented. The results of the CW and CD triaxial tests under different matric suctions and at different net confining stresses are presented in the subsequent section. The triaxial test results are presented in forms of the shear strength, pore-water pressure and volume change during shearing. Meanwhile, the interpretation of the CW and CD triaxial test results using the extended Mohr – Coulomb failure envelope are presented in the last section of this chapter.

5.2 Basic Soil Properties

5.2.1 Index Properties

The index properties of the kaolin are presented in Table 5.1. The kaolin consists of 85 % of silt and 15 % of clay. The kaolin or silt has a liquid limit of 51, a plastic limit of 35.6 and a plasticity index of 15.4. The coarse kaolin is classified as silt with high plasticity (MH) according to the Unified Soil Classification System (ASTM D 2487 – 93, 1997). The standard Proctor compaction test on the silt gave a maximum dry density, $\rho_{d\max}$, of 1.35 Mg/m³ and an optimum water content, w_{opt} , of 22%, as indicated in Figure 5-1. The dry densities corresponding to the water contents that were obtained from the compaction curve are presented in Table 5.2 and Figure 5-1.

Table 5.1 Soil properties of statically compacted silt specimens

Soil properties	Value
Specific gravity, G_s	2.65
Liquid limit, LL (%)	51.0
Plastic limit, PL (%)	35.6
Plasticity index	15.4
Clay (%)	15
Silt (%)	85
Unified Soil Classification System (USCS)	MH (silt of high plasticity)
Coefficient of permeability at saturation, k_s (m/s)	6.4×10^{-8}
Coefficient of permeability at k_w (100kPa) (m/s)	2.4×10^{-9}
Coefficient of permeability at k_w (200kPa) (m/s)	2.4×10^{-10}

Figure 5-1 Compaction curve of the silt under standard Proctor compaction tests

Table 5.2 Dry densities with respect to water contents of the compaction silt specimen based on standard Protor

	Maximum dry density	Wet side	Dry side
Dry density, ρ_d (Mg/m ³)	$\rho_{d\max} = 1.35$	$\rho_{dw} = 1.25$	$\rho_{dd} = 1.30$
Water content, w (%)	$w_{opt} = 22.0$	$w_w = 36.0$	$w_d = 13.0$

5.2.2 Soil-Water Characteristic Curves

As discussed in Chapters 2 and 3, SWCC relates gravimetric water content, w , or volumetric water content, θ_w , to matric suction. The volumetric water content is defined as the ratio between volume of water and total volume of the soil (i.e., $\theta_w = V_w / V_0$). The SWCC can also be presented in the form of normalized volumetric water content (i.e., Θ) (see Equation (3.44)).

5.2.2.1 SWCC from Pressure Plate

Figure 5-2 shows the SWCC of a silt specimen that was statically compacted at the maximum dry density of 1.35 Mg / m³ and at the optimum water content of 22%. The SWCC is presented the graph of the volumetric water content versus matric suction. As indicated in Figure 5-2, there was a significant decrease in the volumetric water content when the matric suction in the specimen exceeded the air-entry value. The soil-water characteristic curve of the silt specimen indicated that the air-entry value (AEV) of the statically compacted silt specimen was 47 kPa.

No measurement of the total volume of the compacted silt specimen during testing was taken. Difficulty in measuring the total volume of the compacted silt specimen in the pressure plate was encountered. The measurement of the soil specimen using Vernier calliper might not be accurate and it caused the loss soil mass during the measurement. The loss in soil mass of the specimen would affect the weight measurement of the specimen. Therefore, the total volume of the soil specimen was assumed to remain constant during testing in the pressure plate.

Figure 5-2 SWCC of a statically compacted silt specimen from pressure plate

5.2.2.2 SWCCs from the Modified Triaxial Apparatus

In order to investigate the effects of the net confining stress and dry density on SWCC, the SWCC of the statically compacted silt was measured at different net confining stresses and at three different initial dry densities using the modified triaxial apparatus.

➤ **SWCCs of the Compacted Specimens at the Maximum Dry Density,**

$$\rho_{d \max} = 1.35 \text{ Mg/m}^3, \text{ and the Optimum Water Content, } w_{opt} = 22\%$$

Typical results of the change in pore-water pressure from the base plate, top, middle and bottom NTU mini suction probes for specimen SWCC-200 during the drying and wetting stages are presented in Figures 5.3 and 5.4, respectively. During the drying stage, the water pressure in soil specimen decreased rapidly to approach the pore-water pressure at the base plate (Figure 5-3). The pore-water pressure in the soil specimen came to equilibrium and reached the pore-water pressure at the base plate after approximately 50 hours. During the wetting stage, the water pressure in the soil specimen increased rapidly to approach the pore-water pressure at the base plate (Figure 5-4). The water pressure in the soil specimen came to equilibrium and reached the water pressure at the base plate after about 65 hours.

Figure 5-3 Matric suction equalization during drying stage for SWCC-200

Figure 5-4 Matric suction equalization during wetting stage for SWCC-200

The pore-water pressure from the NTU mini suction probes showed some variations during drying and wetting stages. However, all pore-water pressure measurements from the NTU mini suction probes approached the controlled water pressure at the base plate at the end of the matric suction equalization stages. Therefore, the NTU mini suction probes that were placed along the height of the soil specimen provided a good

indication that matric suction equalization in the soil specimen had been achieved.

Figures 5.5 to 5.11 show the SWCCs under the net confining stresses of 10 kPa, 50 kPa, 100 kPa, 150 kPa, 200 kPa, 250 kPa and 300 kPa, respectively. The SWCC tests were conducted using the modified triaxial apparatus. There are two curves in each figure (V_v/V_0 versus matric suction and V_w/V_0 versus matric suction (i.e., SWCC)). The V_v/V_0 term is equivalent to porosity or void ratio of a soil specimen. Figures 5.5 to 5.11 show that the volume of the soil specimen decreased with the increase in matric suction. Matric suction at the inflection point (i.e., the point at which the volume of the soil specimen decreased rapidly) defined as the yield suction, s_0 , of the soil specimen (Alonso et al. 1990). The volumetric water content is defined as the ratio between volume of water and initial total volume of the soil (i.e., $\theta_w = V_w/V_0$). The air-entry value (AEV) is defined as the matric suction value at which air first enters the pores of the soil. The test results shown in Figures 5.5 to 5.11 indicate that the air-entry value, AEV increased with the increase in the net confining stresses. The SWCCs under different net confining stresses are summarized in Figure 5-12.

Figure 5-5 Volume change and water volume change with respect to matric suction for specimen SWCC – 10

Figure 5-6 Volume change and water volume change with respect to matric suction for specimen SWCC – 50

Figure 5-7 Volume change and water volume change with respect to matric suction for specimen SWCC – 100

Figure 5-8 Volume change and water volume change with respect to matric suction for specimen SWCC – 150

Figure 5-9 Volume change and water volume change with respect to matric suction for specimen SWCC – 200

Figure 5-10 Volume change and water volume change with respect to matric suction for specimen SWCC – 250

Figure 5-11 Volume change and water volume change with respect to matric suction for specimen SWCC - 300

Figure 5-12 Soil-water characteristic curve tests at different net confining stresses

Figure 5-13 Specific volume versus matric suction for the compacted silt specimen at the maximum dry density and optimum water content

The summary of the specific volume (i.e., $v = I + e$) versus matric suction as obtained from the triaxial tests for obtaining SWCCs under different net confining stresses are presented in Figure 5-13. The results from Figure 5-13 indicate that the yield points with respect to matric suction (i.e., yield suction, s_0) increased with the increase in net confining stresses. The slope of the drying paths beyond s_0 seems to be independent of net confining stress. The air-entry and the yield suction values of the compacted silt specimens conducted under different net confining stresses are presented in Table 5.3. Table 5.3 indicates that the air-entry and yield suction values increased with the increase in the net confining stresses.

Table 5.3 Summary of the soil parameters obtained from SWCC on the compacted silt specimens at the maximum dry density and optimum water content

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Air-entry value, AEV (kPa)	Suction increase yield point, s_0 (kPa)
10	50	42
50	60	49
100	64	50
150	68	52
200	71	54
250	72	56
300	73	57

➤ **SWCCs of the Compacted Specimens at a Initial Dry Density,**

$\rho_{dd} = 1.30 \text{ Mg/m}^3$, **and Initial Water Content, $w_d = 13\%$**

A series of tests for obtaining SWCCs were conducted on a compacted silt specimen at the initial dry density of 1.30 Mg/m^3 and initial water content of 13%. Figure 5-14 shows the SWCCs under different net confining stresses (i.e., 10 kPa, 100 kPa, 200 kPa and 300 kPa). The test results shown in Figure 5-14 indicate that the AEV increased with the increase in net confining stress. The AEVs of the compacted silt specimens conducted under different net confining stresses on specimens at the initial dry density of 1.30 Mg/m^3 and the initial water content of 13 % are presented in Table 5.4.

Figure 5-14 SWCCs at a constant net confining stress on the compacted silt specimens at the initial dry density of 1.30 Mg/m^3 and initial water content of 13 %

Figure 5-15 Specific volume versus matric suction on compacted silt specimens at initial dry density of 1.30 Mg/m^3 and initial water content of 13 %

The specific volume, v (i.e., $v = 1 + e$) versus matric suction of the SWCC tests under different net confining stresses is shown in Figure 5-15. The results from Figure 5-15 indicate that the yield points increased with the increase in net confining stress.

The slope of the drying paths beyond the yield suction appears to be essentially independent of net confining stress. The yield suction values obtained from tests conducted under different net confining stresses on the compacted silt specimens at the initial dry density of 1.30 Mg/m^3 and initial water content of 13% are presented in Table 5.4.

Table 5.4 Summary of the soil parameters obtained from SWCC tests on compacted silt specimens at the initial dry density of 1.30 Mg/m^3 and initial water content of 13%

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Air-entry value, AEV (kPa)	Suction increase yield point, s_0 , (kPa)
10	48	34
100	58	38
200	60	44
300	65	51

➤ **SWCCs of the Compacted Specimens at a Initial Dry Density, $\rho_{dw} = 1.25 \text{ Mg/m}^3$, and Initial Water Content, $w_w = 36\%$**

A series of tests for obtaining SWCCs were also conducted on a compacted silt specimen at the initial dry density of 1.25 Mg/m^3 and initial water content of 36%. Figure 5-16 shows the SWCCs under different net confining stresses (i.e., 10 kPa, 100 kPa, 200 kPa and 300 kPa). The test results shown in Figure 5-16 indicate that the AEV increased slightly with the increase in net confining stress. The specific volume, v (i.e., $v = 1 + e$) versus matric suction of the SWCC tests under different net confining stresses is shown in Figure 5-17. The results from Figure 5-17 indicate that the yield points increased with the increase in net confining stress.

Figure 5-16 SWCCs at a constant net confining stress on the compacted silt specimens at the initial dry density of 1.25 Mg/m^3 and initial water content of 36 %

Figure 5-17 Specific volume versus matric suction on compacted silt specimens at initial dry density of 1.25 Mg/m^3 and initial water content of 36 %

The AEVs of the compacted silt specimens conducted under different net confining stresses on specimens at the initial dry density of 1.30 Mg/m^3 and the initial water content of 13 % are presented in Table 5.5. The slope of the drying paths beyond the yield suction appears to be essentially independent of net confining stress. The yield suction values obtained from tests conducted under different net confining stresses on the compacted silt specimens at the initial dry density of 1.25 Mg/m^3 and initial water content of 36% are presented in Table 5.5.

Table 5.5 Summary of the soil parameters obtained from SWCC tests on compacted silt specimens at the initial dry density of 1.25 Mg/m^3 and initial water content of 36%

Net confining stress, ($\sigma_3 - u_a$) (kPa)	Air-entry value, AEV (kPa)	Suction increase yield point, s_0 , (kPa)
10	39	33
100	44	35
200	45	43
300	47	46

5.2.3 Isotropic Consolidation Curves

5.2.3.1 Isotropic Consolidation Curves for a Compacted Specimen at a Maximum Dry Density and at an Optimum Water Content

Six isotropic consolidation tests were conducted on the compacted silt specimens at the maximum dry density, $\rho_{d \max}$, of 1.35 Mg/m^3 and optimum water content, w_{opt} , of 22%. One specimen was conducted under a saturated condition, while five other specimens were isotropically consolidated under an unsaturated condition. Figure 5-18 shows the isotropic consolidation curves for saturated and unsaturated soil specimens. The test results shown in Figure 5-18 clearly indicate that the soil specimens started to yield as the mean net stress or net confining stress increased. The yield stress at the saturated condition was 22 kPa. The results also indicate that the yield stress increased with the increase in matric suction. In other words, the increase in matric suction increased the strength of the specimen and as a result, the

yield stress increased. These results are in agreement with the results from Burland (1965), Alonso et al. (1990) and Rampino et al. (2000).

Figure 5-18 Isotropic compression curves at constant matric suction for the compacted silt specimens at the maximum dry density and optimum water content

Figure 5-19 shows the change in slope of the normal compression curve, $\lambda(s)$, with respect to matric suction for the compacted silt specimens. The $\lambda(s)$ value of 0.063 was found from the isotropic consolidation curve at the saturated condition. The $\lambda(s)$ decreased with the increase in matric suction. The $\lambda(s)$ value decreased rapidly as matric suction increased from 50 kPa to 200 kPa. The $\lambda(s)$ value decreased gradually when matric suction exceeded 200 kPa.

Figure 5-20 shows the intercepts of the normal compression lines at reference stress, $N(s)$, with respect to matric suction. In this study, the reference stress of 100 kPa was chosen as suggested by Wheeler and Sivakumar (1995).

The intercepts of the normal compression lines with respect to matric suction at the reference stress were not a constant. These measured results are in agreement with

the results reported by Rampino et al. (2000) (see Figure 5.20). The $N(s)$ value at reference stress of 1.936 was obtained from the isotropic consolidation curve at the saturated condition. The $N(s)$ decreased with the increase in matric suction. The significant change in $\kappa(s)$ occurred at matric suctions from 50 kPa to 200 kPa. The changes in $N(s)$ with respect to matric suction were small when matric suction was higher than 200 kPa.

Figure 5-19 Measured $\lambda(s)$ values with respect to matric suction from isotropic consolidation curves

Figure 5-21 shows the slopes of the unloading curve, $\kappa(s)$, for the isotropic consolidation curves. Most of the researchers in the past (Alonso et al. 1990; Wheeler and Sivakumar 1995; Wheeler 1996; Bolzon et al. 1996; Robles and Elorza 2002; Chiu and Ng 2003) assumed that the slopes of the unloading curves are independent with matric suction. However, the results from Figure 5-21 indicate that the slopes of the unloading curves varied with respect to matric suction. The slopes of the unloading curves decreased with the increase in matric suction. The $\kappa(s)$ value of 0.027 was obtained from the unloading curve at the saturated condition. The significance change in $\kappa(s)$ occurred at the matric suction range from 50 kPa to 200 kPa. The $\kappa(s)$ decreased gradually when matric suction was higher than 200 kPa and remained essentially constant when matric suction exceeded 200 kPa.

Figure 5-20 Measured $N(s)$ values with respect to matric suction from isotropic consolidation curves

Figure 5-21 Measured $\kappa(s)$ values with respect to matric suction from isotropic consolidation curves

The yield stress is equivalent to preconsolidation stress. The yield stress is defined as the mean net stress at which the specific volume of soil specimen decreased

significantly with the increase in mean net stress. Figure 5-22 shows the yield stresses with respect to matric suction. The yield stress at the saturated condition was about 22 kPa. The results shown in Figure 5-22 indicate that the yield stress increased dramatically when matric suction increased from zero to 200 kPa. The change in the yield stress decreased when matric suction exceeded 200 kPa.

Figure 5-22 Measured p_c values with respect to matric suction from isotropic consolidation curves

Table 5.6 summarizes of the isotropic consolidation parameters of the compacted silt specimens at the maximum dry density and the optimum water content. Most of the isotropic consolidation parameters were measured directly from the isotropic consolidation curves. The parameters $\alpha(s)$ and $A(s)$ representing the water phase were calculated in accordance with the respective equations given in Chapter 3. The specific volume at the reference stress, $N(s)$, the stiffness parameter of the normal consolidation line, $\lambda(s)$, and the elastic stiffness parameter, $\kappa(s)$, decreased with the increase in matric suction.

Table 5.6 Summary of the soil parameters obtained from isotropic consolidation curves of the compacted silt specimens at the maximum dry density of 1.35 Mg/m^3 and optimum water content of 22%

Matric suction (kPa)	$\lambda(s)$	$\kappa(s)$	$N(s)$	p_c (kPa)	$\lambda_w(s)$	$\alpha(s)$	$A(s)$
0	0.063	0.027	1.937	22	0.063	0.000	0.0000
50	0.057	0.024	1.929	72	0.038	0.014	0.1062
100	0.047	0.015	1.914	160	0.011	0.038	0.4587
150	0.043	0.013	1.907	210	0.008	0.043	0.5772
200	0.040	0.011	1.902	235	0.004	0.047	0.6429
300	0.039	0.010	1.897	244	0.001	0.053	0.7134

5.2.3.2 Isotropic Consolidation Curves of the Compacted Specimens at the Initial Dry Density, $\rho_{dw} = 1.25 \text{ Mg/m}^3$ and Initial Water Content, $w_w = 36\%$

The isotropic consolidation tests were also conducted on a compacted silt specimens at an initial dry density, ρ_{dw} , of 1.25 Mg/m^3 and initial water content, w_w , of 36%. The isotropic consolidation tests were conducted at different matric suctions (i.e., zero kPa, 50 kPa, 100 kPa, 150 kPa, 200 kPa and 300 kPa). Figure 5-23 shows the isotropic consolidation curves for the saturated and unsaturated soil specimens. The yield stress at the saturated condition was 21 kPa. The results also indicate that the yield stress increased with the increase in matric suction. The specific volume at the reference stress, $N(s)$, stiffness parameter of the normal consolidation line, $\lambda(s)$, and elastic stiffness parameter, $\kappa(s)$, decreased with the increase in matric suction. The parameters associated with the isotropic consolidation curves of the compacted silt specimen at the initial dry density of 1.25 Mg/m^3 and initial water content of 36% are shown in Table 5.7

Figure 5-23 Isotropic compression curves at constant matric suction for the compacted silt specimens at the initial dry density of 1.25 Mg/m^3 and initial water content of 36%

Table 5.7 Summary of the soil parameters obtained from isotropic consolidation curves of the compacted silt specimens at the initial dry density of 1.25 Mg/m^3 and initial water content of 36%

Matric suction (kPa)	$\lambda(s)$	$\kappa(s)$	$N(s)$	p_c (kPa)	$\lambda_w(s)$	$\alpha(s)$	$A(s)$
0	0.102	0.022	2.164	22	0.102	0.000	0.000
50	0.091	0.014	2.160	70	0.067	0.031	0.071
100	0.088	0.012	2.148	147	0.031	0.068	0.461
150	0.080	0.011	2.145	175	0.017	0.083	0.605
200	0.074	0.010	2.142	200	0.008	0.095	0.709
300	0.072	0.009	2.140	205	0.007	0.096	0.760

5.2.3.3 Isotropic Consolidation Curves of the Compacted Specimens at the Initial Dry Density, $\rho_{dd} = 1.30 \text{ Mg/m}^3$ and Initial Water Content, $w_d = 13\%$

A series of the isotropic consolidation tests were also conducted on compacted silt specimens at an initial dry density, ρ_{dd} , of 1.30 Mg/m^3 and initial water content, w_d , of 13%. The isotropic consolidation tests were conducted at different matric suctions (i.e., zero kPa, 50 kPa, 100 kPa, 150 kPa, 200 kPa and 300 kPa). Figure 5-24 shows the isotropic consolidation curves for the saturated and unsaturated soil specimens. The results also indicate that the yield stress increased with the increase in matric suction. The specific volume at the reference stress, $N(s)$, stiffness parameter of the normal consolidation line, $\lambda(s)$, and elastic stiffness parameter, $\kappa(s)$, decreased with the increase in matric suction. The parameters associated with the isotropic consolidation curves of the compacted silt specimen at the initial dry density of 1.30 Mg/m^3 and initial water content of 13% are shown in Table 5.8

Figure 5-24 Isotropic compression curves for the compacted silt specimens at the initial dry density of 1.30 Mg/m^3 and initial water content of 13%

Table 5.8 Summary of the soil parameters obtained from isotropic consolidation curves of the compacted silt specimens at the initial dry density of 1.30 Mg/m^3 and initial water content of 13%

Matric suction (kPa)	$\lambda(s)$	$\kappa(s)$	$N(s)$	p_c (kPa)	$\lambda_w(s)$	$\alpha(s)$	$A(s)$
0	0.065	0.025	2.046	22	0.065	0.000	0.000
50	0.062	0.019	2.045	72	0.033	0.031	0.415
100	0.055	0.014	2.035	155	0.010	0.056	0.599
150	0.05	0.013	2.027	189	0.007	0.060	0.706
200	0.048	0.011	2.023	214	0.006	0.061	0.818
300	0.047	0.010	2.020	219	0.003	0.065	0.902

Three-dimensional views of the constitutive surfaces for the compacted silt specimens are presented in Figure 5.25. The specific volume, v , and specific water volume, v_w , are plotted with respect to stress state variables in Figure 5.25(a) and 5.25(b), respectively.

(a)

Figure 5-25 Three-dimensional views of the constitutive surfaces for the compacted silt specimens. (a) specific volume with respect to stress state variables; (b) specific water volume with respect to stress state variables

5.3 Constant Water Content (CW) Triaxial Test Results

A series of the CW triaxial tests were carried out on statically compacted silt specimens at the maximum dry density and optimum water content (i.e., $\rho_{d\max} = 1.35 \text{ Mg/m}^3$ and $w_{opt} = 22\%$). The specimens were tested in the modified triaxial apparatus under net confining stresses of 25 kPa, 50 kPa, 100 kPa, 150 kPa, 200 kPa, 250 kPa, 300 kPa and 350 kPa and at different initial matric suctions (i.e., matric suction of zero kPa, 100 kPa, 150 kPa, 200 kPa and 300 kPa). A name convention for the tested soil specimens was adopted using a designation of CWx-y (see Chapter 4). The terms x-y mean that the test was conducted under a net

confining stress of x kPa and matric suction of y kPa. The CW triaxial tests results could be presented for the same initial matric suction but at different net confining stresses

5.3.1 Failure Criteria

Ong (1999) conducted a series of the CW triaxial tests under different net confining stresses and initial matric suctions. Various failure criteria were used in this study such as: (i) the peak deviator stress; (ii) the principal stress ratio; and (iii) the stress path approach. The study also showed that the maximum deviator stress could be used as a failure criterion for the specimens with strain softening behaviour. The specimen was considered to have failed when the deviator stress had reached a maximum value during shearing. The failure criteria are discussed in detail later in this chapter.

5.3.2 Shear Strength Behaviours

The results from shearing tests under different net confining stresses but at the same initial matric suction of 0 kPa are presented in Figure 5-26. Figure 5-26 shows that the deviator stress remained essentially constant after reaching its maximum value. As shown in Figure 5-26, the stiffness of the soil specimens generally increased with the increase in net confining stress. The void ratio, e , water content, w , and degree of saturation, S , at each stage of the CW triaxial test are presented in Table 5.9.

Figure 5-26 Deviator stress versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of zero kPa

The stresses at failure can be calculated as follows:

$$q_f = (\sigma_{1f} - \sigma_{3f}) \quad (5.1)$$

$$p_f = \frac{\sigma_{1f} + \sigma_{3f}}{2} - u_a \quad (5.2)$$

where:

- q_f = deviator stress at failure,
- σ_{1f} = major principal stress at failure,
- σ_{3f} = minor principal stress at failure, and
- p_f = mean net stress at failure.

Table 5.9 Void ratio (e), water content (w), and degree of saturation (S) of the CW triaxial tests under different net confining stresses but at the same initial matric suction of zero kPa

State	Before saturation				After consolidation			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
50	0.962	22.07	60.80	1.35	0.972	36.22	98.14	0.972	36.22	98.14
100	0.962	21.94	60.44	1.35	0.929	34.69	98.32	0.929	34.69	98.32
150	0.973	21.88	59.59	1.35	0.909	34.06	99.29	0.909	34.06	99.29
200	0.969	21.73	59.43	1.35	0.903	33.24	98.86	0.903	33.24	98.86
250	0.964	21.87	60.12	1.35	0.892	33.03	99.69	0.892	33.03	99.69
300	0.986	22.02	59.18	1.35	0.888	31.97	97.49	0.888	31.97	97.49

Table 5.10 Summary of the axial strain, deviator stress, mean net stress and matric suction at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 0 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)
50	20	135	118
100	11	239	220
150	14	354	327
200	16	462	431
250	18	537	519
300	15	576	588

Table 5.10 summarizes the axial strain, ϵ_{yf} , deviator stress, q_f , mean net stress, p_f and matric suction, $(u_a - u_w)_f$ at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of zero kPa.

The results from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa are presented in Figure 5-27. Figure 5-27 shows that most of the stress-strain curves showed post-peak strain softening. However, the ductility of the specimens increased with the increase in the net confining stress. The higher the net confining stress was, the less distinct the peak deviator stress would be. This could be explained by the fact that the tests conducted under a high net confining stress would cause the specimens to be more restrained in dilation during shearing. Therefore, the dilation of the specimen during shearing would be small. As a result, the deviator stresses decreased gradually after reaching the peak value. At low net confining stresses, the soil specimen behaved as an overconsolidated soil, where the soil specimen would be more brittle and the deviator stress could decrease significantly after reaching the peak value. The higher the net confining stress was, the higher the axial strain required for the specimen to reach the maximum deviator stress. The shear strength and stiffness of the soil specimen increased with the increase in net confining stress. The void ratio, water content and degree of saturation at each stage of the CW triaxial tests are presented in Table 5.11. The water content at the end of the shearing stage was close to the initial water content of the compacted silt specimen (i.e., 22%). The void ratio at the end of shearing appears to decrease with the increase in net confining stress.

Figure 5-27 Deviator stress versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Table 5.11 Void ratio (e), water content (w), and degree of saturation (S) of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

State	Before saturation				After matric suction equalization			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
25	0.962	21.98	60.6	1.35	0.953	22.55	62.70	0.963	22.55	62.05
50	0.968	22.01	60.3	1.35	0.915	21.99	63.69	0.919	21.99	63.41
100	0.963	22.05	60.7	1.35	0.902	21.52	63.22	0.896	21.52	63.65
150	0.965	21.87	60.1	1.35	0.890	21.20	63.12	0.881	21.20	63.77
200	0.959	21.92	60.6	1.35	0.882	21.13	63.49	0.869	21.13	64.44
250	0.967	21.78	59.7	1.35	0.875	21.38	64.75	0.851	21.38	66.58
300	0.963	21.82	60.0	1.35	0.871	21.80	66.33	0.840	21.80	68.77
350	0.967	22.03	60.4	1.35	0.864	21.87	67.08	0.829	21.87	69.91

Table 5.12 Summary of the axial strain, deviator stress, mean net stress and matric suction at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)	$(u_a - u_w)_f$ (kPa)
50	4.5	261	181	82.4
100	5.0	446	323	81.5
150	6.0	494	397	81.3
200	7.0	652	526	79.1
250	8.0	670	585	78.8
300	10.0	825	713	76.0
350	12.0	924	812	81.3

Table 5.12 summarizes the axial strain, deviator stress, mean net stress and matric suction at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa. The results from Table 5.12 indicate that matric suctions at failure varied from 76.0 kPa to 82.4 kPa.

The results from the CW triaxial tests conducted under different net confining stresses but at the same initial matric suction of 150 kPa are presented in Figure 5-28. The characteristics of the stress-strain curves shown in Figure 5-28 were similar to the characteristics of the stress-strain curves of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa. The void ratio, water content and degree of saturation at each stage of the tests are presented in Table 5.13. The water content at the end of the shearing stage was about 17.5 %. The void ratio at the end of the shearing stage decreased with the increase in net confining stress.

Figure 5-28 Deviator stress versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

Table 5.13 Void ratio (e), water content (w), and degree of saturation (S) of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

State	Before saturation				After matric suction equalization			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
50	0.963	21.84	60.1	1.35	0.909	18.01	52.50	0.911	18.01	52.39
100	0.967	21.78	59.7	1.35	0.892	18.24	54.19	0.902	18.24	53.59
150	0.961	22.06	60.8	1.35	0.881	18.05	54.29	0.876	18.05	54.60
200	0.966	21.83	59.9	1.35	0.877	17.99	54.36	0.864	17.99	55.18
250	0.959	21.77	60.2	1.35	0.872	17.92	54.46	0.850	17.92	55.87
300	0.962	22.08	60.8	1.35	0.866	17.75	54.32	0.829	17.75	56.74
350	0.966	22.14	60.7	1.35	0.859	17.77	54.82	0.818	17.77	57.57

Table 5.14 Summary of the axial strain, deviator stress, mean net stress and matric suction at failure of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)	$(u_a - u_w)_f$ (kPa)
50	4.4	377	239	118.5
100	5.0	477	339	118.0
150	6.0	582	441	117.8
200	7.0	662	531	116.5
250	8.8	764	632	114.3
300	10.1	827	714	111.7
350	10.8	1046	873	108.1

The axial strain, deviator stress, mean net stress and matric suction at failure of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa are summarized in Table 5.14. The results from Table 5.14 indicate that the matric suction at failure varied from 108.1 kPa to 118.5 kPa. The deviator stress, axial strain, and mean net stress at failure increased with the increase in net confining stress.

Figure 5-29 shows the results from the CW triaxial shearing tests under different net confining stresses but at the same initial matric suction of 200 kPa. The graphs in Figure 5-29 indicate that most of the stress-strain curves showed evidence of post-peak strain softening. The characteristics of the stress-strain curves shown in Figure 5-29 were similar to the characteristics of the stress-strain curves of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa and 150 kPa. The void ratio, water content and degree of saturation at each stage of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa are presented in Table 5.15.

Figure 5-29 Deviator stress versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

Table 5.15 Void ratio (e), water content (w), and degree of saturation (S) of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

State	Before saturation				After matric suction equalization			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
50	0.965	22.10	60.7	1.35	0.906	14.41	42.15	0.913	14.41	41.83
100	0.962	21.89	60.3	1.35	0.887	14.21	42.60	0.888	14.21	42.41
150	0.968	22.00	60.2	1.35	0.876	14.28	43.20	0.863	14.28	43.85
200	0.960	22.09	60.9	1.35	0.872	14.53	44.16	0.862	14.53	44.67
250	0.962	21.76	59.9	1.35	0.870	14.87	45.29	0.858	14.87	45.93
300	0.967	21.92	60.1	1.35	0.864	15.10	46.31	0.837	15.10	47.81

Table 5.16 Summary of the axial strain, deviator stress and matric suction at failure of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)	$(u_a - u_w)_f$ (kPa)
50	4.7	425	263	160.0
100	5.5	506	359	156.1
150	6.1	622	486	154.3
200	6.8	746	573	151.6
250	8.9	849	675	148.5
300	9.0	954	777	147.2

Table 5.16 presents the summary of the axial strain, deviator stress, mean net stress and matric suction at failure from each CW triaxial test. Table 5.16 shows that the variation in matric suctions at failure of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa was small. The

matric suctions at failure varied from 147.2 kPa to 160.0 kPa.

Figure 5-30 shows the results from a series of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa. The results from Figure 5-30 show that the stress-strain curves exhibited similar characteristics to those of the CW triaxial tests under different net confining stresses but at the same initial matric suctions of 100 kPa, 150 kPa and 200 kPa. The void ratio, water content and degree of saturation at each stage of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa are presented in Table 5.17. The water content at the end of the shearing stage was about 11 %. This water content was much less than the initial water content (i.e., about 50% of the initial water content) of the compacted silt specimen. Table 5.18 summarizes the axial strain, deviator stress, mean net stress, and matric suction at failure of the CW triaxial tests.

Figure 5-30 Deviator stress versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa.

Table 5.17 Void ratio (e), water content (w), and degree of saturation (S) of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

State	Before saturation				After matric suction equalization			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
50	0.958	22.20	60.7	1.35	0.898	11.67	34.44	0.908	11.67	34.06
100	0.965	21.94	60.3	1.35	0.878	11.46	34.59	0.880	11.46	34.51
150	0.961	22.10	60.2	1.35	0.870	11.65	35.49	0.873	11.65	35.36
200	0.964	21.89	60.9	1.35	0.864	11.73	35.98	0.851	11.73	36.53
250	0.958	21.86	59.9	1.35	0.862	11.84	36.40	0.848	11.84	37.00
300	0.963	21.95	60.1	1.35	0.858	12.16	37.56	0.844	12.16	38.18

Table 5.18 Summary of the axial strain, deviator stress and matric suction at failure of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)	$(u_a - u_w)_f$ (kPa)
50	4.3	436	268	257.0
100	4.5	525	363	250.7
150	5.4	677	489	249.7
200	6.1	798	602	240.8
250	7.8	882	691	241.5
300	9.8	967	784	236.8

5.3.3 Characteristics of the Excess Pore-water Pressure

Figure 5-31 shows the change in pore-water pressure during shearing of the CW triaxial tests on the saturated specimens under different net confining stresses. As shown in Figure 5-31, the magnitude of the change in pore-water pressure increased

with the increase in net confining stress. It could be attributed to the fact that the higher the net confining stress, the higher the degree of saturation in the soil specimen would be and consequently the excess pore-water pressure during shearing would be higher. The trends of the pore-water pressure changes of the four specimens (i.e., CW50-0, CW100-0, CW150-0, CW200-0, CW250-0 and CW300-0) were similar in pattern. The change in pore-water pressure increased rapidly at the beginning of the shearing stage and then decreased subsequently. The change in pore-water pressure approached zero at the end of the shearing stage of the specimens CW50-0, CW100-0, CW150-0 and CW200-0.

Figure 5-31 Change in pore-water pressure versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of zero kPa

Figures 5.32 to 5.35 show the change in pore-water pressure during shearing of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa, 150 kPa, 200 kPa and 300 kPa, respectively. As shown in Figures 5.32 to 5.35, the change in pore-water pressure increased as shearing progressed in the triaxial test on specimens under the constant water content condition. Figures 5.32 to 5.35 indicate that the specimens with a high net confining stress exhibited a large change in pore-water pressure. The trends of the pore-water pressure changes during the shearing stage were similar. The variations of the pore-

water pressure during shearing on specimens under different net confining stresses but at the same initial matric suction of 100 kPa, 150 kPa, 200 kPa and 300 kPa were small. However, it can be seen that the magnitude of the change in pore-water pressure increased with the increase in net confining stress. In general, the pore-water pressure eventually remained essentially constant, indicating the critical state.

Figure 5-32 Change in pore-water pressure versus axial strain of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Figure 5-33 Change in pore-water pressure versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

Figure 5-34 Change in pore-water pressure versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

Figure 5-35 Change in pore-water pressure versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

The results from Figures 5.32 to 5.35 indicate that the patterns of the changes in pore-water pressure were in agreement with those data presented by Satija (1979) (as discussed in Chapter 2). Figures 5.32 to 5.35 show that specimens with a high

net confining stress exhibited a high magnitude of the change in pore-water pressure during shearing. This could be explained by the fact that the higher the net confining stresses, the lower the void ratio and the higher the degree of saturation in the soil specimen would be. As a result, the magnitude of the pore-water pressure response during shearing would be high.

Figures 5.32 to 5.35 indicate that the pore-water pressure increased rapidly at axial strains less than 3%. The pore-water pressure then increased slightly and reached nearly a constant value at the peak deviator stress. This could be attributed to the fact that during the first period (i.e., axial strains less than 3%), the soil specimens normally decreased in volume. As a result, the pore-water pressure in the soil specimen would increase quickly during the first period of shearing. The soil specimen dilated subsequently, causing the pore-water pressure to decrease. However, at the same time the axial load increased during shearing. Therefore, the pore-water pressure would increase slightly or remained essentially constant.

5.3.3.1 Characteristics of the Change in Pore-water Pressure in the NTU Mini Suction Probes and at the Base Plate

The non-uniformities in pore-water pressure in a specimen during shearing were discussed by Bishop et al. (1960); Barend and McDemott (1965) and Blight (1965). The maximum difference between pore-water pressure measurements at the mid-height and the base plate was about 30 kPa as reported by Bishop et al. (1960).

Figure 5-36 shows the change in pore-water pressure measurement from the NTU mini suction probes and the base plate during shearing under the CW triaxial test. As indicated in the Figure 5-36, the difference in the pore-water pressure measurements between the NTU mini suction probes and the base plate was quite small. The difference in the pore-water pressure measurements among the NTU mini suction probes was less than 1 kPa. The maximum difference in the pore-water pressure measurements between the NTU mini suction probes and the base plate was about 1.5 kPa. This difference is much smaller than that (i.e., 30 kPa) reported by Bishop et al. (1960). The reason for the difference could be that the compacted silt specimens used in this study has a coefficient of permeability, which

is much higher than that of compacted shale tested by Bishop et al. (1960). In addition, the strain rate used in this study (i.e., 0.009 mm/minute) is much smaller than the strain rate (i.e., 0.042 mm/minute) used by Bishop et al. (1960).

As shown in Figure 5-36, the pore-water pressure or matric suction during shearing for the CW triaxial tests in this study was uniform throughout the specimen. Therefore, the measurements of the pore-water pressure from any one of the NTU mini suction probes would represent the pore-water pressure during shearing for the entire specimen. As a result, the selection of the strain rate is an important factor in conducting the CW triaxial tests.

Figure 5-37 plots the matric suction at failure, $s_f = (u_a - u_w)_f$, versus initial matric suction, $s_i = (u_a - u_w)_i$, for the CW tests. Figure 5-37 shows that matric suction at failure decreased with the increase in net confining stress for a given initial matric suction.

The change in matric suction during shearing under the CW triaxial tests can be expressed in terms of percentage. Figure 5-38 shows the percentage of the matric suction change at failure versus initial matric suction for the CW triaxial tests. It can be seen that the trends of matric suction change in all the CW tests under different matric suctions were similar in pattern. Figure 5-38 indicates that the percentage of the matric suction changes at failure increased gradually with the increase in the initial matric suction and then decreased gradually when the initial matric suction was higher than 200 kPa. In general, the percentage of the matric suction change in all the CW tests on the compacted silt specimens ranged from 15% to 25%.

Figure 5-36 Measurements of the change in pore-water pressure in NTU mini suction probes and base plate during shearing of specimen CW150-100

Figure 5-37 Matric suction at failure versus matric suction at the initial condition for the CW triaxial tests

Figure 5-38 Percentage of matric suction changes versus initial matric suction during shearing under the CW triaxial tests

5.3.3.2 Pore-water Pressure Parameter

Fredlund and Rahardjo (1993) presented an expression for the pore-water pressure development in the specimen under triaxial loading conditions:

$$du_w = B_w d\sigma_3 + D_w d(\sigma_1 - \sigma_3) \quad (5.3)$$

where:

du_w = change in pore-water pressure,

B_w = pore-water pressure parameter,

D_w = tangent pore-water pressure parameter, and

$d(\sigma_1 - \sigma_3)$ = change in deviator stress.

In the CW triaxial test, the confining stress and pore-air pressure remain constant. Therefore, an equivalent parameter, D'_w can be defined as a ratio between $\Delta(u_a - u_w)$ and $\Delta(\sigma_1 - \sigma_3)$ by simplifying Equation (5.3) as follows:

$$D'_w = -\frac{d(u_a - u_w)}{d(\sigma_1 - \sigma_3)} \quad (5.4)$$

Typical results of the development of D'_w parameter with respect to the deviator stress from the beginning of shearing to failure for the CW triaxial tests are presented in Figure 5-39. Figure 5-39 plots the development of the D'_w parameter for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa. It can be noted that the D'_w parameter increased with the increase in net confining stress. This can be attributed to the fact that under a high net confining stress the soil specimens became denser and the degree of saturation was higher. As a result, the response of the pore-water pressure to the application of the deviator stress would also increase. It also appears that the D'_w parameter decreased with the increase in the deviator stress and remained essentially constant at failure. The D'_w parameters at the beginning of shearing were found to range from 0.10 and 0.12 for the specimens with an initial matric suction of 150 kPa.

Figure 5-39 The D'_w parameter versus deviator stress for the CW triaxial tests under the same initial matric suction of 150 kPa but at the different net confining stresses

5.3.4 Volume Change Behaviours during Shearing Stage

The triaxial tests were conducted in a controlled temperature room. The shearing was started after three to five days of matric suction equalization. Therefore, the thermal volume change of the external cell would be minimized. The volumetric strain versus axial strain from a series of the CW triaxial tests on the compacted specimens under different net confining stresses but at the same initial matric suction of 100 kPa (i.e., specimens CW25-100, CW50-100, CW100-100, CW150-100, CW200-100, CW300-100 and CW350-100) are presented in Figure 5-40. The signs for the volumetric strain are negative for compression and positive for dilation. The volumetric strain of the specimens exhibited similar characteristics. The results show that the specimens compressed in the early stage of shearing and then dilated afterwards. However, Figure 5-40 shows that specimens CW300-100 and CW350-100 underwent compression throughout the entire shearing stage. It can be seen that, the higher the net confining stress was, the higher the compression would be during shearing. Meanwhile, the lower the net confining stresses, the higher the dilation would be. The volumetric strains during shearing under the constant water content condition eventually became constant, indicating the critical state condition. Therefore, volumetric strain can be used as a one of the criteria for determining the critical state condition in the CW triaxial tests.

Figure 5-40 Volumetric strain versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Figures 5.41 to 5.43 show the volumetric strains versus axial strain from the CW triaxial tests on the compacted specimens under different net confining stresses but at the same initial matrix suction of 150 kPa, 200 kPa and 200 kPa, respectively. The characteristics of the volumetric strains from the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 150 kPa, 200 kPa and 300 kPa are the same as those data presented in Figure 5-40. The specimens CW350-150, CW300-200 and CW300-300 underwent compression throughout the entire shearing stage.

Figure 5-41 Volumetric strain versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 150 kPa

Figure 5-42 Volumetric strain versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 200 kPa

Figure 5-43 Volumetric strain versus axial strain from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

In general, the results shown in Figures 5.40 to 5.43 indicate that the specimens with a high initial matric suction showed a small compressibility and a large dilatancy. This could be attributed to the fact that the high initial matric suction in the soil specimen resulted in a “highly overconsolidated” soil. Therefore, the specimen would be stiffer and the compressibility of the soil specimen would be small. However, the specimen would exhibit a high dilatancy. As a result, the dilation of the specimen during shearing increased with the increase in initial matric suctions. Figures 5.40 to 5.43 also show that, some of the compacted silt specimens (i.e., CW350-100, CW350-150, CW300-200 and CW300-300) underwent compression throughout the entire shearing stage. It can be seen that, the higher the net confining stress was, the higher the compression would be during shearing under the same initial matric suction. This could be due to the fact that, the high net confining stress in the soil specimen caused a decrease in the overconsolidation of the soil specimen. As a result, the compressibility of the soil specimen would be high and dilatancy would be small. The volumetric strains eventually became constant, indicating the critical state condition. This could be understood that, the deviator stress, pore-water pressure remained

essentially constant at the critical state. Therefore, the volume change of the soil specimen would remain essentially constant at the critical state.

5.3.5 Water Content Characteristics of the Specimen at the End of the Shearing Stage

At the end of the shearing stage, the specimen was divided into ten layers and the water content was determined for each layer. A typical profile of the water content of specimen CW100-100 is shown in Figure 5-44. The profile in Figure 5-44 shows that the water content distribution from the top to the bottom of the specimen was fairly uniform. The water content profile indicated that matric suction was also uniform throughout the specimen at the end of the shearing stage. Therefore, the strain rate chosen for the shearing stage was reasonable for the compacted silt specimen.

Figure 5-44 Water content profile of the specimen CW100-100

5.4 Consolidated Drained (CD) Triaxial Test Results

A series of the CD triaxial tests were conducted on compacted silt specimens. The specimens were tested in the modified triaxial apparatus under net confining stresses of 100 kPa, 200 kPa and 300 kPa and at different matric suctions (i.e., initial matric suction of zero kPa, 100 kPa, 200 kPa and 300 kPa). The consolidated drained triaxial test results are presented for the same matric suction but under different net confining stresses. Similar to the CW triaxial tests, a name convention for the tested soil specimens was adopted using a designation of CDa-b (see Chapter 4). The terms a-b mean that the test was conducted under a net confining stress of a kPa and matric suction of b kPa. The details of the CD test results are discussed in the following sections.

5.4.1 Shear Strength Behaviours

The results from shearing tests under different net confining stresses but at the same matric suction of 0 kPa are presented in Figure 5-45. Figure 5-45 shows that the deviator stress remained essentially constant after reaching a maximum deviator stress. As shown in Figure 5-45, the stiffness of the soil specimens increased with the increase in net confining stress.

Figure 5-45 Deviator stress versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa

Table 5.19 Void ratio (e), water content (w), and degree of saturation (S) of the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa

State	Before saturation				After consolidation			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
100	0.967	21.85	59.91	1.35	0.960	36.09	99.58	0.961	36.07	99.48
200	0.961	21.95	60.28	1.35	0.935	34.65	98.21	0.917	33.98	98.18
300	0.963	22.00	60.54	1.35	0.922	34.61	99.46	0.875	32.84	99.43

Table 5.20 Summary of the axial strain, deviator stress and mean net stress at failure of the CD triaxial tests under different net confining stresses but at the same matric suction of 0 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)
100	5.6	235	218
200	7.5	474	437
300	8.1	714	657

The void ratio, water content and degree of saturation at each stage of the CD triaxial test are presented in Table 5.19. Table 5.20 summarizes the axial strain, deviator stress and mean net stress at failure of the CD triaxial tests conducted on saturated specimens but under different net confining stresses.

The results from shearing tests under different net confining stresses but at the same matric suction of 100 kPa are presented in Figure 5-46. Figure 5-46 shows that most of the stress-strain curves exhibited evidence of post-peak strain softening. However, the ductility of the specimens increased with the increase in the net confining stress. The lower the net confining stress was, the more distinct the peak deviator stress would be. The higher the net confining stress was, the higher the

axial strain required for the specimen to reach the maximum deviator stress. The void ratio, water content and degree of saturation at each stage of the CD triaxial tests are presented in Table 5.21. Table 5.22 summarizes the axial strain, deviator stress, and mean net stress at failure of the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa.

Figure 5-46 Deviator stress versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

Table 5.21 Void ratio, water content and degree of saturation of the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

State ($\sigma_3 - u_a$) (kPa)	Before saturation				After matric suction equalization			After shearing		
	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
100	0.969	22.02	60.2	1.35	0.900	21.31	59.85	0.916	19.35	53.47
200	0.963	22.00	60.5	1.35	0.881	20.75	59.53	0.855	18.80	55.48
300	0.958	21.70	60.0	1.35	0.867	20.81	61.36	0.843	18.74	55.98

Table 5.22 Summary of the axial strain, deviator stress and matric suction at failure of the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

$(\sigma_3 - u_a)$ (kPa)	ε_{yf} (%)	q_f (kPa)	p_f (kPa)	v_f
100	5.5	479.0	340.0	1.912
200	7.3	705.0	553.0	1.881
300	7.8	907.0	754.0	1.859

Figure 5-47 shows the deviator stress versus axial strain from shearing tests for the compacted silt specimens under different net confining stresses but at the same matric suction of 200 kPa. Figure 5-47 indicates that the characteristics of the stress-strain curves are similar as those stress-strain curves that are presented in Figure 5-46.

Figure 5-47 Deviator stress versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

Chapter 5 Presentation of Results

The void ratio, water content and degree of saturation at each stage of the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa are presented in Table 5.23. The water content at the end of the shearing stage was about 12.5 %. Table 5.24 presents the summary of the axial strain, deviator stress, and mean net stress at failure of the CD triaxial test under different net confining stresses but at the same matric suction of 200 kPa.

Table 5.23 Void ratio (e), water content (w), and degree of saturation (S) of the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

State	Before saturation				After matric suction equalization			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
100	0.961	21.85	60.2	1.35	0.889	14.09	39.80	0.918	12.89	35.86
200	0.963	21.82	60.3	1.35	0.872	13.94	41.40	0.875	12.20	36.11
300	0.965	21.85	60.8	1.35	0.860	13.77	40.88	0.839	12.02	36.52

Table 5.24 Summary of the axial strain, deviator stress and mean net stress at failure of the CD tests under different net confining stresses but at the same matric suction of 200 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)
100	5.5	557.0	379.0
200	6.0	779.0	590.0
300	7.2	997.0	799.0

The results from a series of CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa are presented in Figure 5-48. Figure 5-48 shows that the characteristics of the stress-strain curves are similar as those stress-

strain curves that are presented in Figures 5.46 and 5.47. The void ratio, water content and degree of saturation at each stage of the CD triaxial tests are presented in Table 5.25. Table 5.26 presents the axial strain, deviator stress and mean net stress at failure of the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa.

Figure 5-48 Deviator stress versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

Table 5.25 Void ratio, water content and degree of saturation of the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

State	Before saturation				After matric suction equalization			After shearing		
$(\sigma_3 - u_a)$ (kPa)	e	w (%)	S (%)	ρ_d Mg/m^3	e	w (%)	S (%)	e	w (%)	S (%)
100	0.966	21.15	60.8	1.35	0.882	10.84	31.02	0.889	9.75	27.65
200	0.963	22.00	60.5	1.35	0.867	10.68	31.37	0.871	9.47	27.73
300	0.961	21.85	60.3	1.35	0.858	10.65	31.66	0.857	9.52	28.31

Table 5.26 Summary of the axial strain, deviator stress and mean net stress at failure of the CD tests under different net confining stresses but at the same matric suction of 300 kPa

$(\sigma_3 - u_a)$ (kPa)	ϵ_{yf} (%)	q_f (kPa)	p_f (kPa)
100	4.3	585.0	393.0
200	5.2	787.0	594.0
300	7.0	1018.0	809.0

Figures 5.44 to 5.47 show the stress-strain curves for the CD triaxial tests, which were conducted under different net confining stresses but at the same matric suction of zero kPa, 100 kPa, 200 kPa and 300 kPa, respectively. It can be seen that the higher the initial matric suction was, the higher the shear strength of the soil specimen would be. This could be attributed to the fact that as the matric suction increased, the degree of saturation and void ratio would decrease. As a result, the shear strength would increase. It also appears that, the higher the net confining stress, the less distinct the peak deviator stress would be. This could be explained by the fact that a high net confining stress would cause the specimens to be more restrained in dilation during shearing. Therefore, the increase in volume change during shearing would be small. As a result, the deviator stresses decreased gradually after reaching the peak deviator stress.

5.4.2 Characteristics of the Soil Volume Changes

Figures 5.49 to 5.52 show the volumetric strain versus axial strain during shearing from four series of CD triaxial tests on the compacted specimens under different net confining stresses but at the same initial matric suctions of zero kPa, 100 kPa, 200 kPa and 300 kPa, respectively. The signs for the volumetric strain are also expressed as negative for compression and positive for dilation. The results show that the specimens compressed in the early stage of shearing and then dilated afterwards. However, the specimens CD300-0, CD300-100 and CD300-200 underwent compression throughout the entire shearing stage. It can be seen that the higher the net confining stress was, the higher the

compression would be. Meanwhile, the lower the net confining stress was, the higher the dilation would be. The volumetric strains during shearing eventually remained essentially constant, indicating the critical state condition. Therefore, the volumetric strain can be used as a criterion for determining the critical state condition for the CD triaxial tests.

Figure 5-49 Volumetric strain versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa

Figure 5-50 Volumetric strain versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

Figure 5-51 Total volumetric strain versus axial strain during shearing from the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

Figure 5-52 Total volumetric strain versus axial strain during shearing from the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

5.4.3 Water Volume Change Behaviours during Shearing Stage

Figures 5.53 to 5.55 show the water volumetric strain versus axial strain during shearing from three series of the CD triaxial tests on the compacted specimens under different net confining stresses but at the same initial matric suction of 100 kPa, 200 kPa and 300 kPa, respectively. The signs for the water volumetric strain are negative for water draining out and positive for water flowing into soil specimen during shearing. The water volumetric strains of all CD triaxial tests exhibited similar characteristics.

Figure 5-53 Volumetric strain versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

The results show that during shearing tests, the water volumetric strain increased with the increase in net confining stress. The water volumetric strain increased rapidly at the early stage of shearing and then increased slowly afterwards. The water volumetric strains during shearing eventually became constant, indicating the critical state condition. In general, the higher the net confining stress was, the larger amount of water that drained out from the specimen at the end of the shearing stage. It can be seen that, the higher the matric suction was, the smaller amount of water that would be drained out from the specimen during shearing tests. The

difference in the amount of water that drained out from the specimen during shearing under different net confining stresses but at the same matric suction was quite small.

Figure 5-54 Water volumetric strain versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

Figure 5-55 Water volumetric strain versus axial strain from the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

5.5 Interpretation of the CW and CD Triaxial Test Results using Extended Mohr-Coulomb Failure Envelope

The results of the constant water content (CW) triaxial tests and the consolidated drained (CD) triaxial tests on the compacted silt specimens at the maximum dry density and optimum water content are presented in this section.

5.5.1 Failure Criteria

Several criteria for determining failure in the CW and CD triaxial tests are discussed in this section. Figure 5-56 shows the deviator stress versus axial strain for the CW triaxial test on specimens under different net confining stresses but with the same initial matric suction of 300 kPa (i.e., CW50-300, CW100-300, CW150-300, CW200-300, CW250-300 and CW300-300). The use of peak deviator stress as a failure criterion in the CW triaxial tests is illustrated in Figure 5-56. The corresponding axial strains at failure are also presented in Figure 5-56. The six specimens exhibited a distinct peak deviator stress with a post-peak strain softening behaviour. The peak deviator stresses of specimens CW50-300, CW200-300 and CW300-300 were reached at axial strains of 4.3%, 6.1% and 9.8%, respectively. A summary of the axial strains at failure is presented in Table 5.27.

Figure 5-57 shows the principal stress ratio, $((\sigma_1 - \sigma_3)/(\sigma_3 - u_w))$ versus axial strain for the CW triaxial tests on specimens under different net confining stresses but at the same initial matric suction of 300 kPa. Figure 5-57 illustrates the use of principal stress ratio as a failure criterion for the CW triaxial tests. Figure 5-57 was plotted by normalizing the deviator stress, $(\sigma_1 - \sigma_3)$, with respect to the stress state variable, $(\sigma_3 - u_w)$, in which σ_3 and u_w are the cell pressure and the pore-water pressure, respectively. The six specimens exhibited a distinct peak deviator stress with a post-peak softening behaviour. The axial strains at failure for specimens CW50-300, CW200-300 and CW300-300 were 4.3%, 6.2% and 9.9%, respectively (Figure 5-57). The axial strains at failure based on the principal stress ratio, $((\sigma_1 - \sigma_3)/(\sigma_3 - u_w))$, are presented in Table 5.27. The magnitudes of the strain at

failure obtained from the principal stress ratio approach (i.e., Figure 5-57) were in agreement with the strain at failure as obtained from the peak deviator stress approach (Figure 5-56 and Table 5.27).

Figure 5-56 Peak deviator stress as a failure criterion for the constant water content tests under the same matric suction of 300 kPa on specimens with different net confining stresses

Table 5.27 Summary of the axial strains at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

Specimen	Axial strain at failure, ε_{yf} , (%) based on			
	Maximum deviator stress	Stress ratio, $\frac{(\sigma_1 - \sigma_3)}{(\sigma_3 - u_w)}$	Stress ratio, $\frac{(\sigma_1 - u_w)}{(\sigma_3 - u_w)}$	Stress ratio, $\frac{(\sigma_1 - u_a)}{(\sigma_3 - u_a)}$
CW50-300	4.3	4.3	4.3	4.3
CW100-300	4.5	4.5	4.5	4.5
CW150-300	5.4	5.5	5.5	5.4
CW200-300	6.1	6.2	6.2	6.1
CW250-300	7.8	8.0	7.9	7.8
CW300-300	9.8	9.9	9.9	9.8

Figure 5-57 Principal stress ratio, $(\sigma_1 - \sigma_3)/(\sigma_3 - u_w)$ versus axial strain for the constant water content tests on specimens under different net confining stresses but at the same initial matric suction of 300 kPa

Figure 5-58 shows another form of principal stress ratio, $(\sigma_1 - u_w)/(\sigma_3 - u_w)$, versus axial strain for the CW triaxial tests on specimens under different net confining stresses but with the same initial matric suction of 300 kPa. This form of principal stress ratio could also be used as a failure criterion for the constant water content triaxial tests. The axial strains at failure for specimens CW50-300, CW200-300 and CW300-300 were 4.3%, 6.2% and 9.9%, respectively. The axial strains at failure obtained from the principal stress ratio, $(\sigma_1 - u_w)/(\sigma_3 - u_w)$, were similar as the strain at failure as obtained from the principal stress ratio, $(\sigma_1 - \sigma_3)/(\sigma_3 - u_w)$. Therefore, both forms of principal stress ratios (i.e., $(\sigma_1 - \sigma_3)/(\sigma_3 - u_w)$ and $(\sigma_1 - u_w)/(\sigma_3 - u_w)$) can also be used as a failure criterion for the constant water content triaxial tests.

Figure 5-58 Principal stress ratio, $(\sigma_1 - u_w)/(\sigma_3 - u_w)$ versus axial strain for the constant water content test on specimens under different net confining stresses but at the same initial matric suction of 300 kPa

Figure 5-59 shows another form of principal stress ratio, $(\sigma_1 - u_a)/(\sigma_3 - u_a)$, versus axial strains for the CW triaxial tests on specimens under different net confining stresses but with the same initial matric suction of 300 kPa. This principal stress ratio could also be used as a failure criterion for the constant water content tests. The axial strains at failure for specimens CW50-300, CW200-300 and CW300-300 were 4.3%, 6.1% and 9.8%, respectively. The axial strains at failure obtained from the maximum principal stress ratio, $(\sigma_1 - u_a)/(\sigma_3 - u_a)$, were the same as the strains at failure obtained from the maximum deviator stress approach. The axial strains at failure obtained from Figures 5.56 and 5.59 were in good agreement with the axial strains at failure obtained from Figures 5.57 and 5.58.

In general, the strain softening is evident in overconsolidated soil specimens. The soil specimens compressed during the early stage of shearing. As a result, the shear strength would increase. It was then followed by dilation that resulted in a decrease

in shear strength and strain softening would occur. There were shear bands in the most of soil specimens at the final stage of shearing. Figure 5-60 shows the photos of the specimens with a shear band for the CW200-300 and CD100-200 triaxial tests.

Figure 5-59 Principal stress ratio, $(\sigma_1 - u_a)/(\sigma_3 - u_a)$, versus axial strain for the CW triaxial tests on specimens under different net confining stresses but at the same initial matric suction of 300 kPa

Figure 5-60 Specimens after the CW and CD triaxial tests

5.5.2 Constant Water Content (CW) Triaxial Tests

5.5.2.1 Characteristic of the Stress Paths of the CW Triaxial Tests

It is possible to plot the stress paths of the specimens during shearing under the constant water content condition on the $(q - s)$ plane. Figures 5.60 to 5.65 show the stress paths for the CW triaxial tests under different initial matrix suctions (i.e., matrix suctions of zero kPa, 100 kPa, 150 kPa, 200 kPa and 300 kPa) but at the same net confining stresses of 50 kPa, 100 kPa, 150 kPa, 200 kPa, 250 kPa and 300 kPa, respectively. The results indicate that matrix suction decreased with the increase in deviator stress. It can be seen that matrix suction decreased throughout the CW triaxial shearing tests. In general, the trend of the stress paths on the $(q - s)$ plane during the shearing stage was similar. The deviator stresses dropped after reaching the peak deviator stress towards the direction of decreasing matrix suctions (i.e., to the left of the graph).

Figure 5-61 Stress paths on the $(q - s)$ plane for the CW triaxial tests under different initial matrix suctions but at the same net confining stress of 50 kPa

Figure 5-62 Stress paths on the $(q - s)$ plane for the CW triaxial tests under different initial matric suctions but at the same net confining stress of 100 kPa

Figure 5-63 Stress paths on the $(q - s)$ plane for the CW triaxial tests under different initial matric suctions but at the same net confining stress of 150 kPa

Figure 5-64 Stress paths on the $(q - s)$ plane for the CW triaxial tests under different initial matric suctions but at the same net confining stress of 200 kPa

Figure 5-65 Stress paths on the $(q - s)$ plane for the CW triaxial tests under different initial matric suctions but at the same net confining stress of 250 kPa

Figure 5-66 Stress paths on the $(q - s)$ plane for the CW triaxial tests under different initial matric suctions but at the same net confining stress of 300 kPa

5.5.2.2 Mohr-Coulomb Failure Envelope for the CW triaxial Tests

The failure envelopes of the compacted silt specimens from the CW triaxial tests are presented in this section. Two types of failure envelopes are used to present the tests results: i) the extended Mohr-Coulomb failure envelopes and ii) the stress point envelopes. The shear stresses at failure of the specimens used in the construction of Mohr circles for the CW triaxial tests were determined using the peak deviator stresses from the stress-strain curves. Meanwhile, the stresses at failure used in the construction of the stress point failure envelope for the CW triaxial tests were determined by taking the peak deviator stresses from the stress paths.

Figure 5-67 gives the Mohr-Coulomb failure envelope for the saturated specimens under different net confining stresses. An effective angle of internal friction, ϕ' , of 32° and effective an cohesion, c' , of 0 kPa can be obtained from Figure 5-67.

Figure 5-67 Extended Mohr – Coulomb failure envelope for the CW triaxial tests under different net confining stresses but at zero matric suction

The extended Mohr – Coulomb failure envelopes can be constructed for the CW triaxial tests under different matric suction planes but at the same net confining stress by assuming that $\phi' = 32^\circ$ (i.e., equal to the effective angle of internal friction for saturated specimens) is applicable to all matric suction planes. This assumption is in agreement with the constant M parameter that will be shown later in Chapter 6. The extended Mohr - Coulomb failure envelopes for the CW triaxial tests under different initial matric suctions but at the same net confining stress of 50 kPa, 100 kPa, 150 kPa, 200 kPa, 250 kPa and 300 kPa are presented in Figures 5.67 to 5.72, respectively. The cohesion intercepts, c , from the CW triaxial tests were determined by drawing the Mohr-Coulomb failure envelopes tangentially over the Mohr circles developed from each CW triaxial tests.

Figure 5-68 Mohr circle and cohesion intercepts at the peak deviator stresses in the CW triaxial tests under different matrix suctions but at the same net confining stress of 50 kPa

Figure 5-69 Mohr circle and cohesion intercepts at the peak deviator stresses in the CW triaxial tests under different matrix suctions but at the same net confining stress of 100 kPa

Figure 5-70 Mohr circles and cohesion intercepts for the compacted silt specimens at the peak deviator stresses in the CW triaxial tests under different matric suctions but at the same net confining of 150 kPa

Figure 5-71 Mohr circles and cohesion intercepts for the compacted silt specimens at the peak deviator stresses in the CW triaxial tests under different matric suctions but at the same net confining of 200 kPa

Figure 5-72 Mohr circles and cohesion intercepts at the peak deviator stresses in the CW triaxial tests under different matric suctions but at the same net confining of 250 kPa

Figure 5-73 Mohr circles and cohesion intercepts at the peak deviator stresses in the CW triaxial tests under different matric suctions but at the same net confining of 300 kPa

5.5.2.3 Stress point Failure Envelopes for the CW triaxial Tests

Figure 5-74 shows the stress paths on the $(q - p)$ plane for the CW triaxial tests under different net confining stresses but at the saturated condition. The results in Figure 5-74 indicate that the stress paths from different net confining stresses approached a single failure line (i.e., k_f line). The maximum deviator stress from the stress path could be used as a failure criterion for the CW triaxial tests (Rahardjo et al. 2004). The advantage of this stress path approach is that it is possible to trace the position of the stress paths of the specimens with respect to the failure surface of the extended Mohr-Coulomb envelopes in the CW triaxial tests. Therefore, it is possible to use the stress paths on the $(q - p)$ plane of the specimens to determine if the stress paths would eventually reach the stress point envelope. The stress point envelope can be related to the extended Mohr-Coulomb failure envelope (Fredlund and Rahardjo, 1993).

Figure 5-74 Stress paths from the CW triaxial tests under different net confining stresses on specimens but at the initial matric suction of zero kPa

The stress paths can be used to determine the stress point failure envelope, slope angle of the stress point envelope (i.e., ψ') and ordinate intercept of the stress point

envelope (i.e., d) for the CW tests. The stresses at failure, $\left(\frac{\sigma_{1f} - \sigma_{3f}}{2}\right)$ and $\left(\frac{\sigma_{1f} + \sigma_{3f}}{2} - u_a\right)$ for the specimens used in the construction of the stress point failure envelope for the CW tests were determined by taking the peak deviator stresses in the stress paths. Figure 5-75 plots the stress point envelopes from the CW triaxial tests under different net confining stresses and at different initial matric suctions.

The best-fit line drawn through all the maximum deviator stresses give an average value of ψ' of 28° . The stress point envelope can be related to the extended Mohr-Coulomb failure envelope (Fredlund and Rahardjo, 1993) as follows:

$$\tan \psi' = \sin \phi' \quad (5.5)$$

$$\phi' = \sin^{-1}(\tan \psi') \quad (5.6)$$

$$d = c \cos \phi' \quad (5.7)$$

Substituting ψ' of 28° into Equation (5.6) gives:

$$\phi' = \sin^{-1}(\tan 28) = 32^\circ$$

Figure 5-75 Stress point failure envelopes for the CW tests at different initial matric suctions

The values of the effective angle of internal friction obtained from the stress point envelopes and from the extended Mohr-Coulomb failure envelopes are identical. Similarly, cohesion intercept, c , can be calculated from the intercept of the stress point envelopes, d using Equation (5.7).

Figure 5-76 shows the projection of the shear stress at failure, τ_f , on the $(\tau - (u_a - u_w))$ plane for the CW triaxial tests. The shapes of the projected curves on the $(\tau - (u_a - u_w))$ plane for the CW tests under different net confining stresses were essentially nonlinear. Figure 5-76 indicates that the angle of the shear strength envelope with respect to matric suction had an average value, ϕ^b of 32° (i.e., equal to ϕ' of 32°) at low matric suctions when the soil remained saturated. The ϕ^b angle began to decrease significantly at matric suction values between 55 kPa and 75 kPa and reached an essentially constant value of 12° when matric suction exceeded 150 kPa. Figure 5-77 shows the nonlinearity of ϕ^b angle with respect to matric suction for the compacted silt as obtained from the CW triaxial tests under different net confining

stresses. Nonlinearity of ϕ^b angle with respect to matric suction was in agreement with data reported by Satija (1978), Fredlund et al. (1987) and Gan et al. (1988). The results in Figure 5-77 indicate that the matric suction at the inflection point (i.e., a point where ϕ^b angle started to decrease from an initial value of ϕ') increased with the increase in net confining stress. This can be attributed the fact that when the net confining stress increased, the void ratio would decrease and the air-entry would increase. In other words, the higher the net confining stress was, the higher the maximum matric suction below which the soil specimen would still remain saturated.

Figure 5-76 Intersection line between the failure envelope and the τ_f versus matric suction plane

Figure 5-77 Nonlinearity relationship between ϕ^b and matric suction of the CW triaxial tests

5.5.3 Consolidated Drained Triaxial (CD) Tests

A series of the CD triaxial tests was also conducted on the compacted silt specimens at the maximum dry density, $\rho_{d\max} = 1.35 \text{ Mg/m}^3$ and optimum water content, $w_{opt} = 22\%$. The specimens were tested under the net confining stresses of 100 kPa, 200 kPa and 300 kPa and at different matric suctions (i.e., matric suctions of zero kPa, 100 kPa, 200 kPa and 300 kPa). This section presents the shear strength and volume changes during shearing under the consolidated drained condition.

5.5.3.1 Mohr – Coulomb Failure Envelope for the CD Triaxial Tests

The failure envelopes of the compacted silt specimens from the CD triaxial tests are presented in this section. Figure 5-78 gives the extended Mohr-Coulomb failure envelope for the saturated specimens but at different net confining stresses. An effective angle of internal friction, ϕ' , of 32° and an effective cohesion, c' , of 0 kPa can be obtained from Figure 5-78. The effective angle of internal friction and effective cohesion that were obtained from the CD triaxial tests appears to be the same as those obtained from the CW triaxial tests under the saturated condition.

Figure 5-78 Extended Mohr – Coulomb failure envelope for the CD triaxial tests at zero matric suction

The extended Mohr-Coulomb failure envelopes for the CD triaxial tests under different net confining stresses but at the same matric suctions of 100 kPa, 200 kPa and 300 kPa are presented in Figures 5.77 to 5.79, respectively. Each of the Mohr-Coulomb failure envelopes is plotted based on three Mohr circles from three CD triaxial tests at a constant matric suction. The stresses at failure used in the construction of the Mohr circles at failure for the CD triaxial tests were also determined by taking the peak deviator stresses from stress-strain curves. The cohesion intercepts from the CD triaxial tests on the compacted silt specimens are also shown in Figures 5.77 to 5.79.

Figure 5-79 Mohr circle and cohesion intercepts at the peak deviator stresses in the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

Figure 5-80 Mohr circle and cohesion intercepts at the peak deviator stresses in the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

Figure 5-81 Mohr circle and cohesion intercepts at the peak deviator stresses in the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

Figure 5-82 Intersection line of the extended Mohr – Coulomb failure envelope on the shear strength versus matric suction plane at zero net confining stress

The effective cohesion intercepts of the failure envelopes on the zero net confining stress ($(\sigma_3 - u_a) = 0$) plane can be joined together to give a nonlinear failure envelope with respect to matric suction as shown in Figure 5-82. Figure 5-82

shows that the angle of the shear strength envelope with respect to matric suction had a value, ϕ^b of 32° (i.e., equal to ϕ' of 32°) at low matric suctions when the soil remained saturated. The ϕ^b angle began to decrease at matric suction value greater than the air-entry value (i.e., 75 kPa). The ϕ^b angle remained essentially constant at 12° for matric suctions greater than 200 kPa.

5.5.3.2 Stress Point Failure Envelopes for the CD Triaxial Tests

Figure 5-83 shows the stress point failure envelopes for all the CD triaxial tests under different net confining stresses and at different matric suctions. The shear stresses at failure used in the construction of the stress point failure envelope for the CD triaxial tests were based on the same method as in the CW triaxial tests. The slope of the best fit line drawn through all the points gives an average angle, ψ' of 28° . Substituting ψ' of 28° into Equation (5.6) gives:

$$\phi' = \sin^{-1}(\tan 28^\circ) = 32^\circ$$

The ψ' angle of 28° agrees with the effective angle of internal friction, ϕ' , of 32° obtained from the Mohr-Coulomb failure envelopes. The values of the effective angle of internal friction obtained from the stress point envelopes and the Mohr-Coulomb envelopes are identical.

The cohesion intercept, c , can be calculated from intercept of the stress point envelopes, d using Equation (5.7). The cohesion intercepts that were obtained from the Mohr-Coulomb failure envelopes and the stress point failure envelopes for the CD triaxial tests are presented in Table 5.28.

Figure 5-83 Stress point envelopes for the compacted silt from the CD triaxial tests at different matric suctions

Table 5.28 Cohesion intercepts from the Mohr–Coulomb failure envelopes and stress point envelopes

Matric suction kPa	c from Mohr-Coulomb envelopes, kPa	d from stress points envelopes, kPa	c from stress point envelopes, kPa
0	0	0	0
100	62.0	53.0	62.0
200	82.0	69.0	82.0
300	88.0	75.0	88.0

Table 5.28 shows that both cohesion intercepts that were obtained from the Mohr–Coulomb envelopes and from the stress point failure envelopes are identical.

5.5.4 Comparisons of the Shear Strength for the CW and CD Triaxial Tests

The values of the shear strength parameters (i.e., ϕ' , ϕ^b and c) obtained from the CW triaxial tests are compared with the shear strength parameters that were obtained from the CD triaxial tests. Figure 5-84 shows the cohesion intercepts of the failure envelopes on the zero net confining stress ($(\sigma_3 - u_a) = 0$) plane for the CD and CW triaxial tests. Figure 5-84 indicates a similar pattern of the increase in the cohesion intercept with an increase in the matric suction for the both CD and CW triaxial tests. The effective cohesion intercepts, c' of zero kPa were obtained from both the CD and CW triaxial tests under saturated condition. Figure 5-85 shows the nonlinear relationship between ϕ^b and matric suction for the CW and CD triaxial tests. It is interesting to observe that the cohesion intercepts from the CW triaxial tests fell closely to the cohesion intercept line that was obtained from the CD triaxial tests. It is indicated that uniqueness of the failure surfaces exists for the compacted silt specimens tested in the CD and CW triaxial tests.

Rahardjo et al. (2004) explained that the degree of saturation of the specimen reflects the area of the pore-water in contact with the soil particles, which is responsible for the increase in the shear strength of the soils. It is noted that different degrees of saturation can exist in an unsaturated soil when subjected to the same matric suction. Therefore, the soil specimen would remain saturated and the cohesion intercept is the same for both CW and CD tests at matric suctions less than 50 kPa (see Figure 5-85). However, for the matric suction range from 50 kPa to 200 kPa, the cohesion intercepts obtained from the CW triaxial tests were slightly smaller than the cohesion intercepts obtained from the CD triaxial tests. The deviation of the cohesion intercepts of the CW triaxial tests from the line of cohesion intercept from the CD triaxial tests can be explained by the differing degrees of saturation attained by the specimens the end of shearing in the CD and CW triaxial tests. This difference in the cohesion intercept at failure under the same matric suction is similar to hysteresis in the shear strength behaviour of unsaturated soils under wetting and drying. Han et al. (1995) reported that an unsaturated soil at

drying exhibited a higher shear strength than the shear strength of the same soil at wetting, even though the stress conditions during shearing were exactly the same for both states (i.e., drying and wetting). The difference in shear strength could be contributed to the fact that the degree of saturation of the soil on the drying path (i.e., as the CD triaxial tests) was higher than the degree of saturation of the soil on the wetting path (i.e., as the CW triaxial tests). As a result, the cohesion intercept from the CD triaxial tests was higher than the cohesion intercept of the CW triaxial tests (see Figure 5-85). Meanwhile, the ϕ^b angle remained essentially constant at 12° for both CW and CD triaxial tests for matric suctions greater than 200 kPa. This could be explained by the fact that at the high matric suction range (i.e., matric suction higher than residual matric suction) (see Figure 5-85) hysteresis would become negligible. As a result, the cohesion intercept seems to converge to the same value for both the CW and CD tests.

Figure 5-84 Cohesion intercepts of the failure envelopes on the zero net confining stress ($(\sigma_3 - u_a) = 0$) plane for the CD and CW triaxial tests

(b)

(b)

Figure 5-85 Relationship between ϕ^b and matrix suction for the CW and CD triaxial tests. (a) Nonlinear relationship between ϕ^b and matrix suction; (b) Air – entry value and residual matrix suction of the compacted silt specimen

CHAPTER 6 DISCUSSION OF THE RESULTS

6.1 Introduction

This chapter presents a discussion of the results obtained from this study as given in the previous chapter (i.e., Chapter 5). First, the experimental results from SWCC and isotropic consolidation tests are discussed. In the subsequent section, the simulations results of the CW and CD triaxial tests using the elasto-plastic model with the incorporation of SWCC equations as proposed in Chapter 3 are presented and discussed.

6.2 Soil-Water Characteristics Curve

The experimental results on SWCC and isotropic consolidation tests are discussed in this section. The effects of the initial dry density and water content on SWCC and the characteristics of the isotropic consolidation curves are also discussed.

6.2.1 SWCC of the Compacted Silt Specimen at a Maximum Dry Density and an Optimum Water Content

6.2.1.1 Matric Suction Changes

The changes in matric suction during drying and wetting stages are shown in Figures 5.3 and 5.4, respectively. The results in Figures 5.3 and 5.4 illustrate the transient process of the water flow along the unsaturated soil specimen during the drying and wetting stages. In general, the pore-water pressure measurements at the bottom of the NTU mini suction probe decreased earlier than the pore-water pressure measurements at the middle and top of the NTU mini suction probes

during the drying stage. It could be attributed to the fact that during the drying stage water from the bottom of the specimen first flowed into the water compartment. As a result, the degree of saturation at the bottom of the specimen was lower than the degree of saturation at the middle and top of the specimen. Therefore, the pore-water pressure at the bottom of the specimen would be lower than those at the middle and top of the specimen. At the end of each equalization stage, the water content along the soil specimen would come to equilibrium. As a result, the pore-water pressure along the soil specimen would come to equalisation and approached the pore-water pressure at the base plate.

Meanwhile, the pore-water pressure measurements at the bottom NTU mini suction probe increased earlier than the pore-water pressure measurements at the middle and top of the NTU mini suction probes during the wetting stage. During the wetting process, the pore-water pressure at the water compartment was higher than the pore-water pressure in the soil specimen. As a result, water in the compartment flowed into the soil specimen through the bottom of the specimen first and then flowed slowly into the middle and then to the top of the specimen. The pore-water pressure in the soil specimen would increase or matric suction in the soil specimen would decrease. In this case, the pore-water pressure at the bottom of the specimen increased first, which was the flowed by the one at the middle and then the one at the top. When the water content or pore-water pressure along the soil specimen came to equilibrium, the water pressure in the soil specimen approached the pore-water pressure at the base plate.

6.2.1.2 Air-entry Value (AEV) and Yield Suction

The test results shown in Figures 5.5 to 5.12 indicate that the air-entry value increased with the increase in net confining stresses. This could be attributed to the fact that the high net confining stress in the soil specimen resulted in a low void ratio of the specimen. As a result, the air-entry value of the soil specimen would increase. It also appeared that the AEV of the SWCC that was obtained from the pressure plate was smaller than the AEV of the SWCC that was obtained from the modified triaxial tests (see Figure 5.2 and Figures 5.5 to 5.12). This could be due to

the fact that the SWCC from the pressure plate was obtained under a zero net confining stress. However, the SWCC tests in the modified triaxial apparatus were conducted under a net confining stress greater than 10 kPa. As a result, the void ratio of the soil specimen in the pressure plate would be higher than the void ratio of the soil specimen in the modified triaxial apparatus. Therefore, the AEVs of the soil specimens in the triaxial apparatus would be higher than the AEV of the specimen from the pressure plate.

Figure 5.13 presents the specific volume (i.e., $v = 1 + e$) versus matric suction of the SWCC tests under different net confining stresses on the compacted silt specimens. The results in Figure 5.13 indicate that the yield point (i.e., yield suction) increased with the increase in net confining stresses. This could be explained by the fact that the high net confining stress in the soil specimen resulted in a high density. As a result, the air-entry value and yield suction would increase with the increase in net confining stress. This result agrees with the assumption proposed by Robles and Elorza (2002); Tang and Graham (2002). The air-entry and the yield suction values of the compacted silt specimens at different net confining stresses are presented in Figure 6.1.

Figure 6.1 Air-entry value and yield suction from soil-water characteristic curves for different net confining stresses

6.3 Isotropic Consolidation Tests

6.3.1 Effect of Matric Suction on the Isotropic Consolidation Curves

The results from the isotropic consolidation tests that were conducted under different matric suctions (i.e., $(u_a - u_w) = 0$ kPa; $(u_a - u_w) = 50$ kPa, $(u_a - u_w) = 100$ kPa, $(u_a - u_w) = 150$ kPa, $(u_a - u_w) = 200$ kPa, and $(u_a - u_w) = 300$ kPa) and different initial dry densities and water content are presented in Figures 5.18 to 5.24 and Tables 5.6 to 5.8. The results in Figure 5.18, Figure 5.23 and Figure 5.24 and Tables 5.6 to 5.8 show that the slopes of the normal compression curve of specimens at the same initial dry density and initial water content decreased with the increase in matric suction. This could be attributed to the fact that the higher the matric suction, the stiffer of the soil specimen would be or in other words, the compressibility of the soil specimen would decrease. The results from the isotropic consolidation tests also indicate that the specific volume of the normal compression curve at the reference stress decreased with the increase in matric suction. This could be explained by the fact that the soil specimen would have a lower void ratio or a smaller total volume when matric suction increased.

Alonso et al. (1990) assumed that the slope of the unloading line of the isotropic consolidation curves was independent of matric suction. However, the results from this study show that the slopes of the unloading lines decreased with the increase in matric suction, and this is in agreement with the experimental results reported by Rampino et al. (2000). Tables 5.6 to 5.8 indicate that the yield stresses increased with the increase in matric suction. This result was in agreement with the results reported by Alonso et al. (1990) and Rampino et al. (2000). It could be attributed to the fact that when matric suction increased, the specimen would have a lower void ratio and a stiffer soil structure, causing the yield stress to increase.

6.3.2 Effect of the Dry Densities on the Isotropic Consolidation Curves

Three series of the isotropic consolidation curves for the compacted silt specimens at different initial dry densities and water contents (ie., $\rho_{d\max} = 1.35 \text{ Mg/m}^3$ and

$w_{opt} = 22\%$; $\rho_{dw} = 1.25 \text{ Mg/m}^3$ and $w_w = 36\%$; $\rho_{dd} = 1.30 \text{ Mg/m}^3$ and $w_d = 13\%$; are presented in Figure 5.18, Figure 5.23 and Figure 5.24, respectively. Figure 6.2 shows the slopes of the normal compression lines at different matric suctions for the compacted silt specimens of different initial dry densities. Figure 6.2 indicates that the slope of the normal compression line increased with the decrease in dry density. The lower the dry density was, the higher the slope of the normal compression line would be. This could be attributed to the fact that the higher the initial dry density, the stiffer the soil specimen or the lower the compressibility of the soil specimen would be.

Figure 6.2 The slopes of the normal compression lines with respect to matric suction for the compacted silt specimens at different initial dry densities and water contents

Figure 6.3 presents the slopes of the unloading lines for the isotropic consolidation curves. The results shown in Figure 6.3 indicate that the slopes of the unloading lines increased with the increase in dry density. The lower the dry density was, the lower the slope of the unloading line would be. This could be due to the fact that the lower the initial dry density, the softer the soil specimen would be. As a result, the slope of the unloading would decrease.

Figure 6.3 The slopes of the unloading lines with respect to matric suction for the compacted silt specimens at different initial dry densities and water contents

Figure 6.4 The yield stresses of the isotropic consolidation curves with respect to matric suction for the compacted silt specimens at different initial dry densities and water contents

Figure 6.4 shows the yield stresses for the isotropic consolidation curves on specimens of different initial dry densities. Figure 6.4 indicates that the yield stress increased with the increase in initial dry density. This could be attributed to the fact that when the initial dry density increased, the specimen would have a lower void ratio, became stiffer and as a result, the yield stress of the soil specimen would increase. Figure 6.5 shows the experimental result of the loading – collapse (LC) and suction increase yield curves for the compacted silt specimens at the maximum dry density and optimum water content.

Figure 6.5 Experimental results of the LC and SI yield curves in the $(s - p)$ plane for compacted silt at the maximum dry density and optimum water content

6.4 Combination of the Yield Curves in the $(s - p)$ Plane

Numerous researchers have studied the state boundaries of the elasto-plastic model for unsaturated soils on the $(s - p)$ plane (Alonso et al., 1990; Ho et al., 1992; Robles and Elorza, 2002 and Tang and Graham, 2002). However the effects of the mean net stress on the yield suction have not been tested. Figure 6.6 shows the combination of the loading - collapse (LC) and suction increase (SI) yield curves on the $(s - p)$ plane for compacted silt as obtained from this study.

Figure 6.6 Loading - collapse (LC) and suction increase (SI) yield curves on the $(s - p)$ plane

Alonso and et al. (1990) proposed that the SI yield curve does not depend on the mean net stress. In other words, the SI yield curve is a straight line and perpendicular to the matric suction axis. Robles and Elorza (2002) assumed that LC and SI yield curves increased linearly with the increase in matric suction and mean net stress, respectively. Tang and Graham (2002) stated that the LC and SI yield curves should be coupled. However, the experimental results from this study indicate that the SI yield curve increased nonlinearly with the increase in mean net stress. This could be explained by the fact that when the stress state variables (i.e., $(\sigma - u_a)$ and $(u_a - u_w)$) in the soil increased, the soil would normally decrease in void ratio. As a result, the stiffness and the air-entry values of the soil would increase.

6.5 Critical State Condition of the CW and CD Triaxial Tests

The critical state of a specimen is commonly defined as a condition of an essentially constant shear strength, volume and water content when subjected to triaxial loading under different test paths. This section presents the critical state on the $(q - p)$ and $(v - p)$ planes for the CW and CD triaxial tests under different net confining stresses and different matric suctions.

6.5.1 Critical State on $(q - p)$ plane

6.5.1.1 Critical State on $(q - p)$ Plane of the CW Triaxial Tests

Figure 6.7 presents the deviator stress versus the mean net stress at the critical states for the compacted silt specimens during shearing of the CW triaxial tests under a saturated condition. As shown in the Figure 6.7 the critical state points of the specimens fall on a single straight line. The slope of the critical state line at the saturated condition (i.e, $M(0)$) on the $(q - p)$ plane was 1.28.

Figure 6.7 Critical state on the $(q - p)$ plane from the CW and CD triaxial tests under different net confining stresses on the saturated specimens

Figures 6.8 to 6.11 show the critical state lines on the $(q - p)$ plane for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa, 150 kPa, 200 kPa and 300 kPa, respectively. The variations in matric suctions at the critical state were smaller for the CW triaxial tests under different net confining stresses but at the same initial matric suction (see Section 5.3 in Chapter 5). The average matric suction at the critical state of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa, 150 kPa, 200 kPa and 300 kPa were about 80 kPa, 115 kPa, 150 kPa and 245 kPa, respectively, as shown in Figures 6.8 to 6.11. It can be seen that all the critical states lines of the CW triaxial tests are straight lines with a slope of 1.28. Table 6.1 presents the stress states at the critical state for the CW triaxial tests.

Figure 6.8 Critical state on the $(q - p)$ plane from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Figure 6.9 Critical state on the $(q - p)$ plane from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

Figure 6.10 Critical state on the $(q - p)$ plane from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

Figure 6.11 Critical state on the $(q - p)$ plane from the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

Table 6.1 Stresses at the critical state of the CW triaxial tests

Net confining stress (kPa)	Initial matric suction (kPa)									
	0		100		150		200		300	
	p	q	p	q	p	q	p	q	p	q
50	94	132	105	165	121	212	144	282	138	264
100	178	235	209	327	224	371	222	367	226	378
150	267	351	290	421	312	486	315	495	318	504
200	352	457	382	545	398	595	405	615	409	628
250	413	532	458	625	477	682	489	718	503	760
300	437	564	556	767	565	795	571	814	572	817

Figure 6.12 shows the summary of the critical states of the CW triaxial tests under different net confining stresses but at different matric suctions (i.e., zero kPa, 100 kPa, 150 kPa, 200 kPa and 300 kPa) on the $(q - p)$ plane. Figure 6.13 shows the summary of the critical states of the CW triaxial tests under different net confining stresses but at different matric suctions on the $(q - s - p)$ space. Figure 6.12 indicates that the critical state lines at different matric suctions were parallel on the $(q - p)$ plane. The slopes of the critical state lines for the CW triaxial tests have the same value of 1.28.

Figure 6.12 Critical state lines in the $(q - p)$ plane of the CW triaxial tests

Figure 6.13 Critical state lines in the $(q - s - p)$ space of the CW triaxial tests

6.5.1.2 Critical State on $(q - p)$ Plane of the CD Triaxial Tests

Figures 6.14 to 6.17 show the critical state lines on the $(q - p)$ plane for the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa, 100 kPa, 200 kPa and 300 kPa, respectively. As shown in Figures 6.14 to 6.17, the critical states of the four series of the CD triaxial tests under different net confining stresses but at the same matric suction also fall on a straight line on the $(q - p)$ plane. The slopes of the critical state lines on the $(q - p)$ plane for the CD triaxial tests appear to have the same value as those obtained from the CW triaxial tests (i.e., $M = 1.28$).

Figure 6.14 Critical state on the $(q - p)$ plane from the CD triaxial tests under different net confining stresses on the saturated specimens

Figure 6.15 Critical state on the $(q - p)$ plane from the CD triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Figure 6.16 Critical state on the $(q - p)$ plane from the CD triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

Figure 6.17 Critical state on the $(q - p)$ plane from the CD triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

Table 6.2 Stresses at the critical state of the CD triaxial tests

Net confining stress (kPa)	Matric suction (kPa)							
	0		100		200		300	
	p	q	p	q	p	q	p	q
100	175	225	215	345	225	375	228	385
200	344	432	387	562	399	596	403	609
300	512	635	565	796	572	815	578	835

Table 6.2 presents the stress states at the critical state of the CD triaxial tests. Figure 6.18 shows the summary of the critical states of the CD triaxial tests under different net confining stresses and different matric suctions (i.e., 0 kPa, 100 kPa, 200 kPa and 300 kPa) on the $(q - p)$ plane. Meanwhile, Figure 6.19 shows the summary of the critical states of the CD triaxial tests on the $(q - s - p)$ space. Figures 6.18 and 6.19 indicate that the critical state lines at different matric suctions were parallel on the $(q - p)$ plane. The slopes of the critical state lines for the CD triaxial tests have the same value of 1.28. In other words, the slopes of the critical state lines on the $(q - p)$ plane were unique for the compacted silt as obtained from both the CW and CD triaxial tests.

Figure 6.20 shows the tensile strength due to matric suction from the CW and CD triaxial tests on the compacted silt specimens under different net confining stresses and at different matric suctions. Figure 6.20 indicates the non-linear relationship between the tensile strength and matric suction as obtained from the CW and CD triaxial tests. The experimental results agreed with the results reported by Rampino et al. (1999). This result was also in agreement with the non-linearity of the ϕ^b angle with respect to matric suction as discussed in Chapter 5.

Figure 6.18 Critical state lines in the $(q - p)$ plane from the CD triaxial tests

Figure 6.19 Critical state lines in the $(q - s - p)$ space from the CD triaxial tests

Figure 6.20 The tensile strength due to matric suction from CD triaxial tests for the compacted silt specimens

6.5.2 Critical State on the ($v - p$) Plane

Figure 6.21 shows the stress paths on the specific volume (i.e., $v = 1 + e$) versus logarithm of the mean net stress for the CD triaxial tests under the saturated condition. Figure 6.21 shows that three compacted silt specimens started to shear along the normal compression line. The specific volume decreased and then increased subsequently for specimens CD100-0 and CD200-0. However, the specific volume of specimen CD300-0 decreased throughout the entire shearing stage. Nevertheless, the specific volumes of the saturated CD triaxial tests at the critical state fall on a single straight line, giving the critical state line on the ($v - p$) plane. Figure 6.21 indicates that the slope of the critical state line on the ($v - p$) plane (i.e., $\alpha(0)$) was greater than the slope of the normal compression line (i.e., $\lambda(0)$). Figures 6.22 to 6.25 show the stress paths for the CW and CD triaxial tests under different net confining stresses but at the same matric suctions of 100 kPa, 150 kPa, 200 kPa, and 300 kPa. It appears that the stress paths for the CW and CD triaxial tests showed similar patterns. The specific volume decreased at the early stage of shearing and then increased afterwards. The higher the net confining stress was, the higher the increase in specific volume would be during shearing. However, specimens CW300-100, CW350-100, CW350-150, CW300-200,

CW300-300, CD300-0, CD300-100 and CD300-200 underwent compression throughout the entire shearing stage.

Figure 6.21 Stress paths on the $(v - p)$ plane from the CD tests under saturated condition

Figure 6.22 Stress paths on the $(v - p)$ plane of the CW and CD tests under initial matric suction of 100 kPa

Figure 6.23 Stress paths on the $(v - p)$ plane from the CW tests under initial matric suction of 150 kPa

Figure 6.24 Stress paths on the $(v - p)$ plane from the CW and CD tests under initial matric suction of 200 kPa

Figure 6.25 Stress paths on the $(v - p)$ plane from the CW and CD tests under initial matric suction of 300 kPa

In general, the critical states of the specimens on the same initial matric suction plane fall on a single straight line, which is the critical state line on the $(v - p)$ plane. Tables 6.3 and 6.4 present the stresses at the critical states for the CW and CD triaxial tests, respectively.

Table 6.3 Stress and specific volume at the critical state of the CW triaxial tests

Net confining stress (kPa)	Initial matric suction (kPa)									
	0		100		150		200		300	
	p	v	p	v	p	v	p	v	p	v
100	178	1.931	209	1.918	224	1.916	222	1.914	226	1.907
150	267	1.909	290	1.894	312	1.895	315	1.894	318	1.891
200	352	1.878	382	1.872	398	1.874	405	1.873	409	1.874
250	413	1.868	458	1.859	477	1.861	489	1.861	503	1.862
300	437	1.862	556	1.839	565	1.849	571	1.854	572	1.853

Table 6.4 Stress and specific volume at the critical state of the CD triaxial tests

Net confining stress (kPa)	Initial matric suction (kPa)							
	0		100		200		300	
	p	v	p	v	p	v	p	v
100	175	1.939	215	1.916	225	1.908	228	1.895
200	344	1.881	387	1.868	399	1.872	403	1.871
300	512	1.840	565	1.843	572	1.852	578	1.857

Figures 6.26 and 6.27 show the summary of the specific volume versus logarithm of the mean net stress at the critical state for the CW and CD triaxial tests, respectively. Figures 6.26 and 6.27 indicate that the slope of the critical state line decreased with the increase in matric suction. In other words, a higher the matric suction resulted in a higher stiffness of the soil specimen. As a result, the slope of the critical state line decreased with the increase in matric suction.

Figure 6.26 Critical state lines on the ($v - p$) plane from the CW triaxial tests under different matric suctions

Figure 6.27 Critical state lines on the $(v - p)$ plane from the CD triaxial tests under different matric suctions

Figure 6.28 shows the slope of the critical state line versus matric suction for the CW and CD triaxial tests. The slopes of the critical state lines on the $(v - p)$ plane for the CW and CD triaxial tests at the saturated condition and at the matric suction of 300 kPa were found to be 0.113 and 0.041, respectively. Figure 6.29 plots the specific volume at the reference stress (i.e., $\Gamma(s)$) of the critical state lines versus matric suction. The specific volume at the reference stress also decreased with the increase in matric suction for the CW and CD triaxial tests.

Figure 6.28 Slopes of the critical state line versus matric suction

Figure 6.29 Specific volume at the reference stress versus matric suction

Tables 6.5 and 6.6 present the summary of the critical state parameters of the CW and CD tests on the compacted silt specimen under different net confining stresses and at different matric suctions, respectively.

Table 6.5 Summary of the critical state condition parameters for the CW triaxial tests under different net confining stresses and at different matric suctions

Matric suction (kPa)	$\Gamma(s)$	$\mu(s)$ (kPa)	$\omega(s)$	λ_s	κ_s	$-p_s$ (kPa)
0	2.018	0.0	0.113	0.013	0.002	0.0
80	1.981	66.7	0.074	0.013	0.002	49.6
115	1.970	75.1	0.064	0.013	0.002	57.6
150	1.964	83.6	0.059	0.013	0.002	64.9
245	1.948	99.6	0.047	0.013	0.002	77.1

Table 6.6 Summary of the critical state condition parameters for the CD triaxial tests under different net confining stresses and at different matric suctions

Matric suction (kPa)	$\Gamma(s)$	$\mu(s)$ (kPa)	$\omega(s)$	λ_s	κ_s	$-p_s$ (kPa)
0	2.017	0.0	0.113	0.013	0.002	0.0
100	1.976	70.0	0.066	0.013	0.002	54.7
200	1.957	89.0	0.057	0.013	0.002	69.5
300	1.933	103.0	0.041	0.013	0.002	80.5

6.6 Simulation of the Shearing Test Results under the CW and CD Conditions

6.6.1 Introduction

This section presents the simulation results of the triaxial tests under the CW and CD conditions on compacted silt specimens at the maximum dry density and optimum water content. The theoretical framework for simulating the triaxial tests under the CW and CD conditions was based on the elasto-plastic model with the incorporation of soil-water characteristic curve as described in Chapter 3. The shear strength, excess pore-water pressure and volume change during shearing under the constant water content conditions were simulated using the proposed elasto-plastic model given in Chapter 3. The shear strength and volume change

during shearing under consolidated drained condition were also simulated using the same proposed model. Prior to the simulation of the triaxial tests, the proposed equations were verified first against published data.

6.6.2 Verification of the Proposed Equations

Alonso et al. (1990) used test results on compacted kaolin reported by Josa (1988). The kaolin used in the tests was moderately plastic ($LL = 39\%$, $PL = 27\%$; particles ≤ 0.002 mm = 4% and particles ≤ 0.075 mm = 95%). An estimation of the SWCC can be obtained through the use of a knowledge-based database system such as SoilVision (Fredlund 1997). The SWCC for this kaolin can be estimated using the SWCC equation proposed by Fredlund and Xing (1994).

Figure 6.30 shows the $\lambda(s)$ value obtained from isotropic compression tests reported by Alonso et al. (1990) and the result of the fitted curve using Equation (3.45) presented by Alonso et al. (1990) with r and β values of 0.26 and 0.0164, respectively. Figure 6.30 also shows the fitted curve based on the proposed equation (i.e., Equation (3.46)) with m_λ value of 6.2. The calculated curves from Alonso et al. 1990's and the proposed equations are quite close and in good agreement with the experimental results.

Figure 6.30 Measured and predicted $\lambda(s)$ values with respect to matric suction

The proposed equations were also verified with the experimental results presented by Rampino et al. (2000). Rampino et al. (2000) tested a silty sand from the core of the Metramo dam. The grain size distribution of the silty sand is shown in Figure 6.31. The SWCC for the silty sand can be estimated using the SWCC equation proposed by Fredlund and Xing (1994) with correction factor $C = 1$ (Leong and Rahardjo 1997). Figures 6.32 to 6.34 show the influence of matric suction on the slope of the normal compression lines, the unloading lines and the intercept of the normal compression curves at reference stress. Figures 6.32 to 6.34 also show the best fit curves for the measured $\lambda(s)$, $\kappa(s)$, $N(s)$ of the silty clay using the proposed Equations (i.e., Equations (3.46), (3.47) and (3.48)) with the value of m_λ , m_κ and m_N being equal to 98, 1400 and 23, respectively.

Rampino et al. (2000) used the equation proposed by Alonso et al. (1990) (i.e., Equation (3.45)) to fit the slope of the normal compression lines, $\lambda(s)$, the slope of the unloading lines, $\kappa(s)$ and the intercept of normal compression lines at reference stress, $N(s)$. The results of the fitting curve using Alonso et al.'s (1990) equations are shown in Figures 6.32 to 6.34. The best fit curves for the experimental results of $\lambda(s)$, $\kappa(s)$ and $N(s)$ yield one value of β equal to 0.024 and r values of 0.68, 0.91 and 0.978, respectively.

Figures 6.32 to 6.34 show that the proposed equations (i.e., Equations (3.46) to (3.48)) and Alonso et al (1990)'s equation (i.e., equation (3.45)) can best fit the experimental results closely.

Figure 6.31 The grain size distribution curve of the silty sand (from Rampino, et al. 1999)

Figure 6.32 Measured and predicted $\lambda(s)$ values with respect to matric suction

Figure 6.33 Measured and predicted $\kappa(s)$ values with respect to matric suction Figure 6.34 Measured and predicted $N(s)$ values with respect to matric suction

Figure 6.35 shows the measured yield stresses under a given matric suction as presented by Rampino et al. (2000). The LC yield curve can also be fitted using the proposed equation (i.e., Equation (3.49)) with an n value of 0.0052. Figure 6.35 shows good agreement between the measured and the best fit LC yield curve.

Figure 6.35 Measured and predicted loading-collapse yield curve

6.6.3 Simulation of Soil Parameters for Silt Used in this Study Using the Proposed Equations

The variation of the measured values of $\lambda(s)$ with respect to matric suction for the compacted silt used in this study is shown in Figure 6.36. The variations of the computed $\lambda(s)$ values for different m_λ values (i.e., 30, 39 and 50) with respect to matric suction as obtained from the proposed Equation (3.45) are also shown in Figure 6.36. The parameter m_λ in the proposed Equation (3.46) controls the ratio between the lowest value of $\lambda(s)$ and the value at the saturated condition. It can be seen that the experimental results obtained from this study can be best fit using Equation (3.46) and an m_λ value of 39. Parameter $\lambda(s)$ lies above the experimental results when m_λ equals to 50 and falls below the experimental results when m_λ equals to 30.

Figure 6.36 Effect of the parameter m_λ on the relationship between $\lambda(s)$ and $(u_a - u_w)$

The proposed equations were used to simulate the experimental results obtained from this study. Figures 6.37 to 6.39 show the slope of the normal compression curve, the unloading and the intercept of the normal compression curve. Figures 6.37 to 6.39 present the fitted curves from the proposed equations (i.e., Equations (3.46), (3.47) and (3.48)) with m_λ , m_κ and m_N , values of 39, 55 and 25, respectively. These figures also present the fitted curves from Alonso et al. (1990) with one β value of 0.01 and r values of 0.58, 0.32 and 0.972, respectively. Figures 6.37 to 6.39 show that the fitted curves from the proposed equations and from Alonso et al. (1990) are very close when matric suction exceeded 100 kPa. However, when matric suction is less than 100 kPa, the results based on Alonso et al. (1990) equation deviated from the measured results.

Figure 6.40 shows the measured and the calculated initial yield stresses in the $(s - p)$ plane from the proposed Equation (3.49) with an n value of 0.0031. The results show excellent agreement between the calculated and the measured results.

Figure 6.37 Measured and predicted $\lambda(s)$ values with respect to matric suction Figure 6.38 Measured and predicted $\kappa(s)$ values with respect to matric suction

Figure 6.39 Measured and predicted $N(s)$ values with respect to matric suction

Figure 6.40 Measured data and predicted loading - collapse yield curve

Figures 6.41 and 6.42 shows the measured values of $\alpha(s)$ and $A(s)$ with respect to matric suction, respectively. Figures 6.41 and 6.42 present the calculated values of $\alpha(s)$ and $A(s)$ with respect to matric suction. Figures 6.41 and 6.42 also show the fitted results of $\alpha(s)$ and $A(s)$ from the proposed Equations (3.50) and (3.51) using

m_α and m_A values of 16.5 and 1.3, respectively. The values of $\alpha(s)$ and $A(s)$ could also be back calculated by inserting values of $\lambda_w(s)$ and $N_w(s)$ into Equations (3.72) and (3.73), respectively.

Figure 6.41 Measured and predicted $\alpha(s)$ value with respect to matric suction

Figure 6.42 Measured and predicted $A(s)$ value with respect to matric suction

6.6.4 Simulation of the CW Triaxial Shearing Tests Using the Proposed Model

The theoretical simulation of the shear strength, pore-water pressure and volume changes during shearing under the constant water content condition was carried out using the Microsoft Excel spreadsheet. The soil constants and soil parameter functions used in simulating the shear strength, changes in pore-water pressure and volume during shearing are presented in previous sections.

6.6.4.1 Simulation of Soil Behaviour within the Elastic Region

In order to simulate the shear strength, changes in pore-water pressure and volume within the elastic region, the mean net stress and the deviator stress at the initial yield point were first computed. The mean net stress and the deviator stress at the initial yield point were computed from Equations (3.42) and (3.43), respectively. The elastic volumetric strain induced by the changes in mean net stress was computed from Equation (3.12). The elastic volumetric strain induced by the changes in matric suction was computed from Equation (3.16). The total volumetric strain was computed by summing the elastic volumetric strains induced by the changes in mean net stress and matric suction. The elastic shear strain was computed from Equation (3.14). The change in pore-water pressure or matric suction within the initial yield surface was computed from Equation (3.74). Both sides of Equation (3.74) contain ds (i.e., the change in matric suction), therefore an iterative procedure was used.

6.6.4.2 Simulation of Soil Behaviour within the Plastic Region

The shear strength, pore-water pressure and volume changes beyond the initial yield surface could be calculated by computing the deviator stress at a given mean net stress. The increment of the mean net stress in this simulation was chosen to be 2 kPa (i.e., $p_i = p + \Delta p$ or $p_i = p + 2$).

At any given mean net stress, p_i , the plastic volumetric strain induced by the changes in mean net stress was computed from Equation (3.18). The plastic

volumetric strain induced by the changes in matric suction was computed from Equation (3.20). The plastic shear strain was computed from Equation (3.29). The change in pore-water pressure or matric suction during shearing under the constant water content condition was computed from Equation (3.65). As indicated in Equation (3.65), both sides of the Equation (3.65) also contained ds . Therefore, an iterative procedure was used.

The parameters used in above equations were also computed at any given mean net stress using the proposed equations with the incorporation of SWCC in as explained in Chapter 3. The slope of the normal compression curve (i.e., $\lambda(s)$) was computed from Equation (3.46). The slope of the unloading curve (i.e., $\kappa(s)$) was computed from Equation (3.47). The intercept of the normal compression line at the reference stress (i.e., $N(s)$) was computed from Equation (3.48). The $\alpha(s)$ and $A(s)$ parameters were computed from Equations (3.50) and (3.51) respectively.

6.6.4.3 Simulations Results

The proposed elasto-plastic model with the incorporation of SWCC for an unsaturated soil was used to simulate the shear strength, the changes in pore-water pressure and volumetric strain during triaxial shearing under the constant water content condition. A comparison of the simulated and the measured results of the shear strength, the changes in pore-water pressure and volumetric strain during shearing under the constant water content condition on specimens CW200-100, CW250-100, CW300-100 and CW350-100 is presented in this section. The comparisons of the simulated and the measured results of the shear strength, the changes in pore-water pressure and volumetric strains of other CW triaxial tests are presented in Appendix D.

Figures 6.43 to 6.46 present the comparison of the shear strength during shearing under the constant water content condition from the results of the simulations and the experimental data from specimens CW200-100, CW250-100, CW300-100 and CW350-100, respectively. The simulated shear strength during shearing under the constant water content condition gives a good agreement with the measured result.

Figure 6.43 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the constant water condition of CW200-100 specimen

Figure 6.44 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the constant water condition of CW250-100 specimen

Figure 6.45 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the constant water condition of CW300-100 specimen

Figure 6.46 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the constant water condition of CW350-100 specimen

The simulated changes in pore-water pressure during shearing based on the elasto-plastic theory for the constant water content tests are presented in Figures 6.47 to 6.50 and Appendix D. Figures 6.47 to 6.50 show the comparison between the simulated and measured results of the changes in pore-water pressure during shearing under the constant water content condition for specimens CW200-100, CW250-100, CW300-100 and CW350-100, respectively. The simulated changes in pore-water pressure during shearing using the proposed elasto-plastic model with the incorporation of SWCC were close to the measured data.

Figure 6.47 Comparison between the simulated and the measured results of the changes in pore-water pressure during shearing under the constant water condition of CW200-100 specimen

Figure 6.48 Comparison between the simulated and the measured results of the changes in pore-water pressure during shearing under the constant water condition of CW 250-100 specimen

Figure 6.49 Comparison between the simulated and the measured results of the changes in pore-water pressure during shearing under the constant water condition of CW300-100 specimen

Figure 6.50 Comparison between the simulated and the measured results of the changes in pore-water pressure during shearing under the constant water condition of CW350-100 specimen

Figures 6.51 to 6.54 present the comparisons between the simulated and the measured results of volumetric strains during shearing under the constant water content condition for specimens CW200-100, CW250-100, CW300-100 and CW350-100, respectively. The results show the good agreement between the simulated and the measured data.

Figure 6.51 Comparison between the simulated and the measured results of the volumetric strain during shearing under the constant water condition of CW200-100 specimen

Figure 6.52 Comparison between the simulated and the measured results of the volumetric strain during shearing under the constant water condition of CW250-100 specimen

Figure 6.53 Comparison between the simulated and the measured results of the volumetric strain during shearing under the constant water condition of CW300-100 specimen

Figure 6.54 Comparison between the simulated and the measured results of the volumetric strain during shearing under the constant water condition of CW350-100 specimen

6.6.5 Simulation of the CD Triaxial Shearing Tests Using the Proposed Model

The proposed elasto-plastic model with the incorporation of SWCC for an unsaturated soil was used to simulate the shear strength and volumetric strain during shearing under the drained condition. The theoretical simulation of the shear strength and volume change during shearing under the consolidated drained condition was also carried out using the Microsoft Excel spreadsheet. The shear strength and volume changes during shearing of 12 CD triaxial tests were simulated. The soil constants and soil parameter functions used in simulating the shear strength and volume changes during shearing are presented in previous sections.

6.6.5.1 Simulation of Soil Behaviour within the Elastic Region

In order to simulate the shear strength and volume changes within the elastic region, the mean net stress and the deviator stress at the initial yield point were first computed. The mean net stress and the deviator stress at the initial yield point were computed from Equations (3.42) and (3.43), respectively. The elastic volumetric strain induced by the changes in mean net stress was computed from Equation (3.12). The elastic volumetric strain induced by the changes in matric suction was computed from Equation (3.16). The total volumetric strains were computed by summing the elastic volumetric strains induced by the changes in mean net stress and matric suction. The elastic shear strain was computed from Equation (3.14).

6.6.5.2 Simulation of Soil Behaviour within the Plastic Region

The shear strength and volume changes beyond the initial yield surface could be calculated by computing the deviator stress at a given mean net stress. The increment of the mean net stress in this simulation was chosen to be 2 kPa (i.e., $p_i = p + \Delta p$ or $p_i = p + 2$).

At any given mean net stress, p_i , the plastic volumetric strain induced by the changes in mean net stress was computed from Equation (3.18). The plastic

volumetric strain induced by the changes in matric suction was computed from Equation (3.20). The plastic shear strain was computed from Equation (3.29).

The parameters used in the above equations were computed at any given mean net stress and matric suction using the proposed equations with the incorporation of SWCC as explained in Chapter 3. The slope of the normal compression curve (i.e., $\lambda(s)$) was computed from Equation (3.46). The slope of the unloading curve (i.e., $\kappa(s)$) was computed from Equation (3.47). The intercept of the normal compression line at the reference stress (i.e., $N(s)$) was computed from Equation (3.48).

6.6.5.3 Simulation Results

A comparison of the simulated and the measured results of the shear strength and volumetric strain during shearing under the drained condition for specimens CD300-0, CD300-100, CD300-200 and CD300-300 are presented in this section. The comparisons of the simulated and the measured results of the shear strength and volumetric strains of other CD triaxial tests are presented in Appendix E.

Figures 6.55 to 6.58 present the comparison of the simulated and the measured shear strength during shearing under the drained condition on specimens CD300-0, CD300-100, CD300-200 and CD300-300, respectively. The simulated shear strength during shearing under the drained condition shows a good agreement with the measured result.

Figure 6.55 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the drained condition of CD300-0 specimen

Figure 6.56 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the drained condition of CD300-100 specimen

Figure 6.57 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the drained condition of CD300-200 specimen

Figure 6.58 Comparison between the simulated and the measured results of the deviator stress versus axial strain during shearing under the drained condition of CD300-300 specimen

The simulated volumetric strain during shearing based on the elasto-plastic theory for the CD triaxial tests are presented in Figures 6.59 to 6.62 and Appendix E. Figures 6.59 to 6.62 show the comparison between the simulated and measured

results of the volumetric strain during shearing under the drained condition for specimens CD300-0, CD300-100, CD300-200 and CD300-300, respectively. The results show the agreement between the simulated and the measured results.

Figure 6.59 Comparison between the simulated and the measured results of the volumetric strain during shearing under the drained condition of CD300-0 specimen

Figure 6.60 Comparison between the simulated and the measured results of the volumetric strain during shearing under the drained condition of CD300-100 specimen

Figure 6.61 Comparison between the simulated and the measured results of the volumetric strain during shearing under the drained condition of CD300-200 specimen

Figure 6.62 Comparison between the simulated and the measured results of the volumetric strain during shearing under the drained condition of CD300-300 specimen

6.7 Comparison between Simulation and Experimental Results of the CW and CD Triaxial Tests

6.7.1 Simulation of the CW Triaxial Tests

The comparisons between the simulated and measured shear strength, excess pore-water pressure and volume change during shearing under the constant water content condition are discussed in this section.

6.7.1.1 Simulation of the Shear Strength

The simulations and the measurements of the deviator stress during shearing under the constant water content condition are presented in Figures 6.43 to 6.46 and Appendix D. Figures 6.43 to 6.46 show the simulations and measurements of the deviator stress of the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa. The results show the good agreement between the simulated and the measured deviator stresses of the CW triaxial tests. However, the simulated deviator stress deviated a bit from the measured results at the post-peak strain softening segments. This can be explained by the fact that the proposed elasto-plastic model has not taken into account of the post-peak strain softening behaviours.

Figures 6.63 to 6.66 show the comparison between the simulated and the measured deviator stresses at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa, 150 kPa, 200 kPa and 300 kPa, respectively. The results in Figures 6.63 to 6.66 show the excellent agreement between the simulated and measured deviator stress at failure for the CW triaxial shearing tests. Most of the simulated values were quite close to the straight line representing the condition when the simulated value equal to the measured value.

Figure 6.63 Simulated versus measured deviator stress at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Figure 6.64 Simulated versus measured deviator stress at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

Figure 6.65 Simulated versus measured deviator stress at failure for the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 200 kPa

Figure 6.66 Simulated versus measured deviator stress at failure for the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 300 kPa

6.7.1.2 Simulation of the Pore-water Pressure

Figures 6.47 to 6.50 show the simulations and the measurements of the changes in pore-water pressure for the CW triaxial tests under different net confining stresses but at the same initial of matric suctions 100 kPa, 150 kPa, 200 kPa and 300 kPa, respectively. The results in Figures 6.47 to 6.50 show the good agreement between the simulated and the measured changes in pore-water pressure during shearing under the constant water content condition. However, some of the simulated changes in pore-water pressure during shearing (i.e., specimens CW50-300, CW100-300, CW150-300, CW200-300 and CW300-300) did not follow the same pattern as observed in the measured results. This could be attributed to the fact that the proposed elasto-plastic model has not taken into account the strain softening behaviours.

Figures 6.67 to 6.70 present the summary of the simulated versus measured changes in pore-water pressure at failure for the CW triaxial shear tests under different net confining stresses but at the same initial matric suctions of 100 kPa, 150 kPa, 200 kPa and 300 kPa, respectively. The results in Figures 6.67 to 6.70 show that the simulated changes in pore-water pressure at failure were close to the measured values. However, the simulated changes in pore-water pressure at failure for the CW triaxial tests under the same initial matric suction of 300 kPa deviated from the measured results. It can be seen from Figure 6.70 that the simulated changes in pore-water pressures at failure were lower than the measured results. It can be attributed to the fact that the soil specimen dilated due to strain softening. However, the proposed elasto-plastic model with the incorporation of SWCC did not take into account of the dilation behaviour during shearing. As a result, the simulated changes in pore-water would be higher than the measured results.

Figure 6.67 Simulated versus measured changes in pore-water pressure at failure for the CW triaxial tests under different net confining stresses but at the same initial nitric suction of 100 kPa

Figure 6.68 Simulated versus measured changes in pore-water pressure at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 150 kPa

Figure 6.69 Simulated versus measured changes in pore-water pressure at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 200 kPa

Figure 6.70 Simulated versus measured change in pore-water pressure at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 300 kPa

6.7.1.3 Simulation of Volume Change

Figures 6.71 to 6.74 present the summary of the simulated versus measured volumetric strains at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa, 150 kPa, 200 kPa and 300 kPa, respectively. The results in Figures 6.71 to 6.74 demonstrate that the simulated and measured volumetric strains at failure were in good agreements. However, the simulated volumetric strains for tests at relatively low net confining stresses (i.e., CW50-100, CW50-150, CW100-150, CW50-200 and CW50-300) deviated from the measured results (see Appendix D). This could be due to the fact that the proposed elasto-plastic model with the incorporation of SWCC did not take into account the post-peak strain softening behaviour during shearing under the constant water condition. During the strain softening the volume of specimen dilated. In other words, the volume of the soil specimen increased slightly. As a result, the measured soil volume change was higher than the simulated value as obtained from the proposed equations.

Figure 6.71 Simulated versus measured volumetric strain at failure for the CW triaxial tests under different net confining stresses but at the same initial matric suction of 100 kPa

Figure 6.72 Simulated versus measured volumetric strain at failure for the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 150 kPa

Figure 6.73 Simulated versus measured volumetric strain at failure for the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 200 kPa

Figure 6.74 Simulated versus measured volumetric strain at failure for the CW triaxial tests under different net confining stresses but at the same initial matrix suction of 300 kPa

6.7.2 Simulation of the CD Triaxial Tests

The comparisons between the simulated and measured results of the shear strength and volume change during shearing under the consolidated drained condition are discussed in this section.

6.7.2.1 Simulation of the Shear Strength

The simulations and the measurements of the deviator stress during shearing under the consolidated drained condition are presented in Figures 6.55 to 6.58 and Appendix E. Figures 6.55 to 6.58 present the simulations and measurements deviator stress of the CD triaxial tests under different matrix suctions but at the same net confining stress of 300 kPa. The results show the good agreement between the simulated and the measured deviator stress of the CD triaxial tests.

Figures 6.75 to 6.78 show the comparison between the simulated and the measured values of the deviator stress at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa, 100 kPa, 200 kPa and 300 kPa, respectively. The results in Figures 6.75 to 6.78 show the excellent agreement between the simulated and measured results of the deviator stress at failure for the CD triaxial shear tests. As shown in Figures 6.75 to 6.78, most of the simulated values of the deviator stresses at failure for the CD triaxial tests fall on a single straight line, indicating that the simulated values are equal to the measured value.

Figure 6.75 Simulated versus measured deviator stress at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa

Figure 6.76 Simulated versus measured deviator stress at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

Figure 6.77 Simulated versus measured deviator stress at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

Figure 6.78 Simulated versus measured deviator stress at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

6.7.2.2 Simulation of the Volume Change

Figures 6.79 to 6.82 present the summary of the simulated versus measured volumetric strains at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa, 100 kPa, 200 kPa and 300 kPa, respectively. The results in Figures 6.79 to 6.82 indicate that the simulated volumetric strains at failure were close to the measured results. However, the simulated results of the volumetric strains for tests that were conducted under a relatively low net confining stress and at a high matric suction deviated from the measured results. This could be due to the fact that dilation at post-peak strain softening occurred during tests that were conducted under a relatively low net confining stress and at a high matric suction. In other words, the volume of the soil specimen increased after post-peak. However, the proposed elasto-plastic model did not take into account of the dilation behaviour during shearing. As a result, the simulated result would deviate from the measured data.

Figure 6.79 Simulated versus measured volumetric strain at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of zero kPa

Figure 6.80 Simulated versus measured volumetric strain at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of 100 kPa

Figure 6.81 Simulated versus measured volumetric strain at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of 200 kPa

Figure 6.82 Simulated versus measured volumetric strain at failure for the CD triaxial tests under different net confining stresses but at the same matric suction of 300 kPa

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

All conclusions made from this study are presented in this chapter. Subsequently, the recommendations for future studies are also presented.

7.1 Conclusions

Some conclusions from this study can be summarized as follows:

1. The NTU mini suction probes for the negative pore-water pressure measurements performed well during their use in the tests for obtaining SWCC and the constant water content triaxial tests.
2. The results of the pore-water pressure measurements from the NTU mini suction probes indicate the uniformity of pore-water pressure throughout the specimen during the CW triaxial tests under the strain rate of 0.009mm/min. The difference between pore-water pressure measurements from the NTU mini suction probes and the base plate was about 1.5 kPa. The difference between the pore-water pressure measurements from the NTU mini suction probes was less than 1.0 kPa. The pore-water pressure measurement from any of the NTU mini suction probes along the soil specimen is representative of the pore-water pressure in the entire height of specimen during shearing.
3. The experimental results from SWCC tests on a compacted silt under different net confining stresses show that the air-entry value increased with

- the increase in net confining stress. The slopes of SWCC beyond the air-entry value, AEV, do not appear to depend on the net confining stress. The AEV increased nonlinearly with the increase in mean net stress.
4. The yield points (i.e., yield suction, s_0) obtained from the specific volume versus matric suction curve also increased with the increase in net confining stress. The slopes of the specific volume versus matric suction curve beyond the yield suction, s_0 , also do not appear to depend on the net confining stress. The yield suction, s_0 , increased nonlinearly with the increase in mean net stress on the ($s - p$) plane. This demonstrated the effect of net confining stress on the yield suction on ($s - p$) plane of the SWCC tests.
 5. The results of the isotropic consolidation tests indicate the strong influence of matric suction on the compressibility and stiffness of the compacted silt specimens. The yield stress increased with the increase in matric suction for the compacted silt specimen. The slope of the normal consolidation curve, the slope of unloading curve and the intercept of the normal compression curve at the reference stress decreased with the increase in matric suction.
 6. The results of the isotropic consolidation tests show the effects of initial dry density and water content on characteristics of the consolidation curves. The slope of the normal compression curve increased with the decrease in initial dry density. Meanwhile, the slope of the unloading curve and the yield stress decreased with the decrease in initial dry density.
 7. The experimental results from the CW and CD triaxial tests on the compacted silt specimens at the maximum dry density and optimum water content showed that a high initial matric suction and a low net confining stress in the soil specimen resulted in a highly overconsolidated soil. Therefore, the soil specimen would exhibit a high dilation and post-peak strain softening behaviour.
 8. The results of the CW and CD triaxial tests show that there was a

relationship between shear strength and volume change during shearing. The volume change remained essentially constant when the deviator stress became constant after reaching the post-peak strain softening, indicating the critical state condition.

9. The results indicate that the effective angles of internal friction, ϕ' , and the effective cohesions, c' , of the compacted silt as obtained from both the CW and CD tests are identical. The results of the CW and CD triaxial tests showed that the effective angle of internal friction, ϕ' , and effective cohesion, c' , of the compacted silt were 32° and zero kPa, respectively. Non-linearity was observed in the relationship between ϕ^b and matric suction from the CW and CD triaxial tests for the compacted silt. The ϕ^b angle was found to be the same as the effective angle of internal friction, ϕ' (i.e., 32°) at low matric suctions (i.e., matric suctions lower than the air-entry value). The ϕ^b angle decreased to a magnitude as low as 12° at high matric suctions (i.e., matric suctions higher than the residual matric suction). However, the ϕ^b angles from the CW and CD tests were different at matric suctions between the air-entry value and the residual matric suction value. The difference was due to the lower failure envelope with respect to matric suction from the CW tests as compared to the failure envelope from the CD tests within this range of matric suction. This difference could be attributed to the hysteretic behaviour of the soil-water characteristic curve.
10. The percentage change of the matric suction with respect to the initial matric suction during shearing under the constant water content condition increased gradually with the increase in the initial matric suction and then decreased when the initial matric suction was higher than residual matric suction. The D_w' parameter (i.e., tangent pore-water pressure parameter) increased with the increase in net confining stresses and it remained essentially constant at failure.
11. The critical state lines at different matric suctions on the $(q - p)$ plane were

parallel with a slope of 1.28 for both the CW and CD triaxial tests. This indicated the unique relationship between the deviator stress and mean net stress for both the CW and CD triaxial tests. The results also indicated the unique relationship between the specific volume and mean net stress on the $(v-p)$ plane for both the CW and CD triaxial tests. The slope of the critical state on the $(v-p)$ plane of both the CW and CD triaxial tests decreased with the increase in matric suction.

12. The Alonso et al.'s (1990) equation would require two constant material parameters (i.e., β and r). However, one of the parameters (i.e., β) could be eliminated by incorporating soil-water characteristic curve (SWCC) in the proposed equations for the determination of the soil parameter functions. Furthermore, the Alonso et al.'s (1990) equation required at least three $\lambda(s)$ values at three different matric suctions in order to fit the experimental results. The proposed equations only required two measured $\lambda(s)$ values for fitting the experimental results. Reasonably good agreements between several sets of experimental data and the calculated results based on the equations of the proposed elasto-plastic model with the incorporation of SWCC have been illustrated.
13. Reasonably good agreements between the analytical simulation based on the proposed elasto-plastic model with the incorporation of SWCC and the experimental results of the shear strength, change in pore-water pressure and volume change during shearing under the constant water content condition were obtained in this study. The proposed elasto-plastic model with the incorporation of SWCC provided a useful insight into the relationship between shear strength, pore-water pressure and volume change of an unsaturated soil.
14. A good agreement between the proposed elasto-plastic model with the incorporation of SWCC and the experimental results for the shear strength and volume change during shearing under the consolidated drained condition was achieved in this study.

7.2 Recommendations

The recommendations for future studies of this research can be divided into two main categories namely, laboratory works and theoretical simulations.

1. Laboratory Works

- In order to study the characteristics of SI and LC yield curves on the $(s - p)$ plane, the shape of the SI yield curve and the relationship between SI and LC should be investigated for other types of soil.
- It would be useful to explore a wider range of soil suction in the testing programme.

2. Theoretical Simulations

The proposed elasto-plastic model with the incorporation of SWCC still has some limitations. The proposed elasto-plastic model can not simulate strain softening and soil dilation during shearing. The proposed elasto-plastic model with the incorporation of SWCC should be improved by introducing the softening parameter in order to achieve good simulation results for the strain softening part.

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APPENDIX A

Appendix A Calibration data for modified triaxial apparatus for obtaining SWCC

Table A.1 Calibration data of “Top” NTU mini suction probe

Pressure (kPa)	Transducer reading (V)	Pressure (kPa)	Transducer reading (V)
1	-0.002636719	700.47	0.036296946
23.66	-0.001362017	599.84	0.030682253
48.94	0.000006412	500.94	0.025152473
74	0.001391602	400.03	0.019541277
100	0.002837745	376	0.018209604
125	0.004222548	350	0.016778564
149.44	0.005582503	325.88	0.015411975
174	0.006956599	300.56	0.014007568
200.66	0.008375214	275	0.012591411
224	0.009723772	249.97	0.011185643
248.94	0.011119162	225.31	0.009814453
275.03	0.012574907	200.06	0.008420410
298.5	0.013876318	175.5	0.007052378
324.09	0.015314660	149.88	0.005623065
350.31	0.016754957	125.5	0.004277918
399.97	0.019520569	100.56	0.002890927
499.69	0.025092022	76.09	0.001522667
599.81	0.030674977	51	0.000135953
698.94	0.036209341	26	-0.001229955
799.5	0.041816311	1.03	-0.002621248

Figure A.1 Calibration of the "Top" NTU mini suction probe

Table A.2 Calibration data of “Middle” NTU mini suction probe

Pressure (kPa)	Transducer reading (V)	Pressure (kPa)	Transducer reading (V)
1	0.007250642	700.47	0.047967665
23.66	0.008587311	599.84	0.042093038
48.94	0.010012119	500.94	0.036321066
74	0.011463914	400.03	0.030455280
100	0.012975951	376	0.029040799
125	0.014437734	350	0.027552049
149.44	0.015844184	325.88	0.026113596
174	0.017288411	300.56	0.024654405
200.66	0.018770590	275	0.023170381
224	0.020193685	249.97	0.021711483
248.94	0.021639785	225.31	0.020265230
275.03	0.023154507	200.06	0.018816189
298.5	0.024532720	175.5	0.017379823
324.09	0.026023626	149.88	0.015877821
350.31	0.027537504	125.5	0.014474284
399.97	0.030430464	100.56	0.013018733
499.69	0.036248483	76.09	0.011600617
599.81	0.042087721	51	0.010137874
698.94	0.047870551	26	0.008717534
799.5	0.053741029	1.03	0.007262629

Figure A.2 Calibration of the “Middle” NTU mini suction probe

Table A.3 Calibration data of “Bottom” NTU mini suction probe

Pressure (kPa)	Transducer reading (V)	Pressure (kPa)	Transducer reading (V)
1	-0.004772030	700.47	0.035043403
23.66	-0.003464526	599.84	0.029301405
48.94	-0.002079671	500.94	0.023668515
74	-0.000647554	400.03	0.017918945
100	0.000826211	376	0.016551649
125	0.002251344	350	0.015096721
149.06	0.003619249	325.88	0.013685001
174	0.005037977	300.56	0.012257894
200.66	0.006487088	275	0.010799996
224	0.007879232	249.97	0.009378675
248.94	0.009303595	225.31	0.007965230
275.03	0.010779023	200.06	0.006541341
298.5	0.012129851	175.5	0.005135976
324.09	0.013584682	149.88	0.003671332
350.31	0.015074574	125.5	0.002302797
399.97	0.017900284	100.56	0.000878332
499.69	0.023583984	76.09	-0.000515058
599.81	0.029287949	51	-0.001936164
698.94	0.034939236	26	-0.003329050
799.5	0.040697157	1.03	-0.004753418

Figure A.3 Calibration of the “Bottom” NTU mini suction probe

Table A.4 Calibration data of pore-air pressure transducer

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
0.0	0.0147880122	550.5	0.0928441541
50.5	0.0220210009	599.3	0.0997558594
99.0	0.0288957076	496.9	0.0852561008
155.4	0.0368874290	403.2	0.0719977548
200.0	0.0431997235	331.1	0.0617738236
249.4	0.0501794022	272.4	0.0534624078
300.1	0.0573522587	175.9	0.0398115140
351.3	0.0646007319	136.2	0.0341564586
398.8	0.0713356195	69.4	0.0247081285
449.8	0.0785921028	0.0	0.0147422097
498.6	0.0854743173

Figure A.4 Calibration of pore-air pressure transducer

Table A.5 Calibration data of pore-water pressure transducer

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
0.0	0.0008319930	604.2	0.0867621528
55.1	0.0088168832	575.7	0.0827172256
100.3	0.0152659346	521.5	0.0750440875
152.5	0.0226491292	464.1	0.0669032333
200.1	0.0294265221	408.4	0.0590277168
252.3	0.0368504842	367.5	0.0532105864
300.1	0.0436409884	301.3	0.0438087035
353.4	0.0512077945	217.4	0.0319060630
400.2	0.0578507389	191.0	0.0281724591
451.3	0.0650741577	138.3	0.0206559245
500.5	0.0720465829	80.6	0.0124492381
551.0	0.0792194772	41.4	0.0068739780
599.4	0.0860678711	0.0	0.0008489034
651.5	0.0934561100

Figure A.5 Calibration of pore-water pressure transducer

APPENDIX B

Calibration data for modified triaxial apparatus for the isotropic consolidation tests

Table B.1 Calibration data of pore-air pressure transducer

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
0.0	0.0147880122	550.5	0.0928441541
50.5	0.0220210009	599.3	0.0997558594
99.0	0.0288957076	496.9	0.0852561008
155.4	0.0368874290	403.2	0.0719977548
200.0	0.0431997235	331.1	0.0617738236
249.4	0.0501794022	272.4	0.0534624078
300.1	0.0573522587	175.9	0.0398115140
351.3	0.0646007319	136.2	0.0341564586
398.8	0.0713356195	69.4	0.0247081285
449.8	0.0785921028	0.0	0.0147422097
498.6	0.0854743173

Figure B.1 Calibration of pore-air pressure transducer

Table B.2 Calibration data of pore-water pressure transducer

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
0.0	0.0008319930	604.2	0.0867621528
55.1	0.0088168832	575.7	0.0827172256
100.3	0.0152659346	521.5	0.0750440875
152.5	0.0226491292	464.1	0.0669032333
200.1	0.0294265221	408.4	0.0590277168
252.3	0.0368504842	367.5	0.0532105864
300.1	0.0436409884	301.3	0.0438087035
353.4	0.0512077945	217.4	0.0319060630
400.2	0.0578507389	191.0	0.0281724591
451.3	0.0650741577	138.3	0.0206559245
500.5	0.0720465829	80.6	0.0124492381
551.0	0.0792194772	41.4	0.0068739780
599.4	0.0860678711	0.0	0.0008489034
651.5	0.0934561100

Figure B.2 Calibration of pore-water pressure transducer

APPENDIX C

Calibration data for modified triaxial apparatus for the CW and CD tests

Table C.1 Calibration data of pore-water pressure transducer

Pressure, kPa	Transducer reading (V)
0	0.2516818576
100	0.2633248788
200	0.2748087170
300	0.2863028390
400	0.2977399554
500	0.3092405968
600	0.3206654744
700	0.3320842704
800	0.3435341971
700	0.3320933751
600	0.3206604004
500	0.3092428768
400	0.2977567785
300	0.2862867272
200	0.2748046875
100	0.2633167614

Figure C.1 Calibration of pore-water pressure transducer

Table C.2 Calibration data of pore-air pressure transducer

Pressure, kPa	Transducer reading (V)
0	0.0251201112
100	0.0262775478
200	0.0274185844
300	0.0285644531
400	0.0297022073
500	0.0308426838
600	0.0319831470
700	0.0331191119
800	0.0342616750
700	0.0331205765
600	0.0319822998
500	0.0308444553
400	0.0297034061
300	0.0285637434
200	0.0274187873
100	0.0262793210

Figure C.2 Calibration of pore-air pressure transducer

Table C.3 Calibration data of LVDT for axial displacement

Micrometer reading (mm)	LVDT reading (V)	Micrometer reading (mm)	LVDT reading (V)
0	0.0251845915	24	0.0320766449
1	0.0254720052	23	0.0317915096
2	0.0257567260	22	0.0315010210
3	0.0260447096	21	0.0312107316
4	0.0263322242	20	0.0309218255
5	0.0266185672	19	0.0306354183
6	0.0269078630	18	0.0303480114
7	0.0271953791	17	0.0300574757
8	0.0274831789	17	0.0300582159
9	0.0277724155	16	0.0297704787
10	0.0280618722	15	0.0294836191
11	0.0283441767	14	0.0291939051
12	0.0286356394	13	0.0289095052
13	0.0289209458	12	0.0286196108
14	0.0292087380	11	0.0283290157
15	0.0294994266	10	0.0280450003
16	0.0297868756	9	0.0277570801
17	0.0300723297	8	0.0274666164
18	0.0303621859	7	0.0271825062
19	0.0306484576	6	0.0268945071
20	0.0309318906	5	0.0266075942
21	0.0312207903	4	0.0263236527
22	0.0315106201	3	0.0260372351
23	0.0317994361	2	0.0257507324
24	0.0320867255	1	0.0254629517
25	0.0323770244	0	0.0251754799

Figure C.3 Calibration of LVDT for axial displacement

Table C.4 Calibration data of 5 kN axial load cell

Weight load , (N)	Load cell reading (V)	Weight load , (N)	Load cell reading, (V)
0	0.0248290000	2440.46	0.0258678618
187.263	0.0249000000	2613.46	0.0259516552
360.563	0.0249780000	2786.76	0.0260403878
533.863	0.0250513413	2960.06	0.0261192245
707.163	0.0251248988	3133.36	0.0261974669
880.463	0.0251938178	3306.66	0.0262749082
1053.76	0.0252685547	3479.96	0.0263558524
1227.06	0.0253376286	3653.26	0.0264318557
1400.36	0.0254148211	3826.56	0.0265147427
1573.66	0.0254857680	3999.86	0.0265986851
1746.96	0.0255641128	4173.16	0.0266747190
1920.26	0.0256353106	4346.46	0.0267488920
2093.56	0.0257110963	4519.76	0.0268307334

Figure C.4 Calibration data of 5 kN axial load cell

Table C.5 Calibration data of “Top” NTU mini suction probe

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
1	0.0242131042	750	0.0340916625
49	0.0248511781	700	0.0334294412
99	0.0255174720	650	0.0327683075
149	0.0261720276	600	0.0321072193
199	0.0268259841	550	0.0314496813
249	0.0274767707	501	0.0307888324
299	0.0281346144	450	0.0301197799
350	0.0287993295	399	0.0294458692
400	0.0294697664	350	0.0288045883
449	0.0301061073	301	0.0281592874
498	0.0307638864	249	0.0274829621
549	0.0314184045	201	0.0268442885
599	0.0320966578	151	0.0261876989
649	0.0327570489	101	0.0255329273
699	0.0334155571	51	0.0248726478
749	0.0340756065	1	0.0242196219
799	0.0347323671

Figure C.5 Calibration of “Top” NTU mini suction probe

Table C.6 Calibration data of “Middle” NTU mini suction probe

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
1	0.0247469815	750	0.0344391999
49	0.0253711177	700	0.0337861561
99	0.0260258427	650	0.0331305804
149	0.0266749391	600	0.0324926284
199	0.0273260789	550	0.0318443368
249	0.0279706365	501	0.0311950684
299	0.0286132813	451	0.0305454147
350	0.0292781725	400	0.0299025657
399	0.0299137449	349	0.0292355069
449	0.0305576361	300	0.0286083515
499	0.0312006119	249	0.0279451423
549	0.0318493764	201	0.0273167601
599	0.0325012207	151	0.0266752226
649	0.0331423246	101	0.0260291637
699	0.0337624086	51	0.0253868173
749	0.0344339267	1	0.0247400969
799	0.0350920791

Figure C.6 Calibration of “Middle” NTU mini suction probe

Table C.7 Calibration data of “Bottom” NTU mini suction probe

Pressure, (kPa)	Transducer reading, (V)	Pressure, (kPa)	Transducer reading, (V)
1	0.0248546154	750	0.0345585618
49	0.0254802179	700	0.0339049163
99	0.0261282462	650	0.0332510572
149	0.0267786899	600	0.0326108446
199	0.0274247196	550	0.0319624681
249	0.0280744123	501	0.0313118191
299	0.0287135133	451	0.0306632409
350	0.0293812678	400	0.0300177557
399	0.0300143902	349	0.0293492868
449	0.0306602977	300	0.0287206807
499	0.0313016183	249	0.0280588786
549	0.0319550691	201	0.0274283447
599	0.0326081748	151	0.0267861825
649	0.0332528491	101	0.0261393599
699	0.0338780330	51	0.0254969517
749	0.0345494034	1	0.0248500279
799	0.0352075880

Figure C.7 Calibration of “Bottom” NTU mini suction probe

APPENDIX D

Simulation of the CW triaxial Tests Results Using the Proposed Elasto-plastic Model with the Incorporation of SWCC

Figure D.1 Comparison between the simulated and measured results of the deviators stress versus axial strain during shearing for the specimen CW50-100

Figure D.2 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW50-100

Figure D.3 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW50-100

Figure D.4 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW100-100

Figure D.5 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW100-100

Figure D.6 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW100-100

Figure D.7 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW150-100

Figure D.8 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW150-100

Figure D.9 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW150-100

Figure D.10 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW50-150

Figure D.11 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW50-150

Figure D.12 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW50-150

Figure D.13 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW100-150

Figure D.14 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW100-150

Figure D.15 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW100-150

Figure D.16 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW150-150

Figure D.17 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW150-150

Figure D.18 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW150-150

Figure D.19 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW200-150

Figure D.20 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW200-150

Figure D.21 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW200-150

Figure D.22 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW250-150

Figure D.23 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW250-150

Figure D.24 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW250-150

Figure D.25 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW300-150

Figure D.26 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW300-150

Figure D.27 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW300-150

Figure D.28 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW350-150

Figure D.29 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW350-150

Figure D.30 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW350-150

Figure D.31 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW50-200

Figure D.32 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW50-200

Figure D.33 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW50-200

Figure D.34 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW100-200

Figure D.35 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW100-200

Figure D.36 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW100-200

Figure D.37 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW150-200

Figure D.38 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW150-200

Figure D.39 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW150-200

Figure D.40 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW200-200

Figure D.41 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW200-200

Figure D.42 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW200-200

Figure D.43 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW250-200

Figure D.44 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW250-200

Figure D.45 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW250-200

Figure D.46 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW300-200

Figure D.47 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW300-200

Figure D.48 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW300-200

Figure D.49 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW50-300

Figure D.50 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW50-300

Figure D.51 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW50-300

Figure D.52 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW100-300

Figure D.53 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW100-300

Figure D.54 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW100-300

Figure D.55 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW150-300

Figure D.56 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW150-300

Figure D.57 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW150-300

Figure D.58 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW200-300

Figure D.59 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW200-300

Figure D.60 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW200-300

Figure D.61 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CW250-300

Figure D.62 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW250-300

Figure D.63 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW250-300

Figure D.64 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CW300-300

Figure D.65 Comparison between the simulated and measured results of the change in pore-water pressure versus axial strain during shearing for the specimen CW300-300

Figure D.66 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CW300-300

APPENDIX E

Simulation of the CD triaxial Tests Results Using the Proposed Elasto-plastic Model with the Incorporation of SWCC

Figure E.1 Comparison between the simulated and measured of the deviators stress versus axial strain during shearing for the specimen CD100-0

Figure E.2 Comparison between the simulated and measured of the volumetric strain versus axial strain during shearing for the specimen CD100-0

Figure E.3 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CD200-0

Figure E.4 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD200-0

Figure E.5 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CD100-100

Figure E.6 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD100-100

Figure E.7 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CD200-100

Figure E.8 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD200-100

Figure E.9 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CD100-200

Figure E.10 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD100-200

Figure E.11 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CD200-200

Figure E.12 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD200-200

Figure E.13 Comparison between the simulated and measured results of the deviator stress versus axial strain during shearing for the specimen CD100-300

Figure E.14 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD100-300

Figure E.15 Comparison between the simulated and measured results of the deviators tress versus axial strain during shearing for the specimen CD200-300

Figure E.16 Comparison between the simulated and measured results of the volumetric strain versus axial strain during shearing for the specimen CD200-300