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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**HYSTERESIS EFFECTS ON MECHANICAL
BEHAVIOUR OF UNSATURATED SOILS**

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**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY
SINGAPORE
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ABSTRACT

In response to varying climatic conditions, soils above groundwater table and near the ground surface undergo drying and wetting. Under the same stress state, a soil exhibits different water contents and mechanical behaviour when it follows a drying process as opposed to following a wetting process. These differences are referred to as hysteresis of soil. Many research works were conducted on unsaturated soil; however, most of them were limited to soils under drying. The objective of this research is to study the mechanical behaviour of unsaturated soil, particularly the shear strength of soil under hysteresis effects. Twelve published shear strength equations were selected for evaluation using data from literatures. In this study, equations for predicting the unsaturated shear strengths of a soil under drying and wetting were proposed. Series of unsaturated Consolidated Drained (CD) tests were conducted on three different sand-kaolin specimens under multi-cycles of drying and wetting while series of multi-cycled Soil-Water Characteristic Curves (SWCC) tests were conducted on three different sand-kaolin mixtures and three different sands.

The CD test results showed that the specimens on the first cycle of drying had a higher shear strength, a higher axial strain at failure and exhibited more ductility, less stiffness, and contractive behaviour during shearing. On the other hand, it was observed that the specimens on the first cycle of wetting had a lower shear strength, a lower axial strain at failure and exhibited less ductility, more stiffness and dilative behaviour during shearing. In addition, the differences between the shear strength characteristics on the drying and wetting paths of the first cycle were found to be more significant than the differences from the drying and wetting paths of the second cycle. The SWCC test results showed that the difference between the drying and wetting SWCCs of the first cycle were larger than those of the subsequent cycles. In addition, the multi-cycled SWCCs of soils were affected by the applied net confining pressure, initial dry density and type of soil significantly. The proposed equations were shown to predict the measured drying and wetting shear strength obtained from this study and the shear strength data from literature successfully in the comparative analyses performed in this study.

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LIST OF SYMBOLS

a	= a soil parameter which is primarily a function of the AEV of the soil (kPa)
a_{bi}	= fitting parameter that related to the initial breaking point of the curve
a_{gr}	= fitting parameter that corresponding to initial break of equation (i.e., represent the large particle size)
b	= curve-fitting parameter
b_d	= parameter, b , for drying shear strength prediction
b_w	= parameter, b , for wetting shear strength prediction
c	= cohesion intercept
c_{ult}	= cohesion intercept at maximum matric suction
c'	= effective cohesion
d	= fitting parameter
d	= particle diameter of soil (mm)
d	= ordinate intercept on the q versus r plane where r_f is zero
d_m	= minimum particle diameter of soil (mm)
d_{rhi}	= parameter that related to the amount of fines in a soil (mm)
d'	= intercept of the stress point envelope on the q axis when the p_f and r_f are equal to zero
e	= natural number, 2.71828...
f_w	= fitting parameter that controlling the split between upper and lower portions
g	= gravitational acceleration
h	= head loss
h_b	= head loss along the bottom ceramic disk
h_{gr}	= residual particle size of soil (mm)
h_s	= head loss along the soil specimen
h_t	= head loss along the top ceramic disk

i	= hydraulic gradient, i.e. $i = h/L$
j_{bi}	= fitting parameter that related to the second breaking point
k_b	= water coefficient of permeability of bottom ceramic disk
k_{bi}	= fitting Parameter that related to the maximum slope of the second hump
k_s	= water coefficient of permeability of soil at saturated condition
k_T	= water coefficient of permeability of the disk-soil-disk system
k_t	= water coefficient of permeability of top ceramic disk
k_w	= water coefficient of permeability of soil
l_{bi}	= fitting parameter that related to the shape of the second hump
m	= a soil parameter which is primarily a function of the residual water content
m	= the gradient of the line
m_{bi}	= fitting parameter that related to the shape of the curve
m_{gr}	= fitting parameters that corresponding to curvature of equation
n	= a soil parameter which is primarily a function of the rate of water extraction from the soil once the AEV has been exceeded
n_{bi}	= fitting parameter that related to the steepest slope of the curve
n_d	= fitting parameter, n , from Fredlund and Xing (1994) equation for drying SWCC
n_{gr}	= fitting parameter that corresponding to maximum slope of equation
n_w	= fitting parameter, n , of Fredlund and Xing (1994) equation for wetting SWCC
p	= fitting parameter
p_f	= half of the deviator stress at failure, $((\sigma_{1f} - \sigma_{3f})/2)$
$p, q, x, y, d, h, b,$	= hypothetical parameters used in development of proposed shear strength equation
z	
q	= a fitting parameter which is related to the asymmetry of the SWCC curve
q_t	= total flow rate through the cross-sectional area
q_f	= mean net normal stress at failure, $((\sigma_{1f} + \sigma_{3f})/2 - u_a)$
r_f	= matric suction at failure, $(u_a - u_w)_f$

t	=	elapsed time
u_a	=	pore-air pressure
u_w	=	pore-water pressure
v	=	flow velocity
v_b	=	flow velocity through the bottom ceramic disk
v_s	=	flow velocity through the soil specimen
v_t	=	flow velocity through the top ceramic disk
y_d	=	parameter, y for drying shear strength prediction
y_i	=	actual value of i th data
y_w	=	parameter, y , for wetting shear strength prediction
γ_w	=	parameter, y for wetting shear strength prediction
$\hat{y}_i$	=	actual value of i th data
A	=	cross-sectional area
AEV	=	air entry value
$AEVI$	=	air-entry value when net normal stress is zero (intercept of the line, kPa)
$C(\psi)$	=	correction function that forces the SWCC through a matric suction of 1 GPa and zero water content
L	=	length of sample
L_b	=	thickness of the bottom ceramic disk
L_s	=	height of the soil specimen
L_T	=	height of the disk-soil-disk system, i.e. $L_T = L_s + L_t + L_b$
L_t	=	thickness of the top ceramic disk
N	=	total number of data available
P_{at}	=	atmospheric pressure
P_p	=	passing percentage at any particular grain-size, d
PI	=	plasticity index
Q_w	=	volume of water flow through the soil specimen
R_s	=	radius of curvature of the meniscus
S	=	degree of saturation
T_s	=	surface tension of water
α	=	initial angle of shear strength with respect to matric suction

β	=	calculated fitting parameter
β_1	=	peak value of shear strength contribution from matric suction
β, γ, λ	=	fitting parameters
θ	=	volumetric water content
θ_r	=	residual volumetric water content
θ_s	=	saturated volumetric water content
θ_w	=	volumetric water content
κ	=	fitting parameter
λ	=	fitting parameter
ρ_d	=	dry density
ρ_w	=	density of water
σ_{1f}	=	major principle stress at failure
σ_{3f}	=	minor principle stress at failure
τ	=	shear strength of soil
τ_s	=	the contribution of matric suction to the shear strength of unsaturated soil
τ_{sr}	=	matric suction contribution to shear strength at residual matric suction
χ	=	a parameter dependent on the degree of saturation
ψ	=	matric suction of soil
ψ^b	=	slope angle of the stress point envelope with respect to the stress variable, r_f
ϕ^b	=	angle indicating the rate of change in shear strength with respect to matric suction
ϕ_1^b	=	angle indicating the rate of change in shear strength with respect to matric suction, which is a constant value, introduced in Fredlund et al. (1978) equation.
ψ_e	=	air-entry value
ψ_r	=	residual matric suction
ψ'	=	slope angle of the stress point envelope
ϕ'	=	effective internal friction angle of saturated soil
ϕ	=	calculated fitting parameter

Θ	=	normalized volumetric water content
$(\sigma - u_a)$	=	net normal stress
$(u_a - u_w)$	=	matric suction
$(u_a - u_w)_b$	=	air-entry value of soil
$(u_a - u_w)_r$	=	residual matric suction
Δu	=	Pore-water change after the increment of confining pressure
$\Delta\sigma_3$	=	increment of confining pressure

CHAPTER 1 INTRODUCTION

1.1 RESEARCH BACKGROUND

The emerging areas of the unsaturated soil mechanics has gradually become an important component of geotechnical engineering in the last few decades. It is required for addressing numerous geotechnical problems such as slope stability, retaining wall, excavation, embankment and bearing capacity. Although a number of theories, models and equations have been developed to describe the behaviour of unsaturated soil, there are still a lot of uncertainties and unpredictable characteristics of unsaturated soils that have not been well-studied and fully understood. The interactions of the four phases, air, water, soil particles and contractile skin, which exist in an unsaturated soil, have caused the unsaturated soil mechanics to become more sophisticated than the conventional saturated soil mechanics.

Residual soil covers about two-third of Singapore (Pitts, 1984). Usually, residual soil slopes have a deep groundwater table. The soils between the groundwater table and the ground surface are in unsaturated condition with negative pore-water pressure or matric suction. The matric suction in an unsaturated soil can range from zero at the water table to a maximum of 1 GPa under a very dry condition (Croney et al., 1958).

Matric suction in an unsaturated soil changes with varying climatic conditions. Climatic conditions subject soils near the ground surface to cyclic drying and wetting. During a dry period, due to soil water evaporation, pore-water pressure decreases and matric suction increases. During a wet period, due to rain water infiltration, pore-water pressure increases and matric suction decreases. The changes in matric suction result in the changes of soil properties and mechanical behaviours. The unsaturated soil behaviour changes along the alternate drying and wetting cycles result in hysteresis in the soil. Hysteresis is referred to the differences in the behaviour of soil under the drying and wetting processes.

Hysteresis is a complex mechanism that has not been fully investigated and understood. During drying and wetting of a soil, the soil experiences hysteresis. Shear strength and volume change as well as permeability of a soil behave differently under the drying and wetting processes or on the scanning paths. These differing behaviours could introduce difficulties in geotechnical designs. Engineers need to decide which condition (drying or wetting) should be modeled and subsequently use the appropriate unsaturated functions in the design. The mechanical behaviours of soil under wetting is crucial for practical purposes since most of the geotechnical problems, such as slope failure, are induced by rainfall that generally induces a wetting process in soil. Although, many research works have been carried out on the mechanical behaviour and permeability of an unsaturated soil, most of them were limited to the soil under the drying process. In contrast, the mechanical behaviour and permeability of unsaturated soil during wetting are still not completely understood. Therefore, it is a necessity to understand the hysteresis effects on the mechanical behaviour and permeability of unsaturated soil to provide a better geotechnical design.

1.2 OBJECTIVE

The main objective of this research is to study the hysteresis effects on the characteristics of shear strength, volume change and permeability of an unsaturated soil that have undergone cycles of drying and wetting processes.

1.3 SCOPES

In this research, the scopes consist of two main parts that are theoretical development and experimental programme. The theoretical development includes a comprehensive literature review, comparison studies and the model developments. The scopes are as follows,

- To study and review the general unsaturated soil behaviours as well as the mechanical behaviours, permeability and macrostructure characteristics of soils during drying and wetting processes from literature.

- To study, review and evaluate the unsaturated shear strength equations and Soil-Water Characteristic Curve (SWCC) estimation methods from literature.
- To develop formulations to predict shear strength of unsaturated soils under drying and wetting processes.
- To compare the predicted shear strength of unsaturated soils with the experimental data from this research as well as literatures.

For the experimental programme, it consists of soil selection and series of laboratory works. The scopes are as follows,

- To investigate suitable soils that can fulfill the experimental requirements.
- To modify experimental apparatus that can accommodate the designed laboratory tests.
- To investigate the hysteresis of different soils under single cycle and multi-cycles of drying and wetting processes.
- To investigate the shear strength, permeability and volume change characteristics of unsaturated soils under single cycle and multi-cycles of drying and wetting processes.

1.4 METHODOLOGY

This research mainly focused on the laboratory tests as well as the development of shear strength equations for unsaturated soil. A comprehensive literature review of SWCC and mechanical behaviour of unsaturated soil was conducted. Various SWCC estimation methods were studied and reviewed in order to be used for selecting suitable soils for the entire research programme. All the controlling factors that affect the SWCC and the mechanical behaviour of unsaturated soils were studied. The macrostructure, mechanical behaviour and permeability characteristics of unsaturated soil on the drying and wetting paths were studied. Various existing shear strength equations were reviewed and evaluated using data from literatures. Subsequently, the equations for predicting unsaturated shear strength of soil under drying and wetting processes were developed. Data from literatures were used in the comparison between the proposed equations and various existing equations. In order to perform unsaturated permeability testing and to

accelerate the unsaturated triaxial testing using the same triaxial apparatus, the conventional triaxial apparatus was modified. Thermal sensors were installed inside the triaxial cell and next to the triaxial cell in order monitor the cell water and ambient temperatures, respectively, during the tests. Reconstituted kaolin and well-graded sand were used to minimize the heterogeneity and improve the permeability of soil specimens. Different designed sand-kaolin ratios were used in the SWCC prediction using the Soil Vision programme, in order to obtain the suitable soil mixtures for the entire research programme. Identical sand-kaolin specimens were prepared at the specified water content and dry density using the static compaction method. Basic properties tests, saturated triaxial test and saturated permeability test were conducted. The SWCC tests of different soils were conducted using Tempe cell, modified Tempe cell and pressure plate under zero net confining pressure and using the modified triaxial test under a net confining pressure. Unsaturated Consolidated Drained (CD) triaxial test and unsaturated permeability test under single and multi-cycles of drying and wetting processes were conducted using the modified triaxial apparatus in order to investigate the effects of hysteresis on the characteristics of shear strength, volume change and permeability of sand-kaolin mixtures. At the end, the experimental results were analyzed and compared with the predicted results from the proposed equations and existing equations.

1.5 OUTLINE OF THESIS

This thesis was organized into seven chapters. The seven chapters are as follows:

Chapter 1 includes the research background, objectives, scopes, methodology and outline of this report.

Chapter 2 presents a review of unsaturated soil mechanics and examines the available literature on the characteristics of SWCC, SWCC equations and hysteresis of unsaturated soil. The review of shear strength characteristics under drying and wetting processes as well as various shear strength equations are described. The brief reviews on unsaturated permeability of soil and the laboratory testing are then presented.

Chapter 3 presents the applicable theories and the theoretical development of shear strength equations for unsaturated soils. The applicable theories include the grain-size distribution models, SWCC equations, permeability functions, determination of unsaturated permeability, extended Mohr-Coulomb failure envelope and stress-point envelope, stress path of the CD triaxial test and various published shear strength equations. The comparison of various published shear strength equations is presented. The development of the proposed shear strength equations is given at the end of this chapter.

Chapter 4 describes the outline of the research programme, criteria for the soil selection, various basic properties tests, soil preparation as well as saturated triaxial and permeability tests. Subsequently, the descriptions of the SWCC test apparatus as well as the assembled parts, modification and layout of the modified triaxial apparatus are presented. The testing procedures as well as the experimental programme are included at the end of this chapter.

Chapter 5 presents the results of the soil selection and the characteristics of the soil mixtures and sands. Subsequently, the single and multi-cycled SWCCs of the sands and sand-kaolin mixtures are presented. The CD triaxial tests and permeability tests on single and multi-cycled drying and wetting paths are presented in the following sections.

Chapter 6 contains the discussions of the results presented in Chapter 5. The discussions on the results of multi-cycled SWCCs, saturated triaxial tests and unsaturated triaxial tests are presented. The shear strength and volume change during shearing of sand-kaolin specimens under single and multi-cycled drying and wetting processes are discussed. Subsequently, the unsaturated permeability of sand-kaolin mixtures under multi-cycled drying and wetting processes are discussed. The comparisons between the predicted results and the experimental results are also given in this chapter. The discussions on the shear strength predictions from the proposed equations are presented in this chapter.

Chapter 7 gives the conclusions of this research. Further research recommendations are also given in this chapter.

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

In this chapter, the literature review related to the proposed research is presented. A review of the basic concept of unsaturated soil mechanics is given at first. Subsequently, influencing factors of SWCC, SWCC measurements, SWCC fitting equations, SWCC estimation equations, shear strength characteristics of unsaturated soil under the drying and wetting processes and shear strength equations for unsaturated soil are presented in the following sections. Lastly, some unsaturated testing and testing considerations are reviewed.

2.2 UNSATURATED SOIL MECHANICS

2.2.1 Unsaturated Soil in Nature

Climates affect the soil condition near the ground surface, especially in tropical countries like Singapore. Whether a soil is in saturated or unsaturated condition is directly affected and controlled by the periodical changes in climate as well as the variation of ground water table. Basically, soils below ground water table are in saturated condition with positive pore-water pressure while soils above ground water table are in unsaturated condition with negative-pore water pressure (matric suction).

As shown in Figure 2-1, the unsaturated soil zone is situated above ground water table and exposes to atmosphere. Unsaturated soil zone is named as vadose zone in unsaturated soil mechanics (Bouwer, 1978). Water is removed from a soil by either evaporation from soil surface or evapo-transpiration from vegetation. These processes result in an upward flux of water in soil which causes a gradual drying, cracking and desaturation of soil. Line 2 in Figure 2-2 shows the pore-water pressure profile corresponding to an upward flux. On the other hand, water is supplied into a soil by precipitation or rainfall, creating a downward flux of water which results in increasing the degree of saturation and pore-water pressure of the soil (line 3). Thus, the pore-water pressure condition in a soil is directly related to these two flux boundary conditions on a local scale. The depth of the

groundwater table is influenced by the net surface flux. Meanwhile, the equilibrium condition is reached when there is no flux across the unsaturated soil layer, which is represented by the hydrostatic line (line 1) as shown in Figure 2-2.

Figure 2-1: Illustration of the natural hydrology cycle and the role of environment on unsaturated soil in nature (Lu and Likos, 2004).

Figure 2-2: Illustration of the vadose zone (unsaturated soil zone) in soil layer (Fredlund and Rahardjo, 1993a).

2.2.2 Osmotic Suction and Matric suction

In the early 1900's, the theoretical concept of soil suction was developed in soil physics (Edlefsen and Anderson, 1943). Soil suction is quantified in terms of the relative humidity (Fredlund and Rahardjo, 1993a) and is commonly referred to as total suction. It consists of two components, osmotic and matric suctions (Fredlund and Rahardjo, 1993a).

Osmotic suction is associated with the dissolved salts in pore-water of a soil. The change of salt concentration results in changes in osmotic suction of a soil. However, compared to matric suction, osmotic suction plays a less important role in soil suction since its changes are less significant, less frequent and slower than matric suction changes. Osmotic suction could be ignored in practice as long as the soil solution is dilute enough and the contained solutes do not affect the matric suction significantly (Hillel, 1998). Besides, most of the unsaturated laboratory testings involve matric suction because most of the engineering problems in unsaturated soil are the results of environmental changes which cause matric suction changes (Fredlund and Rahardjo, 1993a).

The negative pore-water pressure in the vadose zone is named as matric suction or matric potential or soil-water suction. In this research, matric suction is used to represent the negative pore-water pressure of soil. Aitchison (1965) defined matric suction as “the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with the soil water, relative to the partial pressure of the water vapour in equilibrium with a solution identical in composition with the soil water”. According to the International Society of Soil Science (ISSS), “matric suction is defined as the negative gauge pressure, relative to the external gas pressure on the soil water, to which a solution identical in composition with the soil solution must be subjected in order to be in equilibrium through a porous membrane wall with the water in the soil” as stated by Hillel (1998). Alternatively, matric suction, $(u_a - u_w)$, is the difference between pore-air pressure, u_a , and pore-water pressure, u_w , that acts across the contractile skin (Fredlund and Rahardjo, 1993a). Thus, the pore-water pressure usually determines the value of matric suction since pore-air pressure (atmospheric pressure, or assumed as zero pressure in the field) is constant.

During a rainy season or a wetting period, pore-water pressure increases and matric suction decreases due to an increase in degree of saturation of soil. In contrast, during hot and drying period, pore-water pressure decreases or matric suction increases due to the desaturation of soil. The matric suction in a soil near the ground surface is more affected significantly by the changes in the climatic condition than the soils at the greater depths.

Figure 2-3: Rise of water in capillary tubes of various sizes at hydrostatic equilibrium.

Matric suction is associated with the capillary effects which arise from the surface tension of water and its contact angle with the solid particles (Fredlund and Rahardjo, 1993a). Surface tension is the result of the intermolecular forces which act on the molecules in the contractile skin (Fredlund and Rahardjo, 1993a). The capillary phenomenon can be illustrated using the rise of water in the capillary tubes as shown in Figure 2-3.

The pores in a soil are like a bundle of capillary tubes of different sizes. The capillary effects cause water to rise above the water table in a capillary tube. The smaller the radius of a tube, the higher the water rise above the water table will be. The capillary water has a negative pressure with respect to the air pressure which is atmospheric pressure (i.e. $u_a = 0$). Similarly, the capillary effects in the pores of a soil result in the negative pressure (matric suction) and cause the soil water to rise above the ground water table. The matric suction in an unsaturated soil can be ranged from zero (under fully saturated condition) to 1 GPa (under dry condition) (Fredlund, 2006).

2.2.3 Four-Phase Mixture in Unsaturated Soils

An unsaturated soil is commonly referred to as a three-phase system (i.e. soil solids, air and water). However, there are now strong justifications to include the fourth independent phase, contractile skin or air-water interface into the soil system. The contractile skin is like a very thin membrane, which is in the order of 1.5 – 2 water molecules diameter (Israelachvili, 1991; Townsend and Rice, 1991) interwoven throughout the voids of the soil and acts as a partition between air and water phases. Thus, an unsaturated soil has air and water phases that flow under the influence of stress gradient. The contractile skin is known to play an important role in unsaturated soil behavior (Terzaghi, 1943).

Figure 2-4: Idealized an unsaturated soil element with continuous air phase (from Fredlund and Rahardjo, 1993a).

Contractile skin properties are different from ordinary water properties. The water molecular structure in contractile skin is similar to the water molecular structure in ice (Matsumoto and Kataoka, 1988). The interaction between the contractile skin and the soil particles affects the mechanical behavior of unsaturated soil. The unsaturated soil properties change according to the position of the contractile skin (i.e. water degree of saturation) (Fredlund, 2006). Figure 2-4 illustrates the idealized four-phase mixture in unsaturated soil with a continuous air phase. The mass and volume of each phase is schematically described in a phase diagram as shown in Figure 2-5.

Although the contractile skin has different physical properties from the contiguous air and water phases, it can be assumed as a part of the water phase without any significant error when considering volume-mass soil properties. However, it must be recognized as an

independent phase when considering the stress analysis of a multiphase continuum and the phenomenological behaviour of an unsaturated soil (Fredlund, 2006). The simplified three-phase diagram of unsaturated soil is shown in Figure 2-6.

Figure 2-5: Schematic phase diagram of four-phase unsaturated soil system.

Figure 2-6: Simplified phase diagram for an unsaturated soil.

2.2.4 Stress State Variables

The single-valued effective stress, $(\sigma - u_w)$, for saturated soil has often been recognized as a physical law. The effective stress is the stress state variable which is independent of soil properties, used to describe the mechanical behavior of saturated soil. It had led to similar formulation for unsaturated soil using a single-valued effective stress or one stress

state variable. The incorporation of soil properties in all the proposed equations in the description of a stress state variable has lead to difficulties in the application of single-valued effective stress in unsaturated soil mechanics.

Biot (1941) was probably the first researcher who suggested the need of two independent stress state variables to describe unsaturated soil behavior. Other researchers like Bishop and Blight (1963) and Matyas and Radakrishna (1968) began to recognize the need of using two independent stress state variables for an unsaturated soil. Fung (1977) suggested that the variables used for the description of a stress state should be independent of soil properties. Theoretical basis and justifications were provided to use two independent stress state variables for an unsaturated soil by Fredlund and Morgenstern (1977). The theoretical basis and justifications were based on the multiphase continuum mechanics and the incorporation of contractile skin which was considered as an additional and independent phase of unsaturated soil. A four-phase system of unsaturated soil was considered and the soil particles were assumed to be incompressible and chemically inert. These assumptions are consistent with the assumptions in saturated soil mechanics.

Fredlund and Morgenstern (1977) proposed three possible combinations of stress state variables which can be used to describe the stress state of an unsaturated soil. The three combinations are: $(\sigma - u_a)$ and $(u_a - u_w)$, $(\sigma - u_w)$ and $(u_a - u_w)$ as well as $(\sigma - u_a)$ and $(\sigma - u_w)$, where σ is total stress, u_a is the pore-air pressure and u_w is the pore-water pressure. The stress state variables of an unsaturated soil form two independent stress tensors in a three-dimensional stress analysis. All the proposed stress state variables have been experimentally tested and proven (Fredlund and Rahardjo, 1993a). Among the three combinations, the first combination of stress state variables is the most suitable and is always chosen for engineering application. This is due to the fact that the effects of total normal stress changes can be separated from the effects caused by pore-water pressure changes (Fredlund, 1979; Fredlund and Rahardjo, 1987). Therefore, this set of stress state variables is used to characterize the volume change behavior and shear strength behavior of an unsaturated soil (Fredlund and Rahardjo, 1993a) and is used throughout this research.

2.3 SOIL-WATER CHARACTERISTIC CURVE

Soil-water Characteristic Curve (SWCC), typically a sigmoid, is a graphical relationship between the amount of water in a soil and the soil suction. It is also referred as water retention curve, moisture retention curve, soil-moisture retention curve, soil-water release curve or soil-moisture characteristic curve in soil physics. In civil engineering related discipline, the term, SWCC, is recommended to be used to represent the relationship between the amount of water and the matric suction of a soil (Fredlund, 2002).

SWCC plays an important role in the implementation of unsaturated soil mechanics (Fredlund, 2002). It is the primary information of an unsaturated soil as many properties of an unsaturated soil such as shear strength, coefficient of permeability, pore-size distribution, particle size distribution and water content of soil at given matric suction can be related to or obtained from the SWCC directly or indirectly. SWCC is defined as the relationship between volumetric water content, θ_w , and matric suction, $(u_a - u_w)$, in logarithmic term. However, it can also be defined as the relationship between gravimetric water content, w , or degree of saturation, S , and matric suction in many circumstances. All three designations give similar information when the volume change of a soil is negligible or when a soil does not undergo volume change. In addition, normalized water content is also used in SWCC in order to isolate the physical behaviour of a soil from the saturated condition to the residual condition. SWCC of a soil is not unique but is affected by many factors as discussed in Section 2.3.1.

A complete SWCC consists of drying and wetting curves. In general, drying and wetting SWCCs are different. The differences in the drying and wetting SWCCs can be explained with hysteresis mechanisms in soil-water interaction as presented in Section 2.4. The drying curve represents the water desorption of a soil when the matric suction increases while the wetting curve represents the water absorption of a soil when the matric suction decreases. However, most of the existing SWCC data include the drying curve only as it is easier to be measured (Fredlund, 2006) as compared to the wetting curve. The measurement of wetting curve is a troublesome and time-consuming task. Figure 2-7 shows an example of a complete SWCC with best-fit curves of compacted coarse kaolin.

Figure 2-7: A complete SWCC of compacted coarse kaolin as obtained from pressure plate test (Meilani et al., 2005).

An analogous curve to the SWCC of soil in a saturated soil is the consolidation curve (Fredlund and Rahardjo, 1993a). When the stress is increasing, the soil is compressed and the water content of the soil is reduced. On the other hand, when the stress is decreasing, the soil swells and the water content of the soil is increased. Therefore, the compression and swelling lines of consolidation curve are similar to the drying and wetting SWCCs, respectively.

A comparison study on saturated consolidation curve and SWCC was conducted by Blight (2007). Blight (2007) found that the SWCC of soil is closely related to the saturated consolidation curve of soil as both curves change in a similar pattern (see Figure 2-8(a) and Figure 2-8(b)). Figure 2-8(a) shows the comparison between the drying SWCC and the consolidation curve of OFS clay. It was found that there is no difference between the drying SWCC and the consolidation curve at low matric suction range (lower than AEV) or low stress range since the soil remained in saturated condition (see Figure 2-8(a)). Therefore, the soil behaviour at matric suction lower than AEV is similar to the saturated soil behaviour as suggested in various studies, e.g. Gan et al. (1988), Vanapalli et al. (1996b), Khalili and Khabbaz (1998), Houston et al. (2008) etc.

Figure 2-8: (a) Drying SWCC and the consolidation curve of OFS clay; (b) Comparison between saturated consolidation curve and SWCC of Claren silty clay (Blight, 2007).

Figure 2-9: Illustration of the desaturation zones in a SWCC (Vanapalli et al., 1999).

White et al. (1970) divided the desaturation process of soil into different stages. Later, Vanapalli et al. (1996b) modified the concepts from White et al. (1970) and identified the desaturation process of soil in SWCC into three different saturation stages, i.e. boundary effect stage, transition stage and residual stage, as shown in Figure 2-9. When a soil is in boundary effect stage, all the pores in the soil are still filled with water and the soil is assumed to be in saturated condition. Desaturation of the soil is assumed to start at AEV

(the point which air starts to enter the largest pores of the soil) which is the starting point of the transition stage. The water content of the soil reduces rapidly and significantly with increasing matric suction in this stage. In residual stage, the soil pores are mainly occupied by air and the water phase becomes discontinuous. A small change in the water content of the soil although a large increase in matric suction is applied in residual stage.

2.3.1 Factors Affecting Soil-Water Characteristic Curve

Factors such as grain-size distribution, pore-size distribution, soil specimen preparation method, SWCC test method, soil types, soil contents, mineralogy, dry density, stress state (history), soil state, initial water content and soil structure affect the shape of SWCC. Hillel (1998) mentioned that the direction and the rate of change of water would also affect this relationship. It is important to study and understand these factors before using SWCC information for describing unsaturated soil behaviour or using SWCC for unsaturated soil models or equations such as shear strength equations, unsaturated permeability functions and volume change models etc.

Figure 2-10: Typical SWCCs for sandy, silty soil and clayey soils (Fredlund and Xing, 1994).

SWCC is controlled by the soil type and grain-size distribution of a soil. The slope, water content and AEV etc of the SWCC of a soil change significantly from gravelly soil to clayey soil. Typical SWCCs for clayey soil, silty soil and sandy soil are illustrated in

Figure 2-10. Generally, a soil with a higher fines content, e.g. clayey soil, has a higher initial water content, a higher AEV, a higher water content at a given matric suction, a higher residual matric suction and a gentler slope of the curve than those with a lower fines content (Fredlund and Xing, 1994; Hillel, 1998; Vanapalli et al., 1999; Yang et al., 2004).

Lu et al. (2007) conducted a study on the relationship between the plasticity of soil and the water retention behaviour of soil. It was concluded that the higher the plasticity index of a soil, the higher the water retention ability and the lower the desaturation rate of the soil will be. This is mainly due to the plasticity index of a soil is closely related to the fine particle in the soil. Vanapalli and Fredlund (2000) re-analysed the SWCCs of statically compacted Madrid grey clay, Red clay of Guadalix de la Sierra and Madrid clayey sand from Escario and Juca (1989) testing results as shown Figure 2-28. Vanapalli et al. (1998) concluded that the AEV and the residual matric suction of a soil increases with an increase in the percentage of fine soil particles present in the soil.

Figure 2-11: Grain-size distribution of different soils (a) five different soils from Yang et al. (2004) study; (b) four different soils from Gallage and Uchimura (2010) study.

Yang et al. (2004) conducted a SWCC study on five different soil types. Later, Gallage and Uchimura (2010) conducted a similar SWCC study on four different soils. The grain-size distributions of different soils used in both studies are shown in Figure 2-11 (a) and (b). As shown in Figure 2-12 (a), (b), (c) and (d), the drying and wetting SWCCs of are significantly different. AEV, residual matric suction, and residual volumetric water content of soil are inversely proportional to fines content in soil. The higher the D_{10} of a

soil, the lower the AEV, the residual matric suction and the residual volumetric water content of the soil will be (Aubertin et al., 1998; Yang et al., 2004, Gallage and 2010). In addition, the shape of SWCC is closely related to the shape of grain-size distribution curve. A steep slope of the grain-size distribution curve of a soil results in a steep slope of the SWCC. The SWCC of a uniform soil has a steeper slope than that of a less uniform soil (Yang et al., 2004; Gallage and Uchimura, 2010).

Yang et al. (2004) and Gallage and Uchimura (2010) stated that the hysteresis of a soil is closely related to the D_{10} and the grain-size distribution of the soil as shown in Figure 2-13 (a) and (b) as well as Figure 2-14 (a), (b) and (c). Both studies showed that the hysteresis of a soil is inversely proportional to the D_{10} of the soil. Besides, the steeper the slope of the grain-size distribution of a soil, the smaller the hysteresis of the soil will be. In addition, Gallage and Uchimura (2010) found that the hysteresis of the soil with higher initial dry density is lesser than that of the soil with lower initial dry density as shown in Figure 2-14 (c).

Figure 2-12: Soil-water characteristic curve of (a) gravelly sand; (b) Medium sand; (c) fine sand; (d) clayey sand I (Yang et al., 2004).

Figure 2-13: Relationship between total hysteresis and (a) D_{10} ; (b) slope of grain size distribution (Yang et al., 2004).

Figure 2-14: Relationship between total hysteresis and (a) slope of grain size distribution; (b) D_{10} ; (c) initial dry density (Gallage and Uchimura, 2010).

Initial moulding water content of a soil determines the shape of SWCC (Vanapalli et al., 1999). The specimen that was compacted at dry of optimum (lower initial water content) has a lower AEV and a lower degree of saturation at a given matric suction than those at wet of optimum (higher initial water content) as shown in Figure 2-15. The lower the initial water content of a soil (from wet optimum to dry optimum), the steeper the SWCC of a soil will be (Vanapalli et al., 1999). In addition, the SWCC behaviour of specimens compacted at optimum water content behaves in between the specimens compacted at dry and wet optimum. It was explained that the arrangement of soil particles for specimens compacted at dry of optimum is flocculated and thus, the macrostructure governs the SWCC behaviour of the soil. The macropores provide an easier drainage of water under an applied matric suction (Vanapalli et al., 1996a). The arrangement of soil aggregates is referred to macrostructure while microstructure is the arrangement of the associations of elementary particle within soil aggregate (Mitchell, 1976). On the other hand, the

arrangement of soil particles in the wet of optimum specimen is more homogeneous and well-arranged than those in the dry of optimum (Elsbury et al., 1990; Vanapalli et al., 1996a). The pore channels in the wet of optimum specimen are generally disconnected where the microstructure is more predominant in governing the SWCC behaviour and the micropores are more impervious. As a result, the specimen at wet of optimum is likely to be more resistance to desaturation than those at dry of optimum. However, the influence of the initial moulding water content of a soil becomes negligible at high matric suction range, i.e. higher than residual matric suction, as the SWCC behavior in this range is not affected by the soil structure (Tinjum et al., 1997; Vanapalli et al., 1999; Tarantino and Tombolato, 2005).

Figure 2-15: SWCCs for soil specimens compacted at different initial water contents (Vanapalli et al., 1996a).

The initial dry density or initial void ratio of a soil affects its SWCC. Aubertin et al. (1998), Kawai et al. (2000), Yang et al. (2004), Sun et al. (2006) and Gallage and Uchimura (2010) conducted SWCC comparisons on the soil specimens using the same soil content with different initial dry densities. All studies showed that the soil specimen with a higher initial dry density has a higher AEV, higher water-entry value and higher volumetric water content at a given matric suction higher than AEV than that with a lower dry density, as shown in Figure 2-16, Figure 2-17a, Figure 2-17b and Figure 2-17c. A soil with a higher dry density has a lower porosity, a lower void ratio and smaller average pore sizes than that with a lower dry density. The soil with smaller average pore sizes is more likely to retain water in the pores (macropores) at higher matric suction than that with larger average pore sizes since a higher suction is required for air to enter into

the pores of the soil. Sun et al. (2006) suggests that the SWCC shifts to the right when the specimen becomes denser (i.e. AEV and residual matric suction at higher value).

Figure 2-16: Drying SWCCs for (a) clayey sand (Yang et al., 2004) and (b) Edosaki sand (Gallage and Uchimura, 2010) under different dry densities

Figure 2-17: Relationship between (a) air-entry value and void ratio; (b) water-entry value and void ratio; (c) residual degree of saturation and air-entry value (Kawai et al., 2000).

SWCC of a soil is affected by the confinement on the soil (Vanapalli et al., 1996b; Ng and Pang, 2000; Lee et al., 2005; Thu et al., 2007). The SWCC of soils under different net confining pressures (or net normal stresses) were investigated in various studies. Vanapalli et al. (1996a) showed that the soil under a higher net normal stress has a higher AEV than the soil under a lower net normal stress (see Figure 2-29(a) and Figure 2-29(b)). Ng and Pang (2000) indicated that the specimen under a higher applied load has a gentler SWCC slope and a higher AEV than those under a lower applied load (see Figure 2-18). Lee et al. (2005) found that as the net confining pressure increases, the AEV increases linearly, the water content at a given matric suction increases and the slope of SWCC beyond AEV becomes more flatten (see Figure 2-19). Later, Thu et al. (2007) showed that the AEV increases with the increase in net confining pressure (see Figure 2-20). These findings are generally due to the presence of a lower void ratio with a smaller average pore sizes in the soil under a higher net confining pressure, results in the soil has a higher ability to retain water at a given matric suction as compared to the soil under a lower net confining pressure.

In addition, significant hysteresis was found in all specimens under different net normal stresses in Ng and Pang (2000) study. However, it was reported that there is no significant difference in the hysteresis loops of specimens under different net normal stresses (Ng and Pang, 2000).

Figure 2-18: SWCCs under different stress state (Ng and Pang, 2000).

Figure 2-19: SWCCs of Korea weathered granitic residual soil under different net confining stress in, (a) volumetric water content; (b) degree of saturation (Lee et al., 2005).

Figure 2-20: Relationship between AEV and net confining stress of SWCC (Thu et al., 2007).

SWCC also differs depending on the historical stresses that the soil specimen experienced (Fredlund, 2002). Typically, there is a significant difference between the gravimetric SWCC of an initially slurried soil specimen and a compacted soil specimen as shown in Figure 2-21. The initially slurried soil specimen especially for clayey soil specimen experiences a large volume change during drying and wetting processes while the compacted soil specimen has a relatively low deformability during the drying and wetting processes (Fredlund, 2002).

Figure 2-21: SWCCs of soil specimens prepared in different manners (Fredlund, 2002).

2.3.2 Measurement of Soil-Water Characteristic Curve

The costs for measuring the unsaturated soil properties and behaviour are excessive which could be ten times as much as the cost and time of measuring the saturated soil properties. Therefore, a number of new procedures, equations and models which involve the use of SWCC and saturated soil properties to estimate the unsaturated soil property functions have emerged in the past few decades.

The measurement of SWCC has been proven to be the most important test that is required for applying unsaturated soil mechanics into geotechnical engineering practice (Fredlund, 2006). There are a number of devices that have been developed to measure the SWCC over a wide range of matric suction values. The typical apparatuses for measuring SWCC are Tempe cells which can apply up to 100kPa of matric suction and pressure plate

extractor which can apply up to 1500kPa of matric suction. The measurement of the change in volume or mass of water is used to back-calculate the equilibrium water content at different matric suctions (Fredlund and Rahardjo, 1993a). A detailed description for several testing procedures that includes, hanging column, pressure plate extractor, chilled mirror hydrometer and centrifuge for the determination of SWCCs are provided in the ASTM designation (ASTM D-6836-02, 2003).

In addition, modified triaxial apparatus can also be used to obtain the SWCC of a soil. A net confining pressure, which is similar to the in-situ condition of a soil, can be applied to the soil specimen during the SWCC test. Therefore, SWCC results are closer or more similar to the real condition of the soil. Besides, both total volume change and water volume change can be measured continuously and accurately. This is relatively important for measuring the soil specimen, especially for clayey soil specimen, which experiences a significant total volume change during drying and wetting processes.

2.3.3 Estimation of Soil-Water Characteristic Curve

The measurement of the SWCC of a soil is a time-consuming task, especially for clayey soil. The costs for obtaining the unsaturated soil properties can be further reduced if the SWCC can be estimated from the soil properties such as grain-size distribution curve (Fredlund, 2000). SWCC estimation functions are termed as Pedo-Transfer Function (PTF). Bouma (1989) defined PTF as a function that relates to basic soil data such as the grain-size distribution or porosity and yields a soil property function.

There are a number of methods that were introduced for SWCC estimation in the past. SWCC estimation methods can be divided into three broad categories, Point Regression Method, Functional Parameter Regression Method and Physico-Empirical Method.

2.3.3.1 Point Regression Method

This method involves the statistically estimation of water contents of soil at various matric suctions for the SWCC. There is no assumption on the shape of SWCC is made in this type of method. The estimated SWCC of a soil is mainly based on the statistics of the

existing collected soil information data. An example of this SWCC estimation method is Gupta and Larson model (1979).

2.3.3.2 Functional Parameter Regression Method

The parameters of the SWCC equations are assumed to be correlated to the basic physical properties of a soil. The correlations between the air-entry parameter of a SWCC equation and the soil properties like porosity, dry density or percentage of sand are examples of functional parameter regression method. Rawls and Brakensiek model (1985) estimates the parameters such as air-entry value, pore-size index and residual water content for Brooks and Corey (1964) equation using basic soil properties. It was found that most of the estimated desaturation rate of a soil from Rawls and Brakensiek model (1985) was overestimated (Fredlund et al., 2002).

Vereecken et al. (1989) model uses a statistical regression analysis to estimate the parameters for the van Genuchten (1980) equation. This model involves a data set of forty Belgians soils that were fitted with the van Genuchten (1980) equation. This model was shown to be able to estimate SWCC with reasonable agreement using soil properties such as grain-size distribution, dry density and carbon content (Vereecken et al. 1989, Fredlund et al, 2002). Vereecken et al. (1989) model is mainly used in the agricultural discipline where organics soils are involved and the emphasis was on the water availability for plant growth (Fredlund et al., 2002).

Scheinost et al. (1997) model estimates the parameters for the van Gunuchten (1980) equation using a linear regression analysis. This model was developed in order to consider the soil parameters with extreme variations such as organic contents varying over a wide range, bulk density varying from 0.80 to 1.85 Mg/m³ and soil textures varying from gravels to clays (Fredlund et al., 2002). Scheinost et al. (1997) model was shown to be able to estimate the SWCC of most soils within reasonable accuracy (Scheinost et al., 1997, Fredlund et al., 2002).

2.3.3.3 Physico-Empirical Method

This method uses the physical characteristics of a soil, such as grain-size distribution for the SWCC estimation. Arya and Paris (1981) model was the first physico-empirical method to estimate SWCC of soil. It was originally developed from a small database and subsequently extrapolated to a larger database. This model requires a well-defined of full range grain-size distribution for SWCC estimation. The volumetric water content is calculated based on the pore-sizes. On the other hand, the pore radii which are estimated based on the assumptions that all pores are in cylindrical and all particles are in spherical are converted to equivalent matric suction using the capillary theory (Taylor, 1948). Arya and Paris (1981) assumed that grain-size distribution and pore-size distribution of a soil are closely related where the larger soil particles produce the larger soil interparticle. In Fredlund et al. (2002) study, Arya and Paris (1981) model was shown to provide the highest level of confidence in the AEV and the maximum slope estimation for the SWCC of a soil. Arya and Paris (1981) model was modified by Haverkamp and Parlange (1986) by incorporating the concept of shape similarity between the SWCC and the cumulative grain-size distribution for sandy soils into the method. Tyler and Wheatcraft (1989) modified Arya and Paris (1981) model and proposed a co-relationship between the fitting parameter, α of the Arya and Paris (1981) model and the physical properties of the soil.

Kovács (1981) proposed a physico-empirical method to estimate SWCC which assumes that a distinction exists between capillary and adhesive forces that both act simultaneously to induce suction. However, the Kovács model (1981) is not easily to be used in practical engineering since some of the key parameters that are needed were not well-defined. Aubertin et al. (1998) modified Kovács model (1981) and proposed the Modified Kovács model (MK model). MK model was successfully applied to predict the SWCC of tailing and silts (Aubertin et al. 1998). Aubertin et al. (2003) further modified the MK model to greater extend for general applications on various types of soils, from a sand to particular types of fine-grained sand. Modified MK model (Aubertin et al., (2003) model) incorporates the Kovács's (1981) assumption that the water retention (suction) is the result of the combined effect of capillary and adhesion forces. However, modified MK model is only applicable for homogeneous and isotropic materials under drying path. Volume change, anisotropy and microstructures of soil are ignored in this model.

Some other models were proposed to incorporate the estimation of the random packing nature of spherical particles in order to improve the estimation of pore size distribution of a heterogeneous structure (Itawa et al., 1994). Fredlund et al. (1997, 2002) proposed a model to estimate the SWCC of a soil based on the capillary theory (model), the grain-size distribution and the estimated initial packing factor of a soils. The initial packing factor which is the primary volume-mass variable used in the estimation of the SWCC is estimated using statistical method or the neural net. Fredlund et al. (1997) defined the neural net as “an artificial-intelligence technique by which an algorithm is trained to respond to various input conditions”. Unimodal and Bimodal equations (Fredlund et al. 1997, 2000) are used for best fitting the entire grain-size distribution curve. The grain-size distribution of a soil is divided into a number of small particle groups with relatively uniform particle sizes. It is assumed that there is a unique SWCC for each group of particle size and its initial condition is in a slurry condition. The individual SWCC of each group is combined using superposition technique to present the SWCC for the overall soil. Reasonable and reliable SWCCs for sands and silts were successfully estimated using Fredlund et al. (2002) model while the estimated SWCCs for clays, tills and loams were less accurate. This model has the limitations that it can only predict a likely SWCC of a soil which is initially prepared from a slurry paste. Besides, the fabric of the soil is not considered in the model.

2.3.3.4 Limitation of Pedo-Transfer Function

Although the SWCC estimation methods are attractive and useful, the limitations and assumptions of each method must be considered. There are a number of different methods that have been developed. Every method was developed using different derivations and based on different soil data bases. Therefore, there is a possibility that a method is not suitable for particular soil type. Besides, the assumptions that are made in the procedures of the estimation methods may cause improper estimation. For example, the physico-empirical models are based on the grain-size distribution curve in order to predict the SWCC. The grain-size distribution is obtained in laboratory and is assumed to represent the even distribution and homogeneous arrangement of different sizes of soil particles in a soil. The effects of density, stress history, confinement, fabric and hysteresis of a soil are

not considered in most of the estimation methods. Therefore, these assumptions may result in the estimated SWCC being different from the actual SWCC of a soil.

2.3.4 Best-Fit Equations for Soil-Water Characteristic Curve

To date, numerous closed form equations have been proposed to describe the SWCC. Generally, these empirical SWCC equations can be used to best-fit SWCC using a least square regression analysis. All the proposed equations have one variable that is related to the AEV of soil and one variable that is related to the desaturation rate of soil. If there is a third variable, it allows the low matric suction range which is near the AEV to have a shape that is independent of the high matric suction range which is near the residual matric suction (Fredlund, 2006).

Gardner (1958) equation is one of the earliest equations for best-fitting the SWCC. Originally, this continuous function was proposed for modeling the unsaturated coefficient of permeability of soil. However, this equation has been adapted for the SWCC of soil (Sillers et al. 2001). Two fitting parameters are used in this equation, where parameter “a” is related to the inverse of AEV while parameter “n” is related to the pore size distribution. Gardner (1958) equation is as follows,

$$\theta_w = \theta_r + \frac{\theta_s - \theta_r}{1 + a\psi^n} \quad \text{Equation 2-1}$$

where:

θ_w = volumetric water content

θ_r = residual volumetric water content

θ_s = saturated volumetric water content

ψ = matric suction of soil

Brooks and Corey (1964) assume constant water content for the range of matric suction less than AEV. For the range of matric suction higher than AEV, SWCC is assumed to decrease exponentially. Two fitting parameters, α and λ , are used in Brooks and Corey (1964) equations (Equation 2-2) Brooks and Corey (1964) equations are as follows,

$$\begin{aligned} \theta_w &= \theta_s && \text{when } \psi < a \\ \theta_w &= \theta_s \left(\frac{\psi}{a} \right)^{-\lambda} && \text{when } \psi > a \end{aligned} \quad \text{Equation 2-2}$$

where:

a = a fitting parameter which is related to the AEV of the soil.

λ = a fitting parameter which is termed as pore size distribution index.

The more uniform the pore-size distribution of a soil, the larger the value of λ and the steeper the SWCC slope of the soil within the desaturation zone. Equation 2-2 is more suitable to be used for relatively coarse particle soil. However, Equation 2-2 is not a continuous function for the entire SWCC as no inflection point is included in the equations. It always results in poor fitting of SWCC over a wide range of matric suction. The abrupt change in the curve may result in the numerical instability when modeling an unsaturated soil behaviour (Sillers et al., 2001; Lu and Likos, 2004).

van Genuchten (1980) equation is a continuous SWCC best-fit equation. This equation provides a wide range of flexibility in fitting the SWCC data of variety types of soil. Three fitting parameters, a , p and q , which have particular physical meanings are used. The best-fitted fitting parameter values may vary depending on the convergence procedure, number of iterations used and the initial values used for iterating the fitting parameters. van Genuchten (1980) equation is given as follows,

$$\theta_w = \theta_s \left[\frac{1}{1 + (a\psi)^p} \right]^q \quad \text{Equation 2-3}$$

where,

a = a fitting parameter which is related to the inverse of the AEV

p = a fitting parameter which is related to the pore-size distribution of the soil

q = a fitting parameter which is related to the asymmetry of the SWCC curve

Fredlund and Xing (1994) proposed a continuous and closed form SWCC best-fit equation. Fredlund and Xing (1994) equation has similar form as van Genuchten (1980)

equation. Three fitting parameters with particular physical meanings are used in Fredlund and Xing (1994) equation which has the following form,

$$\theta = C(\psi) \left[\frac{\theta_s}{\left[\ln \left[e + \left(\frac{\psi}{a} \right)^n \right] \right]^m} \right] \quad \text{Equation 2-4}$$

where:

a = a soil parameter which is primarily a function of the AEV of the soil (kPa)

n = a soil parameter which is primarily a function of the rate of water extraction from the soil once the AEV has been exceeded

m = a soil parameter which is primarily a function of the residual water content

e = natural number, 2.71828...

$C(\psi)$ = correction function that forces the SWCC through a matric suction of 1 GPa and zero water content

Correction factor, $C(\psi)$, was introduced by Fredlund and Xing (1994) to be used in Equation 2-4 to ensure that the water content is zero at matric suction of 1 GPa. The correction factor, $C(\psi)$, has the following form:

$$C(\psi) = 1 - \frac{\ln \left(1 + \frac{\psi}{\psi_r} \right)}{\ln \left(1 + \frac{10^6}{\psi_r} \right)} \quad \text{Equation 2-5}$$

where:

ψ_r = suction at which residual water content occurs (kPa)

Later, the correction factor, $C(\psi)$, in Equation 2-4 was suggested to be unity by Leong and Rahardjo (1997). With $C(\psi) = 1$, the initial portion of the SWCC, which is relatively important, is not affected by $C(\psi)$ (Leong and Rahardjo, 1997b). It is suitable to be used

for best-fitting the SWCC of soil at low matric suction range (i.e. less than 1500 kPa). Besides, the computational effort for determining the fitting parameters could be reduced.

Leong and Rahardjo (1997b) suggested that Fredlund and Xing (1994) equation (Equation 2-4) is the best SWCC best-fit equation for a wide range soils over the entire matric suction range. If compared with van Genuchten (1980) equation, Fredlund and Xing (1994) equation was found to require less iteration to converge towards the best-fit parameters (Sillers, 1997). Besides, the fitting parameters used in Fredlund and Xing (1994) equation have a certain physical meaning, i.e. a is closely related to AEV, n is closely related to pore-size distribution and the slope of the SWCC and m is closely related to residual matric suction (Leong and Rahardjo, 1997; Yang et al., 2004; Gallage and Uchimura, 2010). As the parameter, a , increases, the SWCC curve is shifted to a higher matric suction range without affecting the overall shape of SWCC. The soil with a more uniform pore-size distribution has a higher value of n and a steeper slope of SWCC than those with a less uniform pore-size distribution. The parameter, m , is lower in the SWCC with a higher residual water content.

Fredlund and Xing (1994) equation has the same limitations as van Genuchten (1980) equation in which the best-fitted fitting parameter values may vary depending on the convergence procedure, number of iterations that are used and the initial values that are used for the fitting parameters

2.4 HYSTERESIS OF UNSATURATED SOILS

In nature, soils are subjected to cyclic drying and wetting due to climatic conditions, i.e. dry and wet period. During drying of a soil, matric suction in the soil increases and the soil experiences desorption of water. During wetting of a soil, the soil experiences adsorption of water while matric suction increases. Soil-water relationship follows different paths during drying and wetting where the soil under drying has a higher water content than the soil under wetting at a given matric suction (Fredlund and Rahardjo, 1993a). The difference is referred as hysteresis. The magnitude of the difference is mainly dependent on the soil type. Generally, the difference becomes more significant when the fine (clay) content increases (Hillel, 1998). The hysteresis phenomenon of unsaturated soils basically can be explained using capillary theory or “ink bottle effect”.

When an empty capillary tube of radius, r , as shown in Figure 2-22(a), is placed in a water bath, the water inside the tube will rise above the water table until it reaches an equilibrium level. This corresponds to the wetting process of a soil. When a tube which is filled with water is placed in a water bath, the water inside the tube will drain out until it reaches an equilibrium level. This corresponds to the drying process of a soil. The equilibrium water level (h_c in Figure 2-22(a)) achieved through the drying and wetting processes are the same if a straight tube with constant radius is used. The maximum height of water retained in a capillary tube can be calculated using Equation 2-6 (Fredlund and Rahardjo, 1993a). From the equation, the capillary height is known to be inversely proportional to the opening radius of the capillary tube.

$$h_c = \frac{2T_s}{\rho_w g R_s} \quad \text{Equation 2-6}$$

where,

T_s = surface tension of water

ρ_w = density of water

g = gravitational acceleration

R_s = radius of curvature of the meniscus

Figure 2-22: Height and radius effects on capillarity (modified from Taylor, 1948).

On the other hand, when there is a bulb (expanded space) in the capillary tube, the water equilibrium level under the wetting and drying processes may be different. The opening radius of the capillary tube is the controlling factor in the development of capillary rise. The bulb with a radius of r_1 ($r_1 > r$) is empty in the wetting process (Figure 2-22(b)) while it is filled with water in the drying process (Figure 2-22(c)). The presence of the bulb at the mid-height of the capillary tube prevents water to rise beyond the base of the bulb in the wetting process (Figure 2-22(b)) as the opening (diameter) of the bulb, $2r_1$, is larger than the opening (diameter) of the straight tube, $2r$. However, the bulb and capillary tube that initially full with water (Figure 2-22(c)), are filled with water and reach the same equilibrium water level, h_c , as in the straight capillary tube (Figure 2-22(a)) under the drying process provided the opening radius of both capillary tubes are the same.

Using the same theoretical basis, the capillary rise development in a soil is affected by the pore size distribution of the soil. Water rises to the equilibrium water level, h_c , through the continuous soil pores (Figure 2-22d) whose radii are equal to or smaller than the radius of the straight capillary tube, r (Figure 2-22(a)). Water cannot rise within the large opening at the center of the soil column (Figure 2-22d) since the opening radius is larger than r . The capillary height may be higher than h_c if the height of the soil is extended and the radii of the pores are smaller than r .

The capillary theory can be applied to explain the hysteresis in SWCC of a soil in nature. The irregular shapes of pores and non-uniform pore sizes (similar to bulbs) in a soil results in the water content of soil at a given matric suction being different during the drying and wetting processes, as illustrated by capillary tubes in Figure 2-22(b) and Figure 2-22(c). In addition, another three specific mechanisms that cause hysteresis in the SWCC are as follows:

- The swelling and shrinking of “aged” soil, cause the changes of soil fabric, structure and pore size distribution that depend on the wetting and drying history of a soil (Hillel and Mottas, 1966).
- The contact-angle (between air-water and soil solids) effect (Hillel, 1971; Bear, 1979, Lu and Likos, 2004). The contact angle and the radius of curvature are greater for an advancing meniscus (during wetting) than those of a receding meniscus (during drying) as shown in Figure 2-23. Therefore, for a given water content, a soil tends to exhibit greater matric suction in desorption than in adsorption.
- The presence of entrapped air in a soil decreases the water content of a “newly wetted soil” (Hillel, 1971; Fredlund and Rahardjo, 1993a).

Figure 2-23: Illustration of the difference between drying and wetting contact angles of a water droplet on inclined surface (Lu and Likos, 2004).

2.5 SHEAR STRENGTH OF UNSATURATED SOIL

2.5.1 Shear Strength Characteristic of Unsaturated Soil

Very often, the conventional geotechnical designs, such as slope stability etc, are based on saturated soil mechanics. However, numerous literatures were reported that most of the slope failures occurred in unsaturated soil zone, where it is well above the groundwater table. These designs may not be realistic since the influence of matric suction which play an important role in slope stability, is being ignored. Therefore, it is essential to understand the shear strength of unsaturated soil since it is an important engineering property for addressing various geotechnical designs, especially in slope stability. There were a number of studies conducted in order to understand the shear strength characteristics of soil under an unsaturated condition. Some selected studies were reviewed and presented in the following sections, Sections 2.5.1.1 and 2.5.1.2.

The shear strength, τ , of an unsaturated soil depends on two independent stress-state variables, i.e. net normal stress, $(\sigma - u_a)$, and matric suction, $(u_a - u_w)$. Fredlund et al. (1978) introduced an equation to describe the shear strength of unsaturated soils in terms of the two stress state variables. Later, many research works were done on unsaturated shear strength and a number of shear strength equations using these two stress state variables for unsaturated soil were introduced. Some of the published equations were selected from literature are presented in Section 2.5.2.

Mohr circles in three-dimensional form, τ versus $(\sigma - u_a)$ versus $(u_a - u_w)$, are used to describe the failure envelope for unsaturated soil. This three-dimensional plot for unsaturated soil is an extension of the two dimensional Mohr-Coulomb failure envelope of saturated soil, with the additional third dimension axis for matric suction, as described in Section 3.5.1. The rate of change in shear strength, τ , with respect to net normal stress, $(\sigma - u_a)$, is expressed by ϕ' while the rate of change in shear strength, τ , with respect to matric suction, $(u_a - u_w)$, is characterized by ϕ^b .

The shear strength of an unsaturated soil can be determined using a modified direct shear apparatus, as described in Gan et al. (1988), or a modified triaxial apparatus in the laboratory as described in Fredlund and Rahardjo (1993a). Both types of apparatus were

modified from the conventional direct shear apparatus and saturated triaxial apparatus, respectively, by equipping it with a high air-entry ceramic disk as well as a pore-air pressure system for incorporating the axis translation technique in order to apply matric suction into soil.

Triaxial apparatus was used in this research to determine the shear strength of soil with the consideration of the advantages of Triaxial apparatus as compared with the direct shear apparatus. The advantages of using triaxial apparatus compared with using direct shear apparatus (Holtz and Kovacs, 1981) are as follows,

- There are less significant stress concentration in triaxial test specimen
- The failure plane can occur anywhere in triaxial test specimen
- The stress paths to failure can be controlled effectively in triaxial test
- There is no rotation of σ_1 and σ_3 if Mohr-Coulomb envelope is used.

Generally, there are two types of triaxial compression tests commonly used for unsaturated soil specimens, i.e. Consolidated Drained (CD) triaxial test and Constant Water (CW) triaxial test. The CD triaxial test refers to the test condition of a consolidated specimen where drained conditions are maintained for both pore-air and pore-water phases during shearing while the CW triaxial test refers to the test condition of a consolidated specimen where drained conditions are maintained for the pore-air phase but undrained conditions are maintained for the pore-water phase during shearing.

2.5.1.1 Previous studies on shear strength characteristic of soils at drying

Gan et al. (1988) conducted a series of multistage CD direct shear tests using the Indian Head glacial till at the matric suction range of 0 to 500 kPa. The experimental results show a non-linear failure envelope with respect to matric suction as illustrated in Figure 2-24. At low matric suctions (close to zero), ϕ^b is equal to ϕ' . The ϕ^b starts to decrease significantly in the matric suction range of 50 - 100 kPa and approximately equals to 7° at matric suctions higher than 300 kPa. The decrement of ϕ^b was explained by Gan et al. (1988) as a result of the cross-sectional area where the water pressure acts decreases when the soil becomes unsaturated. Thus, an increase in matric suction is less effective than an

increase in the normal stress in increasing the shear strength when soil becomes unsaturated (Gan et al., 1988).

Figure 2-24: Non-linear failure envelope for unsaturated Indian Head glacial till specimens, (a) failure envelope of the shear stress with respect to matric suction (b) variation of ϕ' with respect to matric suction (Gan et al., 1988).

Table 2-1: Comparisons of triaxial test data (Satija, 1978) and the analyses of experimental data from Satija (1978) by Fredlund and Ho (1982).

Soils	Types of test	CU saturated triaxial test data (Satija, 1978)		Analysis on test results on unsaturated specimens by Ho and Fredlund (1982)	
		c' (kPa)	ϕ' (°)	c' (kPa)	ϕ^b (°)
Low Density $\rho_d=1478$ kg/m^3	CD	7.8	29	20.3	12.6
	CW			11.3	16.5
High Density $\rho_d=1580$ kg/m^3	CD	7.8	28.5	7.3	16.2
	CW			15.5	22.6

Noted: ρ_d = dry density CD = consolidated drained CW = constant water content
Both low density and high density soil specimens had same water content, 22.25%

Satija (1978) conducted a series of CD and CW triaxial tests on compacted Dhanauri clay at low and high densities. Ho and Fredlund (1982) re-analysed and re-interpreted the CD and CW triaxial test results of Satija (1978) works and presented different values of c' and ϕ' by assuming that the failure surfaces were planer with respect to the independent stress state variables. However, the c' determined from the unsaturated CD and CW triaxial tests were not consistent with the c' determined from the saturated triaxial test. Table 2-1 shows the analyses from Ho and Fredlund (1982) and test results from Satija (1978) study.

Figure 2-25: Non-linearity in the failure envelope of the compacted Dhanauri clay at low density. (a) Failure envelope of the shear stress with respect to matric suction (b) Variation of ϕ^b with respect to matric suction (Satija, 1978; Fredlund et al., 1987).

Figure 2-26: Non-linearity in the failure envelope of the compacted Dhanauri clay at high density. (a) Failure envelope of the shear stress with respect to matric suction (b) Variation of ϕ^b with respect to matric suction (Satija, 1978; Fredlund et al., 1987).

Fredlund et al. (1987) re-analysed the shear strength data of Satija (1978) and re-plotted the non-linear failure envelopes on the shear strength with respect to matric suction from the CD and CW triaxial tests as shown in Figure 2-25 and Figure 2-26. The cohesion intercept, c , and ϕ^b , at zero matric suction were plotted using the c' and ϕ' obtained from

the saturated triaxial tests. Both Figure 2-25 and Figure 2-26 show a smooth curve and a smooth transition between the saturated and unsaturated condition of soils as well as a good agreement between the failure envelopes obtained from CD and CW triaxial tests.

Figure 2-27: Direct shear test results from Escario and Sáez (1986).

A series of CD direct shear tests was conducted by Escario and Sáez (1986) on three different types of statically compacted soil, i.e. Madrid grey clay, Red clay of Guadalix de la Sierra and Madrid clayey sand, which have different percentages of clay and plasticity indices. All the failure envelopes with respect to suction of each soil were found to be non-linear as shown in Figure 2-27. The ϕ^b decreases as the matric suction increases. Figure 2-27 shows that ϕ^b is larger than ϕ' for Guadalix Red clay and Madrid clayey sand while ϕ^b is smaller than ϕ' for Madrid clay.

Escario and Juca (1989) extended Escario and Sáez (1986) study by increasing the matric suction range from 1 MPa to 15 MPa. The statically compacted Madrid grey clay, Guadalix Red clay and Madrid clayey sand were tested with CD direct shear tests using the modified direct shear equipment as described by Escario (1980). The SWCCs of the three soils were conducted using the pressure plate apparatus. Figure 2-28 shows the SWCCs and the shear strength with respect to matric suction of the three soils. All the failure envelopes are non-linear. The ϕ^b decreases as the matric suction increases. As shown in Figure 2-28(c) and (d), ϕ^b values of Madrid grey clay and Madrid clayey sand decrease to negative values at the high matric suction range.

Figure 2-28: Direct shear test results from Escario and Juca (1989); (a) SWCC of Madrid grey clay, Guadalix Red clay and Madrid clayey sand from Escario and Juca (1989) (Vanapalli and Fredlund, 2000) (b) Variation of shear strength with respect to matric suction for Guadalix Red clay (Escario and Juca, 1989). (c) Variation of shear strength with respect to matric suction for Madrid grey clay (Escario and Juca, 1989). (d) Variation of shear strength with respect to matric suction for Madrid clayey sand (Escario and Juca, 1989).

Figure 2-29: Variation of SWCCs, shear strengths and ϕ^b with respect to matric suction under different net normal stresses for compacted Indian Head glacial till specimens tested with different initial water content. (a), (c) and (e) Testing results for specimens under 25kPa net normal stress. (b), (d) and (f) Testing results for specimens under 200kPa net normal stress (Vanapalli et al., 1996a).

Vanapalli et al. (1996a) conducted a series of multistage CD direct shear tests on statically compacted Indian Head glacial till. As shown in Figure 2-29(c) and Figure 2-29(d), the shear strength of the soil specimens at dry of optimum water content is the lowest while the shear strength of the soil specimens at wet of optimum water content is the highest. The shear strength of an unsaturated soil mainly depends on the shearing resistance mobilized along the interaggregate contacts (Barbour and Yang, 1992). Matric suction serves as the stress state variable, contributes to the shear strength of an unsaturated soil through the wetted surface of interaggregate contact points (Vanapalli et al., 1996a). Therefore, the specimens at wet of optimum water content have a higher shear

strength as more interaggregate contact area points are wetted than those at dry of optimum since the water content of the specimens at wet of optimum is higher than those at dry of optimum (Figure 2-29(a) and (b)). Figure 2-29(e) and Figure 2-29(f) show the variation of ϕ^b with respect to matric suction. As shown, the ϕ^b decreases significantly as the AEV is exceeded. All the shear strength envelopes are non-linear and related to the SWCCs of the soils. The rate of increase in the shear strength due the contribution of matric suction is closely related to the rate of desaturation characteristics of the soil as shown in SWCCs of the soils (Figure 2-29(a) and Figure 2-29(b)). Besides, Vanapalli et al. (1996a) found that there is no different in c' and ϕ' of a soil obtained from single stage direct shear test for different shearing displacement rates, different initial water contents and different initial densities. This conclusion is similar to the findings of Gibbs and Hilf (1955) and Lee and Haley (1968).

Figure 2-30: (a) Variation of effective internal friction angle at different matric suction. (b) Shear strength versus matric suction under various net confining pressures (Lee et al., 2005).

Lee et al. (2005) investigated the effects of net confining pressure on the shear strength of unsaturated soil. A series of CD triaxial tests were conducted on compacted Korea weathered granitic residual soil specimens. The shear strength increases as the net confining pressure increases at a given matric suction. The effective internal friction angle slightly increases as the net confining pressure increases as shown in Figure 2-30(a). On the other hand, non-linear failure envelopes of shear strength with respect to matric suction under different net confining pressures were obtained as shown in Figure

2-30(b). The increment in shear strength induced by matric suction under the higher net confining pressure was slightly more significant than those under the lower net confining pressure (Figure 2-30(b)). At a given matric suction, the degree of saturation (wetted contact area between soils and water) increases as the net confining pressure increases (see Figure 2-19). Therefore, the larger wetted contact area that matric suction acts at, results in the more significant shear strength increment under the higher net confining pressure.

Houston et al. (2008) investigated the shear strength and shear-induced volume change of unsaturated soils. Series of CD triaxial tests were performed on four different compacted soils, i.e. Price Club soil (CL-ML), Sheely clay (CL), ASU east (SM) and Yuma sand (SP), with plasticity index ranging from 0 to 17.4. The results generally show that the shear strength increases nonlinearly as the matric suction increases from AEV and within the range of suction values of interest. However, the clean sand exhibits an exemption which decreased shear strength at high values of suction. On the other hand, the study shows that the volume change behaviour of the specimens were dependent on both net normal stress and matric suction. The higher the value of matric suction and the lower the net confining pressure, the greater the tendency of the soil to dilate as shear stresses are increased (Houston et al., 2008) (see Figure 2-31). The dilative behaviour during shearing was more significant in the soils with low plasticity index (see Figure 2-31). The soils with high plasticity index tend to compress during shearing except at the highest values suction in the test series (Houston et al., 2008).

Figure 2-31: (a) Volume change measurements on Prince Club (CL-ML) soil at 20 kPa net normal stress; (b) Comparison of volume change behaviour for CL-ML and CL samples (Houston et al., 2008).

2.5.1.2 Previous studies on shear strength characteristics of soils at drying and wetting

The shear strength of soil on the wetting path is important in practical proposes as most of reported slope failures were induced by rainfall that generally induces a wetting process in soil. During rainfall, water content increases and matric suction decreases, resulting in the soil state to change from drying path to wetting path. The shear strength behaviour of soil changes from drying soil behaviour to wetting soil behaviour. Generally, the shear strength of soil on the drying path is higher than the shear strength of soil on the wetting path (Han et al., 1995; Nishimura and Fredlund, 2002; Melinda et al., 2004; Rahardjo et al., 2004; Thu et al., 2006; Tse and Ng, 2008; Goh et al., 2010).

Figure 2-32: Relationship between shear strength and matric suction of Bukit Timah Granitic residual soils, (a) at wetting; (b) at drying (Han, 1996).

Han et al. (1995) conducted a series of direct shear tests on statically compacted Bukit Timah Granitic residual soil. The soil specimens at wetting were achieved by applying designated matric suctions lower than the as-compacted matric suction while the soil specimens at drying were prepared by saturating the specimens in the beginning and then subjected to targeted matric suctions for testing. Han et al. (1995) found that the shear strength of soil specimens were affected by hysteresis, where the specimens at wetting

had a lower shear strength and exhibited brittleness while the soils at drying had a higher shear strength and exhibited ductility (see Figure 2-32) (Han et al., 1995). Han et al. (1995) suggested that the interparticle forces in the soil are dependent on the matric suction as well as the contact area of water with soil particles. Soil with a lower water content has a smaller contact area between soil particles and water as compared to the soil with a higher water content, causing the soil at wetting has lower shear strength.

Nishimura and Fredlund (2002) found that the compressive strength of a desiccated soil on the drying path was higher than that of the desiccated soil on the wetting path. This finding was explained to be related to the area of contact of water in the soil, which is related to the drying and wetting history that the soil experienced. Melinda et al. (2004) found the same findings as Han et al. (1995) study, which described that the shear strength of soil on the drying path was higher than that of the soil on the wetting path in the study. It was suggested that the soil specimens on the wetting path behave similar to an over-consolidated soil that dilates during shearing and exhibits a brittle behaviour while the soil specimens on the drying path behave similar to a normally consolidated soil that contracts during shearing and exhibits a ductile behavior (Melinda et al., 2004; Goh et al., 2010).

The shear strength of compacted Jurong sedimentary formation residual soil and compacted kaolin obtained from CD triaxial tests (similar to drying path) were found to be higher than those obtained from CW triaxial tests (similar to wetting path) in Rahardjo et al. (2004) and Thu et al. (2006) studies, respectively. The saturated soil parameters, ϕ' and c' , from both CD and CW triaxial tests were reported to be identical (Thu et al., 2006). Both studies showed that the ϕ^b from the CD and CW triaxial tests were the same as the ϕ' at the range of matric suction lower than AEV. The cohesion intercept, c , and ϕ^b from both CD and CW triaxial tests started to differ when matric suction exceeded the AEV of soil. Rahardjo et al. (2004) suggested that the differences in the shear strength behaviour of a soil on the drying and wetting paths could be attributed to the hysteresis of the soil, i.e. the degree of saturation of soil at the end of shearing in the CD triaxial tests was higher than those from CW triaxial test at a given matric suction. The degree of saturation of a soil could represent the area of the pore-water in contact with the soil particles, which contributes to the increase in shear strength of the soils when subjected to matric suction

(Rahardjo et al., 2004). On the other hand, the c and ϕ^b were found to be the same for both CD and CW triaxial tests for matric suctions higher than the residual matric suction. Rahardjo et al. (2004) and Thu et al. (2006) suggested that this is due to the fact that hysteresis would become negligible at matric suction higher than the residual matric suction. Figure 2-33(a) and Figure 2-33(b) show the non-linearity in the relationships between cohesion intercept and matric suction as well as ϕ^b and matric suction.

Figure 2-33: (a) Variation of cohesion intercepts of the failure envelope on the zero net confining stress plane for CD and CW triaxial tests. (b) Nonlinear relationship between ϕ^b and matric suction for the CW and CD triaxial tests (Thu, 2005; Thu et al., 2006).

From the direct shear test result of completely decomposed tuff (CDT) specimens, Tse and Ng (2008) found that the shear strength of CDT on the first drying path was higher than those on the first wetting path and those on the second drying path. However, the shear strength of CDT on the first wetting path was found to be lower than the shear strength of CDT on the second wetting path.

Later, Goh et al. (2010) conducted a series of CD triaxial tests on compacted sand-kaolin specimens to study the shearing behaviour of soil on the drying and wetting paths. The study indicated that the soils on the wetting path had a lower shear strength and exhibited higher stiffness, more brittleness and dilation during shearing while the soils on the drying path had a higher shear strength and exhibited lower stiffness, more ductility and contraction during shearing.

A series of CD triaxial tests on the statically compacted kaolin was conducted in Uchaipichat (2010) study. The shear strength of the specimens on the drying path was found to be significantly higher than that of the specimens on the wetting paths at the matric suction range higher than AEV.

2.5.2 Shear Strength Equations for Unsaturated Soils

Bishop (1959) proposed a shear strength equation for unsaturated soils by extending Terzaghi's principle of effective stress for saturated soils.

$$\tau = c' + [(\sigma - u_a) + \chi(u_a - u_w)] \tan \phi' \quad \text{Equation 2-7}$$

where:

τ = shear strength of unsaturated soil

c' = effective cohesion

$(\sigma - u_a)$ = net normal stress

$(u_a - u_w)$ = matric suction

ϕ' = effective internal friction angle of saturated soil

u_a = pore-air pressure

u_w = pore-water pressure

χ = a parameter dependent on the degree of saturation

The value of χ was assumed to vary from one to zero, which represents the variation from a fully saturated condition to a totally dry condition (Bishop 1959; Aitchison 1961; Blight 1967). As noted by Coleman (1962), χ is strongly related to the soil structure. Besides, χ also depends on the wetting history, soil type and loading path. Later, Jennings and Burland (1962), Bishop and Blight (1963), Burland (1965), Matyas and Radhakrishna

(1968) and other researchers realized the limitation and the unsuitability of using single stress state variable in shear strength equation for an unsaturated soil.

Fredlund et al. (1978) proposed an equation to describe the shear strength of unsaturated soils in terms of the two independent stress state variables. Fredlund et al. (1978) equation is shown as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi_1^b \quad \text{Equation 2-8}$$

where,

ϕ_1^b = angle indicating the rate of increase in shear strength with respect to matric suction, which is a constant value, introduced in Fredlund et al. (1978) equation.

Equation 2-8 was verified by Fredlund et al. (1978) using two sets of Bishop et al. (1960) test data. Later, Fredlund et al. (1978) equation was verified experimentally by Escario (1980), Ho and Fredlund (1982), Fredlund and Rahardjo (1987), Krahn et al. (1989) and many more. This equation can be considered as an extension of the saturated shear strength equation for saturated soil. It can be used in both saturated and unsaturated soil mechanics and it shows a smooth transition between the shear strength of saturated and unsaturated soils. Under the saturated condition, the pore-water pressure is equal to the pore-air pressure and matric suction is zero and as a result, Equation 2-8 can be simplified to Equation 2-9, which is the conventional shear strength equation (Terzaghi, 1936) for saturated soil.

$$\tau = c' + (\sigma - u_w) \tan \phi' \quad \text{Equation 2-9}$$

Fredlund et al. (1978) equation was initially assumed to be in a linear form. However, later experimental studies showed that the variation of shear strength with respect to matric suction was found to be non-linear (Escario and Sáez, 1986; Gan et al., 1988; Escario and Juca, 1989; Han et al., 1995; Vanapalli et al., 1996a; Khalili and Khabbaz, 1998; Rassam and Williams, 1999; Houston et al., 2008). The ϕ^b angle, which is not a constant value, is used to indicate the rate of change in shear strength with respect to the change in matric suction. The variation of ϕ^b is related to the changes in the contact area

between water menisci and soil particles when matric suction increases. Generally, the ϕ^b angle is equal to or less than ϕ' . However, some researchers like Escario and Sáez Abramento and Carvalho (1989), Mesri and Abdel-Ghaffar (1995), Gan and Fredlund (1996) and Melinda et al. (2004) found that ϕ^b can be larger than ϕ' .

2.5.2.1 Various Shear Strength Equations of Unsaturated Soil

In Vanapalli et al. (1996b) study, two different shear strength equations were introduced. Both shear strength equations are named as Vanapalli et al. (1996b, Eq. 1) shear strength equations and Vanapalli et al. (1996b, Eq. 2) shear strength equations. On the other hand, Shen and Yu (1996) study also introduced two different shear strength equations. Therefore, the naming conventions of Shen and Yu (1996, Eq. 1) shear strength equation, Shen and Yu (1996, Eq. 2) shear strength equation are used in this thesis.

Vanapalli et al. (1996b, Eq. 1) introduced a general and non-linear form of unsaturated shear strength equation using the entire SWCC (i.e. 0 to 1×10^6 kPa) and the saturated shear strength parameters. Vanapalli et al. (1996b, Eq. 1) equation is an extension of the shear strength equation proposed by Fredlund et al. (1978) as shown below:

$$\tau = c' + (\sigma - u_a) \tan \phi' + [(u_a - u_w) \Theta^\kappa] \tan \phi' \quad \text{Equation 2-10}$$

where:

κ = fitting parameter used for obtaining a best fit between the measured and estimated values

Θ = normalized volumetric water content, θ_w / θ_s

θ_w = volumetric water content at any matric suction

θ_s = saturated volumetric water content

Garven and Vanapalli (2006) modified Equation 2-10 and suggested a relationship between the fitting parameter, κ , and the plasticity index, I_p , based on ten sets of shear strength data of compacted soils. Garven and Vanapalli (2006) suggested that κ is closely related to plasticity index of soil and the relationship between them can be mathematically expressed using a polynomial,

$$\kappa = -0.0016I_p^2 + 0.0975I_p + 1 \quad \text{Equation 2-11}$$

The shear strength of an unsaturated soil can be predicted using Equation 2-10 and Equation 2-11 with the basic properties and SWCC of the soil, without using the unsaturated shear strength data of the soil.

Vanapalli et al. (1996b, Eq. 2) suggested another equation by normalizing volumetric water content to predict the shear strength of unsaturated soil for the range between zero to residual matric suction without using any fitting parameter. This equation can be written in terms of gravimetric water content, w , degree of saturation, S , or volumetric water content, θ_w . The common form of the equation is given as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left[\tan \phi' \left(\frac{\theta_w - \theta_r}{\theta_s - \theta_r} \right) \right] \quad \text{Equation 2-12}$$

where:

θ_r = residual volumetric water content

Oberg and Sallfors (1997) introduced an equation for shear strength prediction for non-clayey soils such as sand and silt. The unsaturated shear strength of soil was suggested to be closely related to the pore area occupied by water which can be represented using the degree of saturation of the soil (Oberg and Sallfors, 1997). The equation is as follows:

$$\tau = c' + [\sigma - Su_w - (1 - S)u_a] \tan \phi' \quad \text{Equation 2-13}$$

where:

S = degree of saturation

Equation 2-13 can be re-arranged into the form of two independent stress state variables as shown below:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) [(\tan \phi')(S)] \quad \text{Equation 2-14}$$

Khalili and Khabbaz (1998) suggested an unsaturated shear strength equation for all type of soils without using any soil water content parameter. It is assumed that the relationship between shear strength and matric suction is in linear at matric suction range lower than AEV. In other words, Equation 2-8 with ϕ^b is equal to ϕ' is used to predict unsaturated shear strength of soil up to the AEV. Khalili and Khabbaz (1998) equation is as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) [(\lambda') (\tan \phi')] \quad \text{Equation 2-15}$$

$$\lambda' = \left\{ \frac{(u_a - u_w)}{(u_a - u_w)_b} \right\}^{-0.55} \quad \text{Equation 2-16}$$

where:

$(u_a - u_w)_b$ = air-entry value of soil

The value, -0.55 was chosen from the best fitting analysis of the equation using 14 sets of experimental data from various soils. Therefore, the shear strength prediction from Equation 2-15 depends on the AEV of the soil and the value of -0.55.

Shen and Yu (1996, Eq. 1) proposed a hyperbolic function for shear strength calculation of unsaturated soil. The proposed equation is shown as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left(\frac{1}{1 + (u_a - u_w)d} \right) \tan \phi' \quad \text{Equation 2-17}$$

where:

d = fitting parameter

Another equation with the parameter of peak shear strength contribution from matric suction was proposed by Shen and Yu (1996, Eq. 2) is given as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left(\frac{1}{\cot \alpha + \frac{(u_a - u_w)}{\beta_1}} \right) \tan \phi' \quad \text{Equation 2-18}$$

where:

α = initial angle of shear strength with respect to matric suction

β_1 = peak value of shear strength contribution from matric suction

These two equations are based on the assumption that the shear strength of an unsaturated soil with respect to matric suction can be represented in hyperbolic form.

In order to predict the shear strength of an unsaturated soil in the transition zone, which is commonly encountered in most practical designs, Bao et al. (1998) modified Equation 2-12 and introduced an equation to predict the unsaturated shear strength by assuming a linear variation (in semi-logarithmic form) of SWCC in the transition zone. The equation is shown as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left[\frac{\log(u_a - u_w)_r - \log(u_a - u_w)}{\log(u_a - u_w)_r - \log(u_a - u_w)_b} \right] (\tan \phi') \quad \text{Equation 2-19}$$

where:

$(u_a - u_w)_r$ = residual matric suction

In Bao et al. (1998) equation, the shear strength of soil is assumed to be linearly proportional to matric suction, where $\tan \phi^b$ is equal to $\tan \phi'$, at the matric suction lower than AEV. The non-linear relationship between shear strength and matric suction is assumed to start from AEV. Values of matric suction are used as the controlling parameters for the secant slope between matric suction and unsaturated shear strength.

Vilar (2006) suggested simplified procedures and hyperbolic equation to estimate the shear strength of an unsaturated soil in order to minimize the number of tests needed to measure the shear strength of unsaturated soil. The equation requires the values of shear strength of saturated soil and shear strength of air-dried sample. The followings are the suggested equations and procedures:

$$c = c' + \frac{\psi}{a + b\psi} \quad \text{Equation 2-20}$$

where:

c = cohesion intercept

ψ = matric suction of the soil

a = curve-fitting parameter

b = curve-fitting parameter

The values of a and b can be obtained from the procedure as presented at below. When the soil is nearly saturated and matric suction approaches zero,

$$\left. \frac{dc}{d\psi} \right|_{\psi \rightarrow 0} = \frac{1}{a} = \tan \phi' \quad \text{Equation 2-21}$$

When the matric suction of soil approaches infinity, it is assumed that the shear strength of soil reaches ultimate value.

$$\lim_{\psi \rightarrow \infty} c = c_{ult} = c' + \frac{1}{b} \quad \text{Equation 2-22}$$

$$b = \frac{1}{(c_{ult} - c')} \quad \text{Equation 2-23}$$

where,

c_{ult} = cohesion intercept at maximum matric suction

Therefore, the unsaturated shear strength can be estimated without using any SWCC data. One set of test on the ultimate shear strength from air-dried sample, or the shear strength at any matric suction larger than the maximum matric suction as expected in the analysis, and the saturated effective shear strength parameter are needed in Vilar (2006) model. This simplified procedures and equations provide an easy tool for preliminary estimation of the shear strength parameters of unsaturated soils.

Since the relationship between the shear strength and the matric suction of a soil changes from linear to nonlinear after the matric suction exceeds the AEV, Rassam and Williams (1999) suggested the two surfaces have to be combined together in order to provide a

smooth transition and a complete description for saturated-unsaturated shear strength of a soil at any matric suction. Rassam and Williams (1999) introduced a shear strength equation that incorporates the effect of net normal stress which is written as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + \psi \tan \phi' - (\psi - \psi_e)^\beta [\gamma + \lambda (\sigma - u_a)] \quad \text{Equation 2-24}$$

where:

ψ_e = air-entry value

β, γ, λ = fitting parameters

On the other hand, Rassam and Williams (1999) found that the AEV and the shape of SWCC change with net normal stress, whereas the relationship between AEV and net normal stress can be considered in a linear form (Figure 2-34) as follows,

$$AEV = AEVI + m(\sigma - u_a) \quad \text{Equation 2-25}$$

where:

AEV = air-entry value (kPa)

$AEVI$ = air-entry value when net normal stress is zero (intercept of the line, kPa)

m = the gradient of the line

Figure 2-34: The illustration of the linear relationship between AEV and net normal stress.

Equation 2-24 incorporates the linear relationship between shear strength and matric suction up to AEV where a non-linear relationship for the remaining range of matric

suction begins. The third term of the equation represents the contribution of matric suction to the shear strength by assuming that the soil remains saturated under any matric suction. The last term of equation is the correction term to consider the decrease in matric suction contribution to the shear strength due to desaturation of the soil.

Rassam and Cook (2002) modified Equation 2-24 and introduced a power additive equation and technique to estimate the shear strength of unsaturated soils. The shear strength at residual matric suction, the effective internal friction angle and SWCC are needed in the proposed equation. Two boundary conditions, ϕ^b value is assumed to be zero at the residual matric suction and the contribution of matric suction to the shear strength at residual matric suction, are used in Rassam and Cook (2002) equation.

In Rassam and Cook (2002) equation, $(\gamma + \lambda\sigma)$ from Rassam and Williams (1999) equation is replaced with φ . Therefore,

$$\tau_s = \psi \tan \phi' - \varphi (\psi - \psi_e)^\beta \quad \text{Equation 2-26}$$

where:

τ_s = the contribution of matric suction to the shear strength of unsaturated soil

By rearranging Equation 2-26 and considering it at the residual matric suction. Equation can be rewritten as,

$$\varphi = \frac{\psi_r \tan \phi' - \tau_{sr}}{(\psi_r - \psi_e)^\beta} \quad \text{Equation 2-27}$$

where:

φ = calculated fitting parameter

τ_{sr} = matric suction contribution to shear strength at residual matric suction

ψ_r = residual matric suction

Differentiating Equation 2-26 with respect to matric suction gives

$$\frac{d\tau_s}{d\psi} = \tan \phi' - \phi \beta (\psi - \psi_e)^{\beta-1} = \tan \phi^b \quad \text{Equation 2-28}$$

When Equation 2-28 is considered at the AEV,

$$\tan \phi' - \phi \beta (\psi - \psi_e)^{\beta-1} = \tan \phi^b = \tan \phi' \quad \text{Equation 2-29}$$

Thus, by substituting Equation 2-27 into Equation 2-28, β can be represented by the following equation,

$$\beta = \frac{\tan \phi' (\psi_r - \psi_e)}{\psi_r \tan \phi' - \tau_{sr}} \quad \text{Equation 2-30}$$

where,

β = calculated fitting parameter

Therefore, in Rassam and Cook (2002) equation, net normal stress effects on matric suction contribution to the shear strength of unsaturated soil are ignored. These equations and technique are most suitable for coarse to medium grained soils whose residual matric suction could be obtained easily and reliably in laboratory.

In Lee et al. (2005) study, Equation 2-10 was modified with an additional linear term in order to associate the effect of net normal stress changes on the SWCC for the shear strength estimation of unsaturated soils. Equation 2-25 was adopted to be used together with Lee et al. (2005) equation in order to obtain the AEV under various net normal stresses. Thus, the shear strength under various net normal stresses can be estimated using the SWCC under zero net normal stress only without performing additional triaxial tests (Lee et al., 2005). The modified equations are as follows:

$$\begin{aligned} \tau &= c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' & \text{if } (u_a - u_w) \leq AEV \\ \tau &= c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] [1 + \lambda (\sigma - u_a)] \Theta^{\kappa} \tan \phi' & \text{Equation 2-31} \\ & & \text{if } (u_a - u_w) \geq AEV \end{aligned}$$

where:

λ = fitting parameter

In Equation 2-31, for the range of matric suction below AEV, ϕ^b is equal to ϕ' and all matric suction contributes to the effective stress. Equation 2-31 is only valid for the low matric suction range, where the degree of saturation changes with the net normal stress, to calculate the shear strength under various net normal stresses (Lee et al., 2005). This is due to the macropores, which is sensitive to the changes of net normal stress, is dominant at low matric suction range of SWCC. In addition, the linear relationship between the degree of saturation and the applied net normal stress is only applicable to some soil Tekinsoy et al. (2004) introduced a new method for shear strength prediction with respect to matric suction. An empirical-analytical logarithmic term was suggested to describe the contribution of matric suction to the shear strength of unsaturated soils. The logarithmic term of the Tekinsoy et al. (2004) shear strength equation of unsaturated is as follows:

$$\tau = c' + (\sigma - u_a) \tan \phi' + \tan \phi' (\psi_e + P_{at}) \ln \left[\frac{\psi + P_{at}}{P_{at}} \right] \quad \text{Equation 2-32}$$

where:

P_{at} = atmospheric pressure

Thus, in Equation 2-32, only a single value of matric suction (AEV) from SWCC is needed to predict the shear strength of unsaturated soil without using any other water content data. Equation 2-32 is easy to be used. It is suitable for fine-grained soils as suggested by Tekinsoy et al. (2004).

All presented shear strength equations of unsaturated soil were used in the evaluation on shear strength equations presented in Section 3.6 as well as in the comparison of the proposed equation and the existing equations from literatures presented in Section 6.6.

2.5.2.2 Comparisons on Various Shear Strength Equations

Some comparison studies on various existing shear strength equations for unsaturated soil were reported in various literature (Garven and Vanapalli, 2006; Goh et al., 2009). These studies give a brief review on the advantages and the disadvantages of each equation in describing or estimating the shear strength of unsaturated soil.

Vanapalli and Fredlund (2002) performed a comparison on four shear strength equations for unsaturated soil using three different soil data captured from Escario and Juca (1989) study. Among the four equations, it was found that Vanapalli et al. (1996b, Eq. 1) was able to provide best agreement between the estimated and measured shear strength of three soils for limited matric suction range of 0 to 1500kPa.

Garven and Vanapalli (2006) conducted an evaluation on six equations, which include SWCC as a parameter for estimating the shear strength of unsaturated soils, using up to twenty sets of published shear strength data. The results showed that the Vanapalli et al. (1996b, Eq. 1) equation gave the highest percentage of success for the predictions among the six tested equations. It was concluded that none of the six shear strength equations were able to provide good estimations on the shear strength of the selected data sets for each equation. Each equation was found to be able to successfully estimate particular sets of data only.

Later, Goh et al. (2009) performed an evaluation on ten unsaturated shear strength equations using ten sets of published data that consists of seven sets of drying shear strength data and three sets of drying and wetting shear strength data. The shear strength equations were classified into two groups, i.e. fitting type and prediction type equations, according to the nature of the equations. Among the fitting type equations, Vanapalli et al. (1996b, Eq. 1) and Lee et al. (2005) were able to provide good estimations on all selected drying and wetting shear strength data while among the prediction type equations, Garven and Vanapalli (2006) equation was able to predict all selected drying shear strength data. Goh et al. (2009) concluded that none of the equation was able to predict both drying and wetting shear strengths effectively.

2.6 PERMEABILITY OF UNSATURATED SOIL

The permeability of soil is a soil property which expresses water flow through soils. It refers to the Darcy coefficient of permeability, k , or the simplified term, coefficient of permeability, in civil engineering (Holtz and Kovacs, 1981). The permeability of a saturated soil with respect to water phase is a function of the void ratio of the soil. However, in unsaturated soil, the permeability of soil with respect to water phase is a function of both void ratio and water content of the soil. Various studies showed that water in soil can only flow through soil voids that are filled with water in a continuous path (Fredlund and Rahardjo, 1993b). On the other hand, the water content of an unsaturated soil is dependent on two stress states, i.e., net normal stress and matric suction. Therefore, the changes in the stress states of a soil affect the water coefficient of permeability of the soil.

Matric suction has the dominant influence on the amount of water in the soil. The relationship between matric suction and water content of a soil can be described using SWCC as discussed in Section 2.3. The coefficient of water permeability decreases significantly as the water content decreases when the matric suction increases in the soil. The water coefficient of permeability of soil can vary by several orders of magnitudes when the matric suction varies in the range of practical interest to engineers.

Basically, there are two approaches to obtain the water permeability of an unsaturated soil, i.e. direct and indirect approaches. For the direct approach, although the unsaturated water permeability of soil has a close relevance to engineering practices, it is rarely measured in the field or in the laboratory. The measurement procedures for unsaturated water permeability of soil are time-consuming, especially for those at high matric suction range, and involve sophisticated testing equipments. For the indirect approach, the permeability of a soil can be estimated by unsaturated permeability function using the properties of the soil, e.g. saturated coefficient of permeability and the SWCC of the soil.

2.6.1 Permeability Functions of Unsaturated Soil

Generally, the shape of the permeability function is similar to the shape of SWCC (Fredlund et al., 1994; Gan and Fredlund, 2000). The permeability of soil remains relatively constant at the matric suction range of below AEV. Subsequently, it starts to decrease significantly at the matric suction range of beyond AEV.

Numerous mathematical procedures were introduced to estimate the water coefficient of permeability of unsaturated soil. There are three different methods for estimating the unsaturated permeability of a soil, i.e. empirical equations, macroscopic models and statistical models (Mualem, 1986; Leong and Rahardjo, 1997a, Fredlund, 2006).

Empirical water permeability equation is a mathematical function that describes the variation in the water coefficient of permeability with matric suction. A number of empirical equations, e.g. Richards (1931), Wind (1955), Gardner (1958), Riffema (1965), Dane and Klute (1977), etc, were introduced for estimating the water coefficient of permeability of an unsaturated soil. The parameters used in the equation, which are often in term of non-dimensional, are usually obtained by curve-fitting approaches. Therefore, a minimum number of direct measurements of permeability is required to determine the parameters used in the empirical equation. These equations can be used in engineering practice when the required information and the minimum number of measured data points needed for each equation are available. In the meanwhile, some studies were conducted in order to define some of these fitted parameters used in the empirical equation for unsaturated permeability (Leong and Rahardjo, 1997b).

The macroscopic and statistical models are considered as theoretical models (Fredlund, 2006), which are based on the relationship among permeability, SWCC and pore size distribution of soil. The macroscopic models derive an analytical and closed-form equation for the permeability function (Mualem, 1986). Macroscopic models assume that each pore is only connected to those of the same size. Numerous studies commented that the models neglect the effect of pore-size randomness. Brooks and Corey (1964) model is one the models belongs to this group.

Several statistical models such as Childs and Collis-George (1950), Burdine (1953), Marshall (1958), Kunze et al. (1968), van Genuchten (1980) etc, have been introduced to determine the permeability function for an unsaturated soil. The statistical models assume that pores of different sizes are interconnected. This model is a more realistic and rigorous model as compared to the macroscopic model. The saturated coefficient of permeability and the SWCC are used in the statistical model to determine the permeability function.

2.6.2 Permeability of Unsaturated Soil at Drying and Wetting

The coefficient of water permeability has been shown to be a close relationship or a unique function of the water content of soil during the drying and wetting processes (Fredlund et al., 1994). In other words, the unsaturated permeability of a soil is hysteretic as it depends on the soil property associated with either the drying or wetting paths (Fredlund and Rahardjo, 1993a; Fredlund, 2006). The coefficient of unsaturated permeability of soil on the drying path can be higher than that on the wetting path by several orders of magnitudes. Both drying and wetting permeability of an unsaturated soil can be obtained using those models described in Section 2.6.1 with the measured drying and wetting SWCCs. Although the direct permeability measurement of an unsaturated soil is a time-consuming task, it is always recommended to be conducted. This is due to the estimations using some of the permeability functions may significantly underestimate the actual unsaturated permeability of certain types of soils (van Genuchten, 1980; Fredlund et al., 1994; Meerdink et al., 1996; Chiu and Shackelford, 1998)).

2.6.3 Unsaturated permeameter

Direct permeability measurement can be conducted in the laboratory or in the field. Although field measurement is always considered to be more representative, laboratory measurement is usually more preferred to be conducted due to the lower cost and fewer uncertainties are involved in laboratory measurements (Benson and Gribb, 1997).

Basically, laboratory permeability measurement of an unsaturated soil is performed using steady-state method or unsteady-state method. Steady-state method (or named as constant-head method, constant-flow method) is performed by maintaining a constant

hydraulic head gradient across a specimen (Fredlund and Rahardjo, 1993a). A steady-state water flow across the specimen is created while the matric suction and water content of the specimen are maintained constant. Benson and Gribb (1997) commented that more accurate results can be produced by steady-state method using Darcy's law, although it is always more time consuming, as compared to the unsteady-state method. Unsteady-state method (i.e. variable-head method, infiltration techniques, instantaneous techniques, etc) can be used either in laboratory or in-situ. Unsteady-state method has several variations, which are mainly in the flow process used as well as in the measurement of the hydraulic head and the flow rate (Fredlund and Rahardjo, 1993a, Krisdani, et al., 2009). The flow process can be a wetting process where water flows into the specimen, or vice versa. The use of variable-head method for permeability measurement causes difficulties in maintaining the stress state of the specimen during the test (Agus et al., 2003)

Saturated high-air entry ceramic disk is the key element as the separator of air and water phases in order to measure the air and water permeabilities of an unsaturated soil using permeameter in the laboratory. However, the saturated high-air entry ceramic disk has a low coefficient of water permeability. Therefore, it was suggested that the coefficient of permeability of the ceramic disk should be at least one order of magnitude higher than the saturated coefficient of permeability of the soil being tested (Gan and Fredlund, 2000).

Numerous different designs of permeameter for unsaturated soil have been introduced. In general, there are two types of permeameters used to measure unsaturated permeability of soil, i.e. rigid wall and flexible wall permeameters. Klute (1965), Fleureau and Taibi (1994), Dane et al. (1998), Gan and Fredlund (2000), Lu et al. (2006), Vanapalli et al. (2007), etc, developed a rigid wall permeameter while Barden and Pavlakis (1971), Huang et al. (1998), Agus et al. (2003), Moncada and Campos (2010), etc, developed a flexible wall permeameter for measuring water and (or) air permeability of unsaturated soil. Agus et al. (2003) reported that water and air permeability of soil at matric suction of 300kPa were successfully measured using the modified permeameter from the study.

2.7 REVIEW OF EXPERIMENTAL PROCEDURES FOR TESTING UNSATURATED SOIL BEHAVIOUR

2.7.1 Strain Rate for Triaxial Test

The shearing process of triaxial test is normally performed at a constant strain rate. An appropriate strain rate is crucial for triaxial test and must be selected carefully before the series of tests are started. For the CD triaxial test, the selection of the appropriate strain rate is done with the consideration of the dissipation of induced excess pore-water pressure due to the axial compression stress on soil specimen. Therefore, a low shearing rate is necessary in order to maintain the drained condition for both air and water phases (Lim, 1994).

A strain rate of 0.0009 mm/min was used by Rahardjo et al. (1995) in the CD triaxial tests for the undisturbed residual soils specimens of the Jurong Sedimentary Formation. The diameter and height of the specimens that was used in Rahardjo et al. (1995) study was 71 mm and 142 mm, respectively.

Miao et al. (2002) conducted a study on the shear strength of Nanyang expansive soil from Henan Province, China. The strain rate of 0.009 mm/min was used in the CD triaxial tests.

Rahardjo et al. (2004) applied a strain rate of 0.009 mm/min and 0.0009 mm/min in CW and CD triaxial tests, respectively, for the statically compacted residual soil from the Jurong Sedimentary Formation of Singapore which consists 34 % of sands, 24 % of silts and 42 % of clays. The specimen diameter and height of 50 mm and 100 mm, respectively, was used in Rahardjo et al. (2004) study.

Meilani et al. (2005) used a strain rate of 0.0008 mm/min for CD triaxial tests on statically compacted coarse kaolin specimens (50 mm in diameter and 100 mm in height). The specimens consist of 85 % of silts and 15 % of clays. No excess pore-water pressure built up was found during the shearing process as monitored at the top, middle and bottom of the specimens using the mini suction probes developed by Meilani et al. (2002).

Lee et al. (2005) conducted a series of CD triaxial tests on the compacted Korea Weathered Granitic residual soil specimens that have the characteristics of silty sand. The strain rate of 0.0001 %/s (equivalent to 0.006 mm/min) was adopted for shearing the soil specimen of 50 mm in diameter and 100 mm in height.

Thu et al. (2006) conducted the CD triaxial tests on statically compacted coarse Kaolin specimens (specimen height of 100 mm and diameter of 50 mm) using a strain rate of 0.0009 mm/min.

Kayadelen et al. (2007) applied a strain rate of 0.004 %/min (equivalent to 0.004 mm/min) in the CD triaxial tests on the undisturbed soil specimens of 50mm in diameter and 100mm in height from Diyarbakir, Turkey that consists of approximately 5 % of sands, 30 % of silts and 65 % of clays. Kayadelen et al. (2007) found that this strain rate was sufficiently slow to ensure drained condition during shearing of the soil specimens.

Goh et al. (2010) conducted a series of CD triaxial tests on the statically compacted sand-kaolin specimens under drying and wetting. The sand-kaolin specimens of 50mm in diameter and 100 mm in height that contain 35 % of fine sand, 44.5 % of silt and 20.5 % of clay 35:65 by dry soil mass, were sheared at a strain rate of 0.0009 mm/min.

2.7.2 Axis Translation Technique

Unlike a saturated soil, an unsaturated soil has negative pore-water pressure which may raise difficulties in testing when the negative pore-water pressure approaches -1 atm (i.e., zero absolute pressure). Water may start to cavitate when the water pressure approaches -1 atm (i.e., -101.3 kPa gauge). As cavitation occurs, the measuring system becomes filled with air, resulting in the pore-water pressure and pore-water volume could not be controlled and measured accurately (Fredlund and Rahardjo 1993a).

Cavitation is the term that describes the process of phase translation from liquid phase to vapour phase along a path of decreasing pressure. Figure 2-35 shows the thermodynamic three phase diagram for pure water. The Ideal Cavitation Path crosses the vaporization

curve at cavitation point as the pressure decreases under a constant temperature (see Figure 2-35). Cavitation could occur at a higher pressure under a higher temperature.

Figure 2-35: Thermodynamic phase diagram for pure water (Lu and Likos, 2004).

In order to prevent cavitation of water in unsaturated soil testing, the axis-translation technique (Hilf, 1956) was developed. The procedure involves a translation of the reference, pore-air pressure, to a positive value, i.e. an externally applied air pressure. Therefore, the pore-water pressure can be raised to a positive value since matric suction is defined as $(u_a - u_w)$. As a result, although a high matric suction is involved, cavitation of water can be avoided using the axis translation technique. This technique has been successfully applied by numerous researchers to the SWCC, volume change, shear strength, permeability testing of unsaturated soils (Bishop and Donald, 1961; Satija, 1978; Ho and Fredlund, 1982; Escario and Sáez, 1986; Gan et al., 1988; Vanapalli et al., 1996b; Agus et al., 2003; Houston et al., 2008; Goh et al., 2010). In unsaturated soil testing, the axis-translation technique always incorporates the high air-entry ceramic disk which separates the air and water phases and allows the independent control of the water pressure and the air pressure up to the AEV of the ceramic disk (Fredlund and Rahardjo, 1993a).

2.8 SUMMARY OF CHAPTER

SWCC of a soil, which is the primary information of an unsaturated soil, is affected by various factors. Soil type, grain-size distribution, plasticity index, soil state, fine contents, initial water content, initial dry density, confining pressure and historical stresses of a soil affect the shape of the SWCC of the soil. SWCC can be obtained from experiments or estimated by using various SWCC estimation functions. SWCC can be best-fitted using Fredlund and Xing (1994) equation with the correction factor, $C(\psi)$, is taken as 1, as suggested by Leong and Rahardjo (1997). Ink bottle effect, shrinking and swelling of “aged” soil, contact-angle effect and presence of entrapped air in a soil are the main mechanisms used to explain the hysteresis of soil. Variation of shear strength with respect to matric suction is non-linear. In the boundary effect zone, a linear relationship between shear strength and matric suction is normally obtained experimentally; however, a nonlinear relationship between shear strength and matric suction is observed in the transition zone. In the residual zone, shear strength of an unsaturated soil may increase or decrease gradually or remain constant. Due to hysteresis, the shear strength and shearing behaviour of soils on the drying and wetting paths are different. This is attributed to the different water contents in the soils that control the contact area of water with soil particles in the soil. Various unsaturated shear strength equations, which associate SWCC or hyperbolic and nonlinear failure envelope behaviour of unsaturated soil as the controlling parameter, have been introduced. These equations have been proven to be able to estimate the shear strength of unsaturated soil in various studies. On the other hand, unsaturated permeability curve of a soil has a similar shape as the SWCC of the soil. It can be measured using unsaturated permeameter or estimated using mathematical procedures. The unsaturated permeability of a soil also shows hysteretic behavior during drying and wetting.

CHAPTER 3 THEORY

3.1 INTRODUCTION

In this chapter, the applicable theories and the theoretical developments are presented. Grain-size distribution models, SWCC fitting and estimation methods based on grain-size distributions that are used in soil selection in this study are presented. The applicable theories of permeability and shear strength of unsaturated soil are described. An evaluation on various existing shear strength equations for unsaturated soil is presented and followed by the theoretical development of the drying and wetting shear strength prediction equations.

3.2 GRAIN-SIZE DISTRIBUTION

Grain-size distribution is an important basic soil property which governs the shape of SWCC of soil. Sieve and hydrometer analyses are usually performed to obtain grain-size distribution in laboratory. Some other tests, such as optical and electron microscopy are commonly used in order to obtain the very fine particle ($<2\text{ }\mu\text{m}$) sizes.

There are two methods of mathematically fitting equations for grain-size distribution data, namely unimodal and bimodal method. The unimodal fitting equation is designed to fit a single mode distribution while the bimodal fitting equation is designed to fit two modes of distribution. The entire curve of grain-size distribution can be represented with a mathematical equation using best-fit parameters. These mathematical equations can be used as the basic analysis for estimating soil properties or behaviour such as SWCC.

3.2.1 Unimodal

Fredlund and Xing (1994) SWCC equation, which provides a flexible and continuous fitting for laboratory data using a nonlinear regression, was selected as the basis for the development of a grain-size distribution equation in Fredlund et al. (2000) study. The

original Fredlund and Xing (1994) equation was modified slightly to become a reversed curve form to fit the shape of the grain-size distribution curve. The unimodal equation for the grain-size distribution as introduced by Fredlund et al. (2000) is as follows:

$$P_p(d) = \frac{1}{\ln \left[\exp(1) + \left(\frac{a_{gr}}{d} \right)^{n_{gr}} \right]^{m_{gr}}} \left[1 - \frac{\left[\ln \left(1 + \frac{h_{gr}}{d} \right) \right]^7}{\left[\ln \left(1 + \frac{h_{gr}}{d_m} \right) \right]^7} \right] \quad \text{Equation 3-1}$$

where,

P_p = passing percentage at any particular grain-size, d

d = particle diameter of soil (mm)

a_{gr} = fitting parameter that corresponding to initial break of equation (i.e., represent the large particle size)

n_{gr} = fitting parameter that corresponding to maximum slope of equation

m_{gr} = fitting parameters that corresponding to curvature of equation

h_{gr} = residual particle size of soil (mm)

d_m = minimum particle diameter of soil (mm)

Three fitting parameters in Equation 3-1 are iterated with a quasi-Newton fitting algorithm to fit the equation for each soil. The squared differences between the equation and experimental data were progressively minimized in Soil Vision software in order to best-fit the grain-size distribution data, and plotted in logarithmic scale in this research. The output parameters or results are used for SWCC estimation with Pedo-Transfer equations which are discussed in Section 2.3.3.

3.2.2 Bimodal

Very often, soil has a gap-graded grain-size distribution. It is inadequate to use unimodal equation for representing the grain-size distribution of a gap-graded soil. Therefore, the bimodal equation that consists of two superimposed unimodal curves is needed and used for the best-fit analysis of the grain-size distribution.

The gap-graded soil can be treated as a combination of two or more different soils (Durner, 1994). The double curve in gap-graded soil, indicate that the grain size distribution is concentrated around two different particle size (Fredlund et al., 2000). The fitting algorithm fits the gap-graded soil with a bimodal equation by breaking the curve into upper and lower portions separately. Each portion is fitted with nonlinear least square fitting algorithm. Subsequently, both portions are combined together using the superposition method. The parameter, f_w is the weighting factor for the sub-curves, determines the inflection point between two curves. The bimodal equation for the grain-size distribution as introduced by Fredlund et al. (2000) is as follows:

$$P_p(d) = \left\{ f_w \left[\frac{1}{\ln \left(\exp(1) + \left(\frac{a_{bi}}{d} \right)^{n_{bi}} \right)^{m_{bi}}} \right] + (1 - f_w) \left[\frac{1}{\ln \left(\exp(1) + \left(\frac{j_{bi}}{d} \right)^{k_{bi}} \right)^{l_{bi}}} \right] \right\} \left[1 - \left(\frac{\ln \left(1 + \frac{d_{rbi}}{d} \right)}{\ln \left(1 + \frac{d_{rbi}}{d_m} \right)} \right)^7 \right] \quad \text{Equation 3-2}$$

where,

a_{bi} = fitting parameter that related to the initial breaking point of the curve

d = particle diameter of soil (mm)

n_{bi} = fitting parameter that related to the steepest slope of the curve

m_{bi} = fitting parameter that related to the shape of the curve

j_{bi} = fitting parameter that related to the second breaking point

k_{bi} = fitting Parameter that related to the maximum slope of the second hump

l_{bi} = fitting parameter that related to the shape of the second hump

d_{rhi} = parameter that related to the amount of fines in a soil (mm)

f_w = fitting parameter that controlling the split between upper and lower portions

d_m = minimum particle diameter of soil (mm)

Nine parameters are adjusted when fitting the bimodal equation with gap-graded soil data. Seven parameters are determined using a nonlinear least squares fitting algorithm, while another two parameters, d_m and d_{rhi} are essentially fixed. All the fitting and calculations are performed in Soil Vision software in this research.

3.3 SOIL-WATER CHARACTERISTIC CURVE

SWCC is the central of the unsaturated soil mechanics. Very often, SWCC is referred to the relationship between volumetric water content, θ_w , and matric suction, $(u_a - u_w)$, and this relationship is used throughout of this research. During drying of soil, matric suction of soil increases and soil experiences desorption of water. During wetting of soil, matric suction of soil decreases and soil experiences adsorption of water. Due to hysteresis, soil-water relationship follows different paths during drying and wetting, where soil under drying has a higher volumetric water content than soil under wetting for a given matric suction as shown in Figure 3-1.

SWCC can be determined through laboratory testing or field measurement. Some terms such as saturated volumetric water content, θ_s , residual volumetric water content, θ_r , air-entry value (AEV), $(u_a - u_w)_b$, residual suction, $(u_a - u_w)_r$, wetting saturated volumetric water content, θ_{sw} , water-entry volumetric water content, θ_{rw} , wetting saturated point (also known as air expulsion value), $(u_a - u_w)_{bw}$, water-entry value, $(u_a - u_w)_w$, etc can be defined through this graphically representation (see Figure 3-1). These terms were used throughout in this research.

The AEV is defined as the matric suction at which air first enters the largest pores of a soil during drying process (Brooks and Corey, 1964, 1966). It can be determined as the intersection point between the tangent line at the beginning of the SWCC, which is always taken as a horizontal line from the saturated volumetric water content, and the tangent line at the SWCC in the transition zone (Goh et al., 2011) (see Figure 3-1). This technique has been widely used and adopted in various literature, e.g. Fredlund and Xing (1994), Vanapalli et al. (1996b), Leong and Rahardjo (1997), Fredlund (2002), Yang et al. (2004), Fredlund et al. (2006), Ng and Menzies (2007), etc. which is similar to the interpretation of pre-consolidation pressure. The AEV determined using this technique can be considered as the “least possible” AEV and is used for consistency in the data interpretation.

Figure 3-1: Typical SWCC and the definitions of corresponding terms in SWCC.

As matric suction is increased from zero to AEV (in boundary effect zone), there is no significant change in volumetric water content. The volumetric water content decreases significantly and steadily to the residual water content for matric suctions in the range between AEV and the residual suction (in transition zone). The characteristics of volumetric water content decrease depend on various factors as discussed in Section 2.3.1, especially the soil type and the grain-size distribution of soil. The residual volumetric water content is the water content at residual state, when water phase is discontinuous and does not exhibit significant change in volume with a further increase in matric suction (in residual zone). The soil suction at the residual water content is called residual suction.

On the wetting curve, the water-entry value is defined as the matric suction at which water starts to enter the pores of a soil significantly. It can be obtained as the intersection point between the two tangent lines at the wetting SWCC as illustrated in Figure 3-1. Sometimes, the water-entry value is assumed to be equal to the residual matric suction for simplification. As matric suction decreases beyond the water-entry value, the water phase starts to be continuous. The water content increases rapidly after the water-entry value if water is continuously added to the soil.

Figure 3-2: Illustration for boundary drying and wetting curves and scanning curves of the SWCC of an unsaturated soil (Pham et al., 2003).

The drying curve that starts from saturation (zero matric suction) is commonly called the initial drying curve. The wetting curve that starts from a dry condition is called main (boundary) wetting curve. The subsequent drying curve is called main (boundary) drying curve. Besides the initial and boundary curves, there are a series of scanning curves in between the drying and wetting boundary curves. The scanning curves could start from any particular condition that is neither saturated nor dry condition. Figure 3-2 shows the initial, boundary, and scanning curves of a SWCC. However, in order to simplify the naming convention for each drying or wetting path in this research, the terms, first cycle drying, first cycle wetting and second cycle drying represent the initial drying curve (path), the main (boundary) wetting curve and the main (boundary) drying curve, respectively, as illustrated in Figure 3-2. The drying and wetting paths of the subsequent cycles are named accordingly, e.g. second cycle wetting, third cycle drying, third cycle wetting, etc.

Representation of the SWCC may be accomplished through either fitting of existing data or predicting the SWCC from the basic properties of a soil such as grain-size distribution. There are a number of equations and models developed in the past to describe or predict the SWCC as described in Section 2.3.3 and 2.3.4. The following sections present the SWCC fitting theories as well as the SWCC prediction theories used in this research.

3.3.1 SWCC Fitting

A number of equations have been suggested by various researchers to describe SWCC and basically all the suggested equations can be derived from this following generic form (Leong and Rahardjo, 1997b),

$$a_1 \Theta^{b_1} + a_2 \exp(a_3 \Theta^{b_1}) = a_4 \psi^{b_2} + a_5 \exp(a_6 \psi^{b_2}) + a_7 \quad \text{Equation 3-3}$$

where $a_1, a_2, a_3, a_4, a_5, a_6, a_7, b_1$ and b_2 are constants; ψ is matric suction; Θ is normalized volumetric water content (i.e., $(\theta_w - \theta_r)/(\theta_s - \theta_r)$); θ_w is volumetric water content; θ_s is saturated volumetric water content; θ_r is residual volumetric water content.

Among all the existing SWCC fitting equations, Leong and Rahardjo (1997b) found that Fredlund and Xing (1994) SWCC equation appears to be the best equation for a wide range of soils over the entire range of matric suction. Therefore, Fredlund and Xing (1994) SWCC equation was adopted in this study for the fitting and description of the SWCC. Fredlund and Xing equation (1994) has the following form:

$$\theta = C(\psi) \left[\frac{\theta_s}{\left[\ln \left[e + \left(\frac{\psi}{a} \right)^n \right] \right]^m} \right] \quad \text{Equation 3-4}$$

where,

θ = volumetric water content

θ_s = saturated volumetric water content

a = a soil parameter which is related the AEV of the soil (kPa)

n = a soil parameter which is related to the rate of water extraction from the soil once the AEV has been exceeded

m = a soil parameter which is related to the residual water content

e = natural number, 2.71828...

$C(\psi)$ = correction function that forces the SWCC through a matric suction of 1 GPa and zero water content

The correction factor, $C(\psi)$, which was proposed by Fredlund and Xing (1994) to ensure that the water content is zero at matric suction of 1 GPa, has the following form:

$$C(\psi) = 1 - \frac{\ln\left(1 + \frac{\psi}{\psi_r}\right)}{\ln\left(1 + \frac{10^6}{\psi_r}\right)} \quad \text{Equation 3-5}$$

where

ψ_r = suction at which residual water content occurs (kPa)

The correction factor, $C(\psi)$, in Fredlund and Xing (1994) equation was suggested to be unity by Leong and Rahardjo (1997). $C(\psi) = 1$ was adopted in this study for obtaining α , n and m of the drying and wetting SWCCs from literatures and this study, and correlating them with the proposed equations.

Fredlund and Xing (1994) equation was used with a least-square algorithm in the Soil-Vision software for the soil selection process in this study. Besides, Fredlund and Xing (1994) equation was used with the normalized Sum of Square Error (SSE_{norm}) analysis in theoretical development and in fitting of literature and experimental SWCC data in the Microsoft Office Excel in this study.

3.3.2 SWCC Estimation

SWCC of a soil can be estimated based on basic properties of the soil such as grain-size distribution using Pedo-Transfer Function (PTF). Basically, SWCC estimation approaches can be divided into three categories, Point Regression Method, Functional Parameter Regression Method and Physio-Empirical Method as discussed in Section . There are a number of methods for SWCC prediction developed in the past. However, only eight methods were chosen in this study and tested using a soil similar to the soil used in this study. The tests were performed in order to select appropriate methods for the soil prediction used in this study. The computation of the SWCC estimation with various PTFs was performed using the Soil Vision software. The eight PTFs for estimating SWCC as considered in this study are as follows:

1. Fredlund and Wilson Estimation Method (1997, 2002)
2. Arya and Paris Estimation Method (1981)
3. Scheinost Estimation Method (1996)
4. Rawls Estimation Method (1985)
5. Vereecken Estimation Method (1989)
6. Tyler and Wheatcraft Estimation Method (1989)
7. Gupta and Larson Estimation Method (1975)
8. Aubertin Estimation Method (2003)

3.4 UNSATURATED PERMEABILITY

3.4.1 Permeability Function for Unsaturated Soils

Numerous permeability functions for unsaturated soils were introduced and proven to be able to estimate the unsaturated permeability of soil effectively. Among the various permeability functions, the permeability function suggested by Leong and Rahardjo (1997a) that are associated with SWCC were used in this research. As mentioned in Section 2.6, the SWCC of a soil can be used to estimate the water coefficient of permeability with respect to matric suction of the soil.

Leong and Rahardjo (1997a) suggested an empirical permeability function using SWCC to estimate the water coefficient of permeability of an unsaturated soil. The permeability function is as follows:

$$k_w = k_s (\Theta^p) \quad \text{Equation 3-6}$$

where,

k_w = water coefficient of permeability of soil

k_s = water coefficient of permeability of soil at saturated condition

Θ = normalized volumetric water content

p = fitting parameter

Later, Fredlund et al. (2001) conducted a study to determine the typical values for the fitting parameter, p , that used in the Leong and Rahardjo (1997a) permeability function, with 300 sets of SWCC and permeability data. These data were categorized according to different soil types as shown in Table 3-1. Fredlund et al. (2001) suggested a series of typical ranges of p values for different soil types as summarized in Table 3-1.

The water coefficient of permeability of a soil can be easily estimated down to zero water content using the complete SWCC of the soil. However, it should be noted that in the region beyond residual suction, the function may be more indicative of vapor flow (Fredlund et al., 2001). Thus, it may be more reasonable to assume the coefficient of

permeability as a constant, that equal to the coefficient of permeability at residual matric suction (k_r), for the region beyond residual matric suction (Fredlund et al., 2001).

Table 3-1: Summary of the statistical analyses on value of p for various soil types (Fredlund et al., 2001).

Statistics	Clay	Clay loam	Loam	Sand	Sandy clay loam	Silty clay	Silty clay loam	Silt loam	Sandy loam	Loamy sand
Mean	4.34	3.58	3.78	2.37	2.80	5.59	3.22	3.52	2.86	2.67
Median	4.71	2.62	3.56	2.36	2.62	4.77	3.18	3.46	2.85	2.59
Mode	3.00	2.80	3.25	2.20	2.75	4.60	4.05	3.15	2.75	4.63
Standard Deviation	1.50	1.81	1.16	0.49	1.00	1.31	1.36	1.09	0.84	0.68
Sample Variance	2.25	3.29	1.34	0.24	0.99	1.73	1.86	1.19	0.71	0.46
Range	5.42	5.92	4.14	2.25	4.71	6.33	4.97	5.99	4.20	4.01
Minimum	1.52	1.84	1.63	1.25	0.64	1.11	1.28	0.83	1.02	1.41
Maximum	6.94	7.76	5.76	3.49	5.35	7.44	6.25	6.82	5.22	5.42
Number of Sets	21	18	12	49	17	34	18	74	30	29

3.4.2 Determination of Water Coefficient of Permeability

The water coefficient of permeability of an unsaturated soil can be calculated using Darcy's law (Childs and Collis-George, 1950; Fredlund and Rahardjo, 1993a). Darcy's law is written as

$$q_t = vA = kiA = k \frac{h}{L} A \quad \text{Equation 3-7}$$

where,

q_t = total flow rate through the cross-sectional area

v = flow velocity

A = cross-sectional area

k_w = Darcy coefficient of permeability (water coefficient of permeability)

i = hydraulic gradient

h = head loss

L = length of sample

In this research, the water coefficient of permeability of soil, k_w , was calculated using the results obtained from the permeability test. The permeability tests were conducted using the permeameter as discussed in Section 4.3.5 and the modified permeameter as discussed in Sections 4.4.3 and 4.5.5.

In order to obtain the water coefficient of permeability of unsaturated soil using the modified permeameter, the impedance of the ceramic disks was considered, since the coefficient of the permeability of the ceramic disks might affect the determination of the permeability of the soil. The determination of the water coefficient of permeability of soil using the modified permeameter was done by considering the arrangement of disk-soil-disk as three different layers. The flow velocity, v , is the same for every layer while the total head loss, h_T , is the summation of the head loss in each layer (top disk, soil and bottom disk). The head loss and flow velocity for every layer can be represented as follows,

$$v = v_t = v_s = v_b \quad \text{Equation 3-8}$$

$$h_T = h_t + h_s + h_b \quad \text{Equation 3-9}$$

where,

v_t = flow velocity through the top ceramic disk

v_s = flow velocity through the soil specimen

v_b = flow velocity through the bottom ceramic disk

h_t = head loss along the top ceramic disk

h_s = head loss along the soil specimen

h_b = head loss along the bottom ceramic disk

By substituting Equation 3-7 into Equation 3-9, the following equation is obtained.

$$\frac{v_T L_T}{k_T} = \frac{v_t L_t}{k_t} + \frac{v_s L_s}{k_w} + \frac{v_b L_b}{k_b} \quad \text{Equation 3-10}$$

After v is cancelled from both sides of the equation, Equation 3-10 can be rearranged and written as follows,

$$k_w = \frac{L_s}{\frac{L_T}{k_T} - \left[\frac{L_t}{k_t} + \frac{L_b}{k_b} \right]} \quad \text{Equation 3-11}$$

where,

k_w = water coefficient of permeability of soil specimen

k_T = water coefficient of permeability of the disk-soil-disk system

k_t = water coefficient of permeability of top ceramic disk

k_b = water coefficient of permeability of bottom ceramic disk

L_s = height of the soil specimen

L_t = thickness of the top ceramic disk

L_b = thickness of the bottom ceramic disk

L_T = height of the disk-soil-disk system, i.e. $L_T = L_s + L_t + L_b$

Therefore, the water coefficient of permeability of the soil specimen can be calculated using Equation 3-11. The water coefficient of permeability of the disk-soil-disk system, k_T , which was obtained from the permeability test, was calculated using the following equation,

$$k_T = \frac{Q_w}{iAt} \quad \text{Equation 3-12}$$

where,

Q_w = volume of water flow through the soil specimen

i = applied hydraulic gradient, i.e. $i = h/L$

t = elapsed time

3.5 SHEAR STRENGTH OF UNSATURATED SOILS

Unlike a saturated soil, the shear strength of an unsaturated soil depends on two independent stress-state variables. These variables are net normal stress, $(\sigma - u_a)$, and matric suction, $(u_a - u_w)$. Fredlund et al. (1978) proposed an equation to describe the shear strength of unsaturated soils in terms of the two stress state variables. Later, many research works were done on unsaturated shear strength and a number of shear strength equations using these two stress state variables for unsaturated soil were proposed. Some of the published equations were selected from literature for comparison and are presented in Section 3.6.

3.5.1 Extended Mohr-Coulomb Failure Envelope and Stress-Point Envelope

The failure envelope for unsaturated soil can be described by plotting the Mohr circles in a three-dimensional form. This three-dimensional plot is an extension of the two dimensional Mohr-Coulomb failure envelope of a saturated soil (Figure 3-3), with the additional third dimension axis for matric suction, $(u_a - u_w)$, as illustrated in Figure 3-4. This plot is named as extended Mohr-Coulomb Failure Envelope. The frontal plane is similar to the Mohr-Coulomb failure envelope for a saturated soil, with $(\sigma - u_a)$ axis becomes $(\sigma - u_w)$ axis, as u_w equals to u_a in saturated soil. Thus, the relationship among net normal stress, $(\sigma - u_a)$, matric suction, $(u_a - u_w)$, and shear strength, τ , can be described using this plot. An increase in τ with respect to $(\sigma - u_a)$ is characterized by ϕ' . On the other hand, an increase in τ with respect to $(u_a - u_w)$ is expressed by ϕ^b . The shear strength of unsaturated soil increases as the net normal stress or matric suction increases. The equation for the line of intercept of the failure envelope of shear strength versus matric suction in the extended Mohr-Coulomb failure envelope is as follows:

$$c = c' + (u_a - u_w) \tan \phi^b \quad \text{Equation 3-13}$$

where

c = total cohesion or intercept of the shear stress for various matric suction plane in the extended Mohr-Coulomb failure envelope.

Figure 3-3: Mohr-Coulomb failure envelope for a saturated soil (after Fredlund and Rahardjo, 1993a).

Figure 3-4: Extended Mohr-Coulomb failure envelope for an unsaturated soil (after Fredlund and Rahardjo, 1993a).

Another form of failure envelope to represent the stress state of soil at failure is to use the stress point of Mohr circle at failure, i.e. p_f , q_f , r_f . The stress point envelope is obtained by drawing through the stress points at failure. Similar to the Mohr-Coulomb failure envelope, there are two-dimensional plot and three-dimensional plot of stress point envelope for saturated soil and unsaturated soil, respectively. Two dimensional stress point envelope is a form of the p - q plot which was proposed by Lambe and Whitman (1979) as shown in Figure 3-5.

Figure 3-5: Illustration of p - q plot for saturated soil (Holtz and Kovacs, 1981).

The stress point envelope is always used for tracing the stress path of soil specimen from the beginning to its failure point during a test. Stress point envelope is represented by Equation 3-14 and is illustrated in Figure 3-6 (b).

$$q_f = d' + p_f \tan \psi' + r_f \tan \psi^b \quad \text{Equation 3-14}$$

where

p_f = half of the deviator stress at failure, $((\sigma_{1f} - \sigma_{3f})/2)$

q_f = mean net normal stress at failure, $((\sigma_{1f} + \sigma_{3f})/2 - u_a)$

r_f = matric suction at failure, $(u_a - u_w)_f$

σ_{1f} = major principle stress at failure

σ_{3f} = minor principle stress at failure

d' = intercept of the stress point envelope on the q axis when the p_f and r_f are equal to zero

ψ' = slope angle of the stress point envelope

ψ^b = slope angle of the stress point envelope with respect to the stress variable, r_f

The angle between shear strength, q , and matric suction, r , is defined by the angle ψ^b , while the angle between shear strength, q and mean net normal stress, p is characterized by the angle ψ' . The equation for the line of intercept on the q versus r plan is as follows,

$$d = d' + r_f \tan \psi^b \quad \text{Equation 3-15}$$

where,

d' = ordinate intercept on the q versus r plan where r_f and p_f are zero

d = ordinate intercept on the q versus r plane where r_f is zero

Figure 3-6: Comparison of (a) extended Mohr-Coulomb failure envelope, (b) stress point envelope (Fredlund and Rahardjo, 1993a).

Equation 3-15 indicates that there is an increase in shear strength as the matric suction at failure, r_f increases. Contour lines are used to represent the stress point envelope when the surface is projected onto the q versus p plane. After substituting Equation 3-15 into Equation 3-14, the contour lines can be described with the following equation,

$$q_f = d + p_f \tan \psi' \quad \text{Equation 3-16}$$

As shown in Figure 3-6(a) and Figure 3-6(b), the extended Mohr-Coulomb failure envelope and the stress point envelope have different failure surfaces. The stress point envelope is obtained by joining all the top points of the Mohr circles whereas the extended Mohr-Coulomb failure envelope is obtained by drawing tangent plane through all Mohr circles. The extended Mohr-Coulomb failure envelope and stress point envelope has a slope angle of ϕ' and ψ' with respect to net normal stress, a slope angle of ϕ^b and ψ^b with respect to matric suction and ordinate intercept of c' and d' at zero net confining pressure and zero matric suction, respectively. The extended Mohr-Coulomb failure envelope can be related to the stress point envelope by obtaining the relationship between the parameters, i.e. ϕ' , ϕ^b , c' and ψ' , ψ^b , d' , as shown in Figure 3-7 and Figure 3-8. The relationships were derived and represented as the following equations (Fredlund and Rahardjo, 1993a).

Since the length TC is a common length for triangle TBC and TAC (Figure 3-8), thus

$$\frac{q_f}{\tan \psi'} = \frac{q_f}{\sin \phi'} \quad \text{Equation 3-17}$$

After q_f is cancelled from both sides of the equation,

$$\tan \psi' = \sin \phi' \quad \text{Equation 3-18}$$

Since the length TO is a common part of the TC length of both triangle TBC and TAC, thus

$$\frac{d}{\tan \psi'} = \frac{c}{\tan \phi'} \quad \text{Equation 3-19}$$

Figure 3-7: Relationships among ϕ' , ϕ^b and ψ^b (Fredlund and Rahardjo, 1993a).

Figure 3-8: Relationships among c , d , ϕ' and ψ' (Fredlund and Rahardjo, 1993a).

By substituting Equation 3-18 into Equation 3-19 and rearranging the equation,

$$d = c \cos \phi' \quad \text{Equation 3-20}$$

When the matric suction is equal to zero, Equation 3-20 can be written as the follows:

$$d' = c' \cos \phi' \quad \text{Equation 3-21}$$

By substituting Equation 3-13 and Equation 3-15 into Equation 3-20, the following relationship is obtained.

$$d' + r_f \tan \psi^b = c' \cos \phi' + (u_a - u_w)_f \tan \phi^b \cos \phi' \quad \text{Equation 3-22}$$

By substituting Equation 3-21 into Equation 3-22, Equation 3-22 can be simplified to following equation,

$$\tan \psi^b = \tan \phi^b \cos \phi' \quad \text{Equation 3-23}$$

All the derived relationships and equations were used in the analyses of experimental results obtained from this research and for defining the extended Mohr-Coulomb failure envelope from the stress point failure envelope or vice versa.

3.5.2 Stress Paths of Consolidated Drained (CD) Triaxial Tests

In this research, CD triaxial tests were conducted to measure the shear strength of soil at various matric suctions under a constant net confining pressure. Figure 3-9 illustrates the various stress conditions at different stages during the CD triaxial tests.

During consolidation, a confining pressure is applied and the excess pore-air and pore-water pressures are allowed to dissipate under drained conditions for both pore-air and pore-water phases. Therefore, the net confining pressure and matric suction remain constant. In the following stage as shown in Figure 3-9, shearing is conducted when an axial stress is applied on the specimen. The axial stress is defined as the total major principle stress, σ_1 , in the axial direction while the isotropic confining pressure is defined

as the total minor principle stress, σ_3 , in all-round direction. The axial stress is continuously increased at a constant strain rate until the failure condition or the defined maximum axial strain is reached. The excess pore-air and pore-water pressures due to the axial compression are allowed to dissipate under the drained conditions. Therefore, the net confining pressure and matric suction remain constant during the shearing process. After the failure condition or a defined maximum axial strain is reached, the net confining pressure and the matric suction remain the same as the initial net confining pressure and matric suction (at the end of consolidation) since the pore-air and pore-water are under a drained condition during shearing.

Figure 3-9: Stress condition during CD triaxial test (Fredlund and Rahardjo, 1993a).

The stress paths for CD triaxial tests of this study are illustrated in Figure 3-10. A series of identical soil specimens were tested under a drained condition at various matric suctions under a constant net confining pressure. As shown in Figure 3-10, the diameter of Mohr circle at failure increases as the matric suction is increased. The stress point A represents the stress state corresponds to the end of consolidation under a net confining pressure, $(\sigma - u_a)$, and a matric suction, $(u_a - u_w)$. During shearing, as the axial load is applied to the specimen, the stress point moves from point A to point C along the stress path AC. Stress point C represents the stress state at failure condition. As illustrated in Figure 3-10, the diameter of Mohr circle or the deviator stress increases when the stress path moves from stress point A to the failure condition at stress point C. Since the excess

pore-water and pore-air pressure are allowed to dissipate during shearing, the net confining pressure and matric suction remain constant throughout the stress path AC.

The failure envelope line which is drawn tangent to the Mohr circles at failure has a slope angle of ϕ' with respect to $(\sigma - u_a)$ axis, as shown in Figure 3-6(a). This friction angle, ϕ' , is equal to the effective internal friction angle which is obtained from the saturated shear strength tests (Fredlund and Rahardjo 1993a). However, all the failure stress points (i.e. C_1 , C_2 , and C_3 as in Figure 3-10) cannot be joined together on the same $(u_a - u_w)$ plane as they lie at different net normal stresses. Thus, the angle of ϕ^b is derived by a line joining all the cohesion intercepts, c , at various matric suctions. The cohesion intercepts for various matric suctions are obtained by extending the failure envelope to intercept the shear strength versus matric suction plane at zero net confining stress.

Figure 3-10: Typical stress path of CD tests at various matric suctions under constant net confining pressure (Fredlund and Rahardjo, 1993a).

3.6 EVALUATION OF PUBLISHED SHEAR STRENGTH EQUATIONS FOR UNSATURATED SOIL

An evaluation on the selected published equations for the shear strength of unsaturated soil on the drying and wetting paths was conducted. The evaluation of the predictive and descriptive capabilities of the selected equations were done based on estimated shear strength and measured shear strength of a wide range of different soils types. The applicability of the selected equations to estimate or predict drying and wetting shear strengths of unsaturated soil was studied and examined in order to understand the advantages and limitations of each selected equation. This evaluation study was published in Goh et al. (2009) as well as a part of Goh et al. (2010).

3.6.1 Published Shear Strength Data

Eleven sets of published shear strength data as listed in Table 3-2 were used in the evaluation of unsaturated shear strength equations. These data were extracted either from the plots or tables of the original publications. All of these shear strength data were used in the evaluation of the selected published shear strength equations and some of them were used in the development of the proposed shear strength equation. The selected soils include undisturbed soil, residual soil, expansive soil and man-made soil that have plasticity index (I_p) varying from 0 to 45. Eight sets of the selected data consist of drying shear strength data while three sets of the selected data consist of both drying and wetting shear strength data. The detailed information of each soil is attached in Appendix C3.6.

3.6.2 Evaluation of published shear strength equation

A comprehensive comparison and evaluation of twelve published shear strength equations as summarized in Table 3-3 was conducted. The definitions of the symbols, the estimation procedures and the usage of each equation are elaborated in Section 2.5.2. Generally, the shear strength equations can be classified into fitting type and prediction type shear strength equations according to the nature of shear strength equation (Goh et al., 2009; Goh et al., 2010). Unsaturated shear strength measurements, e.g. peak shear strength or shear strength at different matric suctions, are needed in the fitting type equation in order

to estimate the shear strength at a particular matric suction or to estimate the failure envelope. In contrast, unsaturated shear strength measurement is not needed in the prediction type equation. Parameters that are correlated with index properties or unsaturated soil behaviours, e.g. SWCC, non-linear failure envelope behaviour etc, are used in the prediction type equations to predict the unsaturated shear strength of soil. Prediction type equations can be used to obtain a good first approximation of unsaturated shear strength if the testing for unsaturated shear strength is not possible (Goh et al., 2010). All of the selected published equations were originally developed based on drying shear strength data and proposed to estimate or predict the drying shear strength of soil. However, in this study, they were assumed to be able used for the wetting shear strength of soil using the available soil properties and parameters, e.g. wetting SWCC, peak wetting shear strength.

Table 3-2: Summary of the published data used in the evaluation of shear strength equations.

No.	Authors	Materials	USCS Classification	I_p	Type of data
1	Escario and Juca (1989)	Madrid clay sand	CL	8	Drying
2	Escario and Juca (1989)	Madrid gray clay	MH	35	Drying
3	Escario and Juca (1989)	Red silty soil	CL	13.6	Drying
4	Han (1996)	Bukit Timah granitic residual soil	CL	18	Drying & Wetting
5	Kayadelen et al. (2007)	Diyarbakir residual soil	CH	45	Drying
6	Lee et al. (2005)	Korea Weathered granitic residual soil	SM	NP	Drying
7	Melinda (1998)	Bukit Timah granitic residual soil	CL	18	Drying & Wetting
8	Miao et al. (2002)	Nanyang expansive soil	CH	31.8	Drying
9	Rahardjo et al. (2004)	Jurong sedimentary formation residual soil	CL	14.8	Drying & Wetting
10	Thu et al. (2006)	Compacted kaolin	MH	15.4	Drying & Wetting
11	Vanapalli et al. (1996a)	Indian Head glacial till	CL	18.7	Drying

NP: Non-plastic soil

Table 3-3: Published equations for shear strength prediction of unsaturated soil.

Authors	Nature	Shear strength equations
Fredlund et al. (1978)	Fitting	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi_1^b$ Eq. 3-24
Shen and Yu (1996, Eq. 1)	Fitting	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left(\frac{1}{1 + (u_a - u_w)d} \right) \tan \phi'$ Eq. 3-25
Shen and Yu (1996, Eq. 2)	Fitting	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left(\frac{1}{\cot \alpha + \frac{(u_a - u_w)}{\beta_1}} \right) \tan \phi'$ Eq. 3-26
Vanapalli et al. (1996b, Eq. 1)	Fitting	$\tau = c' + (\sigma - u_a) \tan \phi' + [(u_a - u_w) \Theta^\kappa] \tan \phi'$ Eq. 3-27
Vanapalli et al. (1996a, Eq. 2)	Prediction	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \left[\tan \phi' \left(\frac{\theta_w - \theta_r}{\theta_s - \theta_r} \right) \right]$ Eq. 3-28
Oberg and Sallfors (1997)	Prediction	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) [(\tan \phi')(S)]$ Eq. 3-29
Bao et al. (1998)	Prediction	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) (\zeta) (\tan \phi')$ $\zeta = \left[\frac{\log(u_a - u_w)_r - \log(u_a - u_w)}{\log(u_a - u_w)_r - \log(u_a - u_w)_b} \right]$ Eq. 3-30
Khalili and Khabbaz (1998)	Prediction	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) [(\lambda') (\tan \phi')]$ Eq. 3-31 $\lambda' = \left\{ \frac{(u_a - u_w)}{(u_a - u_w)_b} \right\}^{-0.55}$
Rassam and Cook (2002)	Fitting	$\tau = c' + (\sigma - u_a) \tan \phi' + \psi \tan \phi' - \phi (\psi - \psi_e)^\beta$ Eq. 3-32 $\phi = \frac{\psi_r \tan \phi' - \tau_{Sr}}{(\psi_r - \psi_e)^\beta}$ $\beta = \frac{\tan \phi' (\psi_r - \psi_e)}{\psi_r \tan \phi' - \tau_{Sr}}$

Tekinsoy et al. (2004)	Prediction	$\tau = c' + (\sigma - u_a) \tan \phi' + \tan \phi' (\psi_e + P_{at}) \ln \left[\frac{\psi + P_{at}}{P_{at}} \right]$	Eq. 3-33
Lee et al. (2005)	Fitting	$\tau = c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' \text{ if } (u_a - u_w) \leq AEV$ $\tau = c' + [(\sigma - u_a) + AEV] \tan \phi' +$ $[(u_a - u_w) - AEV] \Theta^\kappa [1 + \lambda(\sigma - u_a)] \tan \phi'$ $\text{if } (u_a - u_w) > AEV$	Eq. 3-34
Garven and Vanapalli (2006)	Prediction	$\tau = c' + (\sigma - u_a) \tan \phi' + [(u_a - u_w) \Theta^\kappa] \tan \phi'$ $\kappa = -0.0016 I_p^2 + 0.0975 I_p + 1$	Eq. 3-35 Eq. 3-36
Vilar (2006)	Fitting	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) / [a + b(u_a - u_w)]$ $1/a = \tan \phi' \quad b = 1/(c_{ult} - c')$	Eq. 3-37

Statistical analyses were performed to access the evaluation of the published shear strength equations. Two criteria were used to compare and understand the descriptive and predictive capabilities of each equation. The first criterion was the degree of curve match. The closer the difference between the estimated curve and the measured curve, the better the descriptive capability of the equation (Goh et al., 2010). Degree of curve match was accessed using average relative error (ARE), which is defined as follows:

$$ARE = \frac{1}{N} \sum_{i=1}^N \left| \frac{y_i - \hat{y}_i}{y_i} \right| \times 100 \quad \text{Equation 3-38}$$

where,

y_i = actual value of i th data

$\hat{y}_i$ = estimated value of i th data

N = total number of data available

Two terms, “agreement” and “discrepancy”, were used in the degree of curve match to identify the ability of the published equations in estimating the shear strength using the given published data. The term, “agreement” is defined as ARE being smaller than or

equal to 20 % while the term, “discrepancy” is defined as ARE being larger than 20 % (Goh et al., 2010).

The second criterion was normalized sum of square error (SSE_{norm}). The smaller the value of SSE_{norm} represents better descriptive and predictive capability of the equation. SSE_{norm} is defined as follows:

$$SSE_{norm} = \sum_{i=1}^N \left(\frac{y_i - \hat{y}_i}{y_i} \right)^2 \quad \text{Equation 3-39}$$

The SWCC under zero net stress and the cohesion intercept of unsaturated soil were used in the comparison and evaluation in this research. Cohesion intercept, c , is the summation of the effective cohesion, c' , and the contribution of matric suction on shear strength of soil. The statistical analysis results of the comparisons on the twelve published shear strength equations based on SSE_{norm} and ARE are summarized in Table 3-4.

As expected, the evaluation results show that the fitting type equations are generally able to provide a better estimation of drying and wetting shear strengths than the prediction type equations (Goh et al., 2009; Goh et al., 2010). However, it must be noted that the main drawback of fitting type equation is that more efforts are needed to obtain the measured shear strength results of a soil before using the equation to estimate the shear strength of the soil at a particular matric suction or to estimate the shear strength failure envelope of the soil (Goh et al., 2010). Measured shear strength results are needed to determine the fitting parameter in Eq. 3-25, Eq. 3-27 and Eq. 3-34 or used as parameters in the Eq. 3-26, Eq. 3-32 and Eq. 3-37 in order to perform the estimation of shear strength of soil. Figure 3-11 illustrates the comparison between the selected prediction type equations while Figure 3-12 illustrates the comparison between the selected fitting type equations using the shear strength data from Escario and Jura (1989) study. Figure 3-13 and Figure 3-14 illustrate the estimated drying shear strength envelopes and wetting shear strength envelopes, respectively, using various published equations on Han (1996) drying and wetting shear strength data.

Table 3-4: Summary of the evaluation on the published equations using the published drying strength data.

Shear strength data	Material	Shear strength equation (authors)											
		Shen and Yu (1996, Eq. 1)	Shen and Yu (1996, Eq. 2)	Vanapalli et al. (1996b, Eq. 1)	Vanapalli et al. (1996b, Eq.2)	Oberg and Sallfors (1997)	Bao et al. (1998)	Khalili and Khabbaz (1998)	Rassam and Cook (2002)	Tekinsoy et al. (2004)	Lee et al. (2005)	Garven and Vanapalli (2006)	Vilar (2006)
(Authors)													
Nature of equation		fitting	fitting	fitting	prediction	prediction	prediction	prediction	fitting	prediction	fitting	prediction	fitting
Escario and Juca (1989)	Madrid clay sand	0.0023 (1) <i>[A]</i>	0.1769 <i>[A]</i>	0.0277 (4) <i>[A]</i>	0.0398 <i>[A]</i>	0.9647 <i>[D]</i>	2.2388 <i>[D]</i>	0.1114 <i>[A]</i>	0.0074 (2) <i>[A]</i>	1.1482 <i>[D]</i>	0.0108 (3) <i>[A]</i>	0.0756 <i>[A]</i>	0.0302 (5) <i>[A]</i>
Escario and Juca (1989)	Madrid gray clay	0.3525 <i>[A]</i>	1.3350 <i>[D]</i>	0.0720 (1) <i>[A]</i>	6.2999 <i>[D]</i>	22.0705 <i>[D]</i>	6.6468 <i>[D]</i>	0.3042 (5) <i>[A]</i>	0.1974 (2) <i>[A]</i>	1.3035 <i>[D]</i>	0.2693 (3) <i>[A]</i>	0.2874 (4) <i>[A]</i>	0.3408 <i>[A]</i>
Escario and Juca (1989)	Red silty soil	0.3015 <i>[A]</i>	0.3734 <i>[D]</i>	0.0267 (1) <i>[A]</i>	2.4842 <i>[D]</i>	11.4595 <i>[D]</i>	3.6286 <i>[D]</i>	425.7613 <i>[D]</i>	0.0632 (4) <i>[A]</i>	0.2155 (5) <i>[A]</i>	0.0494 (3) <i>[A]</i>	0.0277 (2) <i>[A]</i>	0.4107 <i>[A]</i>
Han (1996)	Bukit Timah granitic residual soil	0.2261 (3) <i>[A]</i>	71.0872 <i>[D]</i>	0.0330 (1) <i>[A]</i>	0.6875 (5) <i>[D]</i>	4.8149 <i>[D]</i>	1.3107 <i>[D]</i>	0.8699 <i>[D]</i>	0.9844 <i>[D]</i>	2.4464 <i>[D]</i>	0.1141 (2) <i>[A]</i>	1.2174 <i>[D]</i>	0.2271 (4) <i>[A]</i>
Kayadelen et al. (2007)	Diyarbakir residual soil	0.0050 (2) <i>[A]</i>	1.4013 <i>[D]</i>	0.0327 (3) <i>[A]</i>	0.1857 <i>[A]</i>	0.2414 <i>[A]</i>	0.2296 <i>[A]</i>	0.4067 <i>[D]</i>	0.0400 (5) <i>[A]</i>	0.0528 <i>[A]</i>	0.0030 (1) <i>[A]</i>	0.0372 (4) <i>[A]</i>	0.2084 <i>[A]</i>
Lee et al. (2005)	Korea weathered granitic soil	0.1672 (4) <i>[A]</i>	0.1928 <i>[A]</i>	0.0068 (1) <i>[A]</i>	13.2986 <i>[D]</i>	0.6262 <i>[D]</i>	0.6442 <i>[D]</i>	0.3014 <i>[D]</i>	0.1814 (5) <i>[A]</i>	7.7577 <i>[D]</i>	0.0094 (2) <i>[A]</i>	0.0528 (3) <i>[A]</i>	0.3023 <i>[D]</i>
Melinda (1998)	Bukit Timah granitic residual soil	0.0000 (1) <i>[A]</i>	0.1756 <i>[D]</i>	0.0055 (2) <i>[A]</i>	0.0156 (3) <i>[A]</i>	0.2960 <i>[D]</i>	0.0157 (4) <i>[A]</i>	0.0424 <i>[A]</i>	0.0339 <i>[A]</i>	0.2461 <i>[D]</i>	0.0196 (5) <i>[A]</i>	0.1241 <i>[D]</i>	0.2089 <i>[D]</i>
Miao et al. (2002)	Nanyang expansive soils	0.0008 (2) <i>[A]</i>	0.7595 <i>[D]</i>	0.0011 (3) <i>[A]</i>	0.0012 (4) <i>[A]</i>	0.0053 <i>[A]</i>	0.0984 <i>[A]</i>	0.3285 <i>[D]</i>	0.0110 <i>[A]</i>	0.0025 (5) <i>[A]</i>	0.0003 (1) <i>[A]</i>	0.0392 <i>[A]</i>	0.1698 <i>[A]</i>
Rahardjo et al. (2004)	Jurong sedimentary formation residual soil	0.0461 <i>[A]</i>	1.8618 <i>[D]</i>	0.0217 (4) <i>[A]</i>	0.0213 (3) <i>[A]</i>	0.3471 <i>[D]</i>	0.0392 (5) <i>[A]</i>	0.0847 <i>[A]</i>	0.0081 (2) <i>[A]</i>	0.8005 <i>[D]</i>	0.0077 (1) <i>[A]</i>	0.1508 <i>[A]</i>	0.6385 <i>[D]</i>
Thu et al. (2006)	Compacted Kaolin	0.0904 (4) <i>[A]</i>	1.6155 <i>[D]</i>	0.0767 (3) <i>[A]</i>	2.1319 <i>[D]</i>	0.7616 <i>[D]</i>	0.1809 (5) <i>[A]</i>	0.2310 <i>[A]</i>	0.0134 (1) <i>[A]</i>	0.4536 <i>[D]</i>	0.0300 (2) <i>[A]</i>	2.1844 <i>[D]</i>	0.5099 <i>[D]</i>
Vanapalli et al. (1996a)	Indian Head glacial till	0.0315 (2) <i>[A]</i>	4.6379 <i>[D]</i>	0.0125 (1) <i>[A]</i>	0.0380 (3) <i>[A]</i>	1.0308 <i>[D]</i>	0.0725 <i>[A]</i>	1.5767 <i>[D]</i>	0.4860 <i>[A]</i>	0.3116 <i>[A]</i>	0.0381 (4) <i>[A]</i>	0.0441 (5) <i>[A]</i>	0.3304 <i>[A]</i>

Note: SSE_{norm} value is between zero to infinity, which a smaller value representing a better predicted results. Value in parentheses at every row is the best-prediction ranking for the selected published equations on every set of data. Value of 1 indicates the best prediction for the data set.

The [A] and [D] denote “agreement” and “discrepancy”, respectively, which are defined in text.

Table 3-5: Summary of the evaluation on the published equations using the published wetting strength data.

Shear strength data	Material	Shear strength equation (authors)											
		Shen and Yu (1996) ¹	Shen and Yu (1996) ²	Vanapalli et al. (1996a) ¹	Vanapalli et al. (1996a) ²	Oberg and Sallfors (1997)	Bao et al. (1998)	Khalili and Khabbaz (1998)	Rassam and Cook (2002)	Tekinsoy et al. (2004)	Lee et al. (2005)	Garven and Vanapalli (2006)	Vilar (2006)
(Authors)													
Nature of equation		fitting	fitting	fitting	prediction	prediction	prediction	prediction	fitting	prediction	fitting	prediction	fitting
Han (1996)	Bukit Timah granitic residual soil	0.2645 (3) [A]	72.0667 [D]	0.0082 (1) [A]	1.4783 [D]	6.1205 [D]	0.4355 (5) [A]	1.7829 [D]	1.1954 [D]	2.6325 [D]	0.0705 (2) [A]	1.5712 [D]	0.3103 (4) [A]
Melinda (1998)	Bukit Timah granitic residual soil	0.0000 (1) [A]	0.1756 [D]	0.0055 (2) [A]	0.0156 (3) [A]	0.2960 [D]	0.0157 (4) [A]	0.0424 [A]	0.0339 [A]	0.2461 [D]	0.0196 (5) [A]	0.1241 [D]	0.2089 [D]
Rahardjo et al. (2004)	Jurong sedimentary formation residual soil	0.0461 [A]	1.8618 [D]	0.0217 (4) [A]	0.0213 (3) [A]	0.3471 [D]	0.0392 (5) [A]	0.0847 [A]	0.0081 (2) [A]	0.8005 [D]	0.0077 (1) [A]	0.1508 [A]	0.6385 [D]
Thu et al. (2006)	Compacted Kaolin	0.0576 (4) [A]	1.5371 [D]	0.1473 (3) [A]	3.1265 [D]	1.2974 [D]	0.7417 (5) [A]	1.4483 [D]	0.0156 (1) [A]	0.0392 [A]	0.1095 (2) [A]	2.8005 [D]	0.4368 [D]

Figure 3-11: The predicted shear strength envelopes from the published prediction type equations using the experimental data (Madrid clay sand) from Escario and Juca (1989) study.

Figure 3-12: The calculated shear strength envelopes from the published fitting type equations using the experimental data (Madrid clay sand) from Escario and Juca (1989) study.

Figure 3-13: The estimated drying shear strength envelopes from the published fitting and prediction type equations using the experimental data (Bukit Timah granitic residual soil) from Han (1996) study.

Figure 3-14: The estimated wetting shear strength envelopes from the published fitting and prediction type equations using the experimental data (Bukit Timah granitic residual soil) from Han (1996) study.

Among the fitting type equations, Vanapalli et al. (1996b, Eq. 1) equation (i.e. Eq. 3-27) and Lee et al. (2005) equation (i.e. Eq. 3-34) provide good agreement between measured and estimated shear strengths for all sets of drying and wetting shear strength data. Both equations are shown in the comparisons that are able to estimate the shear strength of a wide range of different types of soils for the large range of matric suction effectively. Both equations includes the fitting parameter, κ , along with normalized volumetric water content, Θ , from SWCC as the controlling soil parameters for the secant slopes between matric suction and unsaturated shear strength. Eq. 3-34 is similar to Eq. 3-27 as the additional term, $[1 + \lambda(\sigma_n - u_a)]$, in Eq. 3-34 is equal to one since all published SWCCs used are under zero net confining pressure in order to be consistent with the comparisons of all shear strength equations. The only difference is that Lee et al. (2005) equation incorporates the assumption of ϕ^b being equal to ϕ' and all the matric suction contributes to the effective stress in the range of matric suctions below the AEV.

Normalized volumetric water content varies in similar manner to the normalized area of water which can be visualized as representing the amount of water in the soil (Vanapalli et al., 1996a). Fung (1977) mentioned that the normalized area of water is assumed to be directly proportional to the volume of water in the soil by applying Greens theorem. In addition, various researchers (Lamborn, 1986; Fredlund and Rahardjo, 1993a; Vanapalli et al., 1996b; Oberg and Sallfors, 1997; Rahardjo et al., 2004; Matsushi and Matsukura, 2006) explained that the shear strength of unsaturated soils appears to have a close relationship to the water content of the soil. Therefore, shear strength equations that incorporate normalize volumetric water content from SWCC are reasonable and suitable to be used to estimate unsaturated shear strength of soil.

Besides, Shen and Yu (1996, Eq. 1) fitting type equation (Eq. 3-25) and Rassam and Cook (2002) fitting type equation (Eq. 3-32) also provide reasonably good comparisons between the measured and estimated shear strength for most of the drying and wetting data sets. Both equations incorporate the hyperbolic and non-linear shear strength envelope to calculate unsaturated shear strength. Fitting parameter, d , is used together with matric suction as the controlling soil parameters for the secant slope between matric suction and unsaturated shear strength in Shen and Yu (1996, Eq. 1) hyperbolic equation. However, Eq. 3-32 was found to be unable to estimate the shear strength at matric suction

beyond the residual matric suction for most of the data sets due to the defined boundary conditions as described in 2.5.2. The estimated shear strength envelopes from Eq. 3-32 reached the peak value and started to decrease at the matric suction range higher than residual matric suction.

Among the prediction type equations, Garven and Vanapalli (2006) equation (Eq. 3-35) is able to predict most of the drying shear strength data reasonably. However, Eq. 3-35 is not able to predict any of the wetting shear strength data in the comparison. This could be due to the reason that the polynomial, Eq. 3-36, was originally developed based on the drying shear strength data in Garven and Vanapalli (2006) study. There is only one κ value that can be obtained from Eq. 3-36 which was suggested by Garven and Vanapalli (2006) that it is suitable for the drying shear strength prediction, but it may not be appropriate for the wetting shear strength prediction (Goh et al., 2009).

Vanapalli et al. (1996b, Eq. 2) equation (Eq. 3-28) is able to provide reasonably good comparisons with both drying and wetting shear strengths for more than half of the data sets. Bao et al. (1998) equation (Eq. 3-30) is also able to provide reasonably good predictions on the drying shear strength for more than half of the data sets but only able to provide moderate predictions on the wetting shear strength. However, both equations were found to be unable to predict the shear strength at matric suction beyond the residual matric suction due the normalized term in both equations that have limited applicability matric suctions up to the residual matric suction only. Therefore, similar to Eq. 3-32, Eq. 3-28 and Eq. 3-30 are basically suitable to be used for the predictions of shear strength at a matric suction range lower than the residual matric suction.

Other prediction type equations, Khalili and Khabbaz (1998) equation (Eq. 3-31), Oberg and Sallfors (1998) equation (Eq. 3-29) and Tekinsoy et al. (2004) equation (Eq. 3-33) are only able to provide moderate agreement in drying and wetting shear strength predictions for a few sets of the selected data. In other words, none of the selected prediction type shear strength equations were found to be able to predict drying and wetting shear strengths of a soil effectively.

3.7 DEVELOPMENT OF PROPOSED SHEAR STRENGTH EQUATION FOR SOIL UNDER DRYING AND WETTING

3.7.1 Shear Strength of Unsaturated Soils under Drying and Wetting

Numerous researchers (Han et al., 1995; Nishimura et al., 2002; Melinda et al., 2004; Rahardjo et al., 2004; Thu et al., 2006; Tse and Ng, 2008; Goh et al., 2010) have studied the hysteresis effects on the shear strength of an unsaturated soil. Generally, the soil at drying has higher shear strength as compared to the soil at wetting under the same matric suction. This is mainly due to the difference in water content of soil as well as the different stress paths that the soil experiences during drying and wetting as described in Section 2.5.1.2. The soil after wetting likely behaves as an over-consolidated soil that exhibits brittleness and dilation during shearing while the soil after drying likely behaves as a normally consolidated soil that exhibits ductility and contraction during shearing.

The information of the shear strength of soil under wetting is crucial for practical proposes as most of slope failures caused by rainfall are associated with the wetting process in soil. The reductions of shear strength that caused the shear deformation of slopes due to the wetting process in soil were described in various studies (Krahn et al., 1989; Rahardjo et al., 1995; Kim et al., 2004). The time, costs and efforts needed to determine the shear strength on the wetting path are excessive. Thus, it is a necessity to have equations to describe the shear strength of soil under wetting in order to provide a better estimation of soil behaviour and assess slope stability, especially during rainfall.

As suggested in Section 3.6, prediction type equation is preferred to be used for shear strength prediction due to the advantage that the measured shear strengths are not needed. The time needed as well as the costs and efforts involved to predict the shear strength at a particular matric suction could be reduced to minimum. To date, there are a number of prediction type shear strength equations that have been introduced as described in Sections 2.5.2.1 and 3.6.2. These equations are mainly semi-empirical equations that are based on either the SWCC of soil, mathematical formulations, basic soil properties or unsaturated soil behaviour. All of these equations have been proven to be workable to predict the shear strength of certain types of soils under drying only. However, there is

limited wetting shear strength equation or shear strength equation, which can be used to predict the shear strengths on the drying and wetting paths of a wide range of soil effectively, in the literature. Therefore, a prediction type equation without using any fitting parameters for drying and wetting shear strengths of soil were proposed in this study and published in Goh et al. (2010). The proposed equations should have the following criteria:

- To predict shear strength without using fitting parameters in the equation
- To provide better shear strength prediction.
- To predict shear strengths for a wider range of soil type.
- To predict wetting shear strength using the same equation.
- To predict shear strength based on basic soil properties and SWCC

3.7.2 Development of Shear Strength Prediction Equations

3.7.2.1 Proposed general form of shear strength prediction equations

The proposed shear strength prediction equations (Goh et al. (2010) equations) were modified from Vanapalli et al. (1996b, Eq. 1) equation and Lee et al. (2005) equation. Normalized volumetric water content ($\Theta = \theta_w / \theta_s$) obtained from SWCC is used as the controlling soil parameter for the secant slope between matric suction and unsaturated shear strength in the proposed equations. Vanapalli et al. (1996b) physical model that identifies the three different saturation stages during the desaturation of a soil, i.e. boundary, transition and residual stages, is adopted (see Figure 3-1).

At matric suction lower than AEV, soil is in the boundary stage and assumed to be in saturated condition. Therefore, the assumptions of ϕ^b being equal to ϕ' at matric suctions lower than AEV and all matric suctions that are in the range below the AEV contribute to the effective stress are incorporated in the proposed equations. The relationship between shear strength and matric suction exhibits a non-linear behavior in the transition stage when the matric suction exceeds the AEV of the soil.

The fitting parameter, κ , originally used in Vanapalli et al. (1996b, Eq. 1) equation and Lee et al. (2005) equation, is a constant which needs to be obtained by curve best-fitting.

The use of fitting parameter in shear strength equation might be considered as a drawback due to the need to have available shear strength data at various matric suctions for obtaining the fitting parameter. In other words, more efforts are needed in order to obtain the shear strength of soil at a particular matric suction if fitting parameter is used.

The fitting parameter, κ , for the selected data sets was back-calculated based on the published shear strengths and SWCC data. Value of κ was found to be a non-constant value and vary with matric suction as illustrated in Figure 3-15. The back-calculated κ value was found to be zero for matric suctions below and equal to the AEV of the soil, and increases nonlinearly when matric suction increases (see Figure 3-15). It was also observed that the calculated logarithm difference of matric suction and the AEV of a soil, $\log(u_a - u_w) - \log(u_a - u_w)_b$, varies with matric suction in a similar fashion (see Figure 3-16) as the variation of the back-calculated κ value with matric suction (see Figure 3-15). Therefore, the term of the logarithm difference of matric suction and the AEV of a soil was used in the development of the proposed shear strength equation for unsaturated soils.

Figure 3-15: Variation of back-calculated κ with respect to matric suction for different soils.

Figure 3-16: Variation of calculated logarithm difference of matric suction and the respective air-entry value with respect to matric suction for different soils.

3.7.2.2 Determination of the parameters using statistical approach

A parametric study was conducted to examine and obtain the suitable parameters based on the selected drying and wetting shear strength data. Six parameters, p , q , x , y , d and h , were initially used in the proposed general form of equation in order to relate $\log(u_a - u_w) - \log(u_a - u_w)_b$ to the variation of κ . The proposed general form of κ equation that includes the six parameters is represented as follows,

$$\kappa = \chi \left[p \left[\log(u_a - u_w) - \log(u_a - u_w)_b \right]^q + d \right]^y + h \quad \text{Equation 3-40}$$

With the consideration of the possibility that more parameters are needed for a better shear strength prediction, two additional parameters, b and z are added into the proposed general form of shear strength prediction equations. Therefore, there are eight hypothetical parameters used in the proposed general form of shear strength prediction equations.

The proposed shear strength prediction equation has two forms of equations with the consideration of soil at matric suction lower than AEV and higher than AEV. The proposed general forms of shear strength prediction equation are shown as follows,

$$\begin{aligned} \tau &= c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' & \text{if } (u_a - u_w) \leq AEV \\ \tau &= c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b \Theta^{z\kappa} \tan \phi' & \text{if } (u_a - u_w) \geq AEV \end{aligned} \quad \text{Equation 3-41}$$

$$\kappa = x[p(\log(u_a - u_w) - \log(u_a - u_w)_b)^q + d]^y + h \quad \text{Equation 3-42}$$

where.

p, q, x, y, d, h, b, z = hypothetical parameters.

The development of the proposed shear strength equations were based on six sets of published data, set 1, 4, 6, 8, 9 and 10 as listed in Table 3-2. Set 1, 6 and 8 consist of drying shear strength data while set 4, 9 and 10 consist of both drying and wetting shear strength data. The parameters needed were obtained using a minimization algorithm of SSE_{norm} . The SSE_{norm} was used as the indicator to determine which single parameter or combination of parameters gave the best comparison between predicted and measured shear strengths. The SSE_{norm} of each combination was set to minimum using the Solver in the Microsoft office Excel in order to obtain the parameters. One thousand iterations and five decimal points were chosen to generate the accurate and reliable results.

In the parametric study, a number of combinations for single, two, three, four and eight parameters were analysed. The guidelines for selecting the appropriate combinations of constant parameters for the proposed equations are as follows,

- Least number of parameters in the equations
- Able to provide the minimum SSE_{norm} for all selected data sets
- The parameters can be related to soil properties
- The equation is valid over the entire matric suction range
- The equation is applicable for all soil types of the selected data sets

Figure 3-17, Figure 3-18 and Figure 3-19 show the typical trend of SSE_{norm} changes with respect to the number of parameters used in the parametric study. The relationships between SSE_{norm} and the number of parameters for other sets of selected data are shown in Figure C3.7.2-1 to Figure C3.7.2-5 of APPENDIX C3.7. The analysis results show that there is a significant decrease in SSE_{norm} from single parameter to two parameters. The subsequent decrement of SSE_{norm} is relatively gentle when the number of parameters is increased from two to eight. As a result, the combinations that consist of two parameters were given priority to be considered and analysed for their suitability as the parameters in the proposed shear strength equations.

Figure 3-17: Trend of SSE_{norm} corresponding to the number of parameters used for compacted kaolin from Thu et al. (2006) shear strength data.

Figure 3-18: Trend of SSE_{norm} corresponding to the number of parameters used for compacted kaolin from Thu et al. (2006) shear strength data.

Figure 3-19: Trend of SSE_{norm} corresponding to the number of parameters used for Nanyang Expansive Soils from Miao et al. (2002) shear strength data.

3.7.2.3 Development of proposed drying shear strength prediction equations

Five sets of published drying shear strength data set 1, 4, 6, 8 and 10 as listed in Table 3-2 were used to establish the relationship between the selected parameters and the soil properties. There are twenty eight combinations of two parameters of each soil were analysed and the results are presented in Table C3.7.2-1 of APPENDIX C3.7. The combinations of 3rd set and 14th set were selected based on the established guidelines as listed in Section 3.7.2.2. Basically, both sets of combination comprise the same general form of equations after the other hypothetical parameters are eliminated as follows,

For the 3rd set of combination

$$p_d = 1$$

$$y_d = 1$$

$$x_d = 1$$

$$z_d = 1$$

$$d_d = 0$$

$$h_d = 0$$

For the 14th set of combination

$$p_d = 1$$

$$q_d = 1$$

$$x_d = 1$$

$$z_d = 1$$

$$d_d = 0$$

$$h_d = 0$$

The proposed general forms of shear strength prediction equations are as follows,

$$\tau = c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' \quad \text{if } (u_a - u_w) \leq AEV$$

$$\tau = c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b_d \Theta^{z_d} \tan \phi' \quad \text{if } (u_a - u_w) \geq AEV$$

$$\kappa = x[p(\log(u_a - u_w) - \log(u_a - u_w)_b)^q + d]^y + h$$

For 3rd set of combination which comprises q_d and b_d , the equations become

$$\tau = c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' \quad \text{if } (u_a - u_w) \leq AEV$$

$$\tau = c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b_d \Theta^{\kappa} \tan \phi' \quad \text{if } (u_a - u_w) > AEV$$

$$\kappa = (\log(u_a - u_w) - \log(u_a - u_w)_b)^{q_d}$$

For 14th set of combination which comprises y_d and b_d , the equations become

$$\begin{aligned}\tau &= c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' & \text{if } (u_a - u_w) \leq AEV \\ \tau &= c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b_d \Theta^\kappa \tan \phi' & \text{if } (u_a - u_w) > AEV \\ \kappa &= [\log(u_a - u_w) - \log(u_a - u_w)_b]^{y_d}\end{aligned}$$

As shown, both combinations result in the same general form of equations. Therefore, the 14th set combination with y_d and b_d was selected. The parameters, y_d and b_d , were evaluated, analysed and related to the soil properties of the data sets. Trial and error analyses as well as best-fit analysis were conducted in order to relate the parameters y_d and b_d with the soil properties directly or indirectly. Some figures and detailed descriptions of the trial and error analyses are attached in APPENDIX C3.7.

The parameter, y_d was found to be closely related to plasticity index (I_p) of soils in natural logarithm ($\ln$) form as shown in Figure 3-20. As a result, the y_d can be related to I_p using the following relationships,

$$y_d = 0.502 \ln(I_p + 2.7) - 0.387 \quad \text{Equation 3-43}$$

where,

y_d = the parameter, y for drying shear strength prediction

I_p = plasticity index

The parameter, b_d , was found to be closely related to I_p and the fitting parameter, n_d (n for drying path), from Fredlund and Xing (1994) equation in natural logarithm form as shown in Figure 3-21. Thus, the b_d can be represented using the following form,

$$b_d = -0.245 \{ \ln[n_d(I_p + 4.4)] \}^2 + 2.114 \{ \ln[n_d(I_p + 4.4)] \} - 3.522 \quad \text{Equation 3-44}$$

where,

b_d = the parameter, b , for drying shear strength prediction

n_d = Fredlund and Xing (1994) SWCC equation fitting parameter, n , for drying SWCC

Figure 3-20: The relationship between y_d and plasticity index (I_p) of the soils.

Figure 3-21: The relationship between b_d and I_p and n_d of the soils.

As a result, the proposed drying shear strength prediction equations are as follows,

$$\begin{aligned}\tau &= c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' && \text{if } (u_a - u_w) \leq AEV \\ \tau &= c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b_d \Theta^\kappa \tan \phi' && \text{if } (u_a - u_w) > AEV \\ \kappa &= [\log(u_a - u_w) - \log(u_a - u_w)_b]^{y_d}\end{aligned}$$

where,

$$y_d = 0.502 \ln(I_p + 2.7) - 0.387$$

$$b_d = -0.245 \{ \ln[n_d(I_p + 4.4)] \}^2 + 2.114 \{ \ln[n_d(I_p + 4.4)] \} - 3.522$$

3.7.2.4 Development of proposed wetting shear strength prediction equations

In the development of proposed wetting shear strength prediction equations, three sets of published data that consist of both drying and wetting shear strengths, set 4, 9 and 10 as listed in Table 3-2, were used. Similar to the development of the proposed drying shear strength prediction equation, twenty eight combinations of two parameters of each soil were analyzed. The analysis results are summarized in Table C3.7.2-2 of APPENDIX C3.7. In order to be consistent to the proposed drying shear strength prediction equations, the combination of 14th set which gave a comparative Sum of SSE_{norm} were selected. As described in Section 3.7.2.3, after the other hypothetical parameters are eliminated, the simplified general form of wetting shear strength prediction equations which has the similar form to the proposed drying shear strength prediction equations are shown as follows,

$$\begin{aligned}\tau &= c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' && \text{if } (u_a - u_w) \leq AEV \\ \tau &= c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b_w \Theta^\kappa \tan \phi' && \text{if } (u_a - u_w) > AEV \\ \kappa &= [\log(u_a - u_w) - \log(u_a - u_w)_b]^{y_w}\end{aligned}$$

In order to represent and relate the parameters, y_w and b_w , (y and b for wetting shear strength prediction) to the soil properties, trial and error as well as best-fit analyses were conducted using the selected three sets of published drying and wetting shear strength

data. The parameter, y_w , was found to be linearly related to the y_d as shown in Figure 3-22 and can be represented as the following relationship,

$$y_w = 3.55y_d - 3.00 \quad \text{Equation 3-45}$$

where,

y_w = parameter, y , for wetting shear strength prediction

y_d = parameter, y , for drying shear strength prediction

On the other hand, the parameter, b_w , was found to be related to the b_d and the ratio between n_d and n_w linearly as shown in Figure 3-23. The relationship can be represented as follows,

$$b_w = 0.542b_d \left(\frac{n_d}{n_w} \right) + 0.389 \quad \text{Equation 3-46}$$

where,

b_w = parameter, b , for wetting shear strength prediction

b_d = parameter, b , for drying shear strength prediction

n_w = fitting parameter, n , from Fredlund and Xing (1994) equation for wetting SWCC

n_d = fitting parameter, n , from Fredlund and Xing (1994) equation for drying SWCC

Therefore, the proposed wetting shear strength prediction equations are as follows,

$$\tau = c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' \quad \text{if } (u_a - u_w) \leq AEV$$

$$\tau = c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b_w \Theta^\kappa \tan \phi' \quad \text{if } (u_a - u_w) > AEV$$

$$\kappa = [\log(u_a - u_w) - \log(u_a - u_w)_b]^{y_w}$$

where,

$$y_w = 3.55y_d - 3.00$$

$$b_w = 0.542b_d \left(\frac{n_d}{n_w} \right) + 0.389$$

Figure 3-22: Linear relationship between y_w and y_d . Figure 3-23: Relationship between b_w and b_d .

As a summary, the proposed shear strength prediction equations for drying and wetting shear strengths are in the same forms with different y and b parameters. The values of y and b for wetting shear strength prediction (y_w and b_w) are dependent on the values of y and b for drying shear strength prediction (y_d and b_d). The parameters, y_d and b_d , are related to the plasticity index and the fitting parameter, n , of Fredlund and Xing (1994) SWCC equation. The parameter, y_w , is linearly related to the y_d while the parameter, b_w , is linearly related to the b_d as well as the ratio between the n for drying SWCC (n_d) and the n for wetting SWCC (n_w). The proposed shear strength prediction equations as published in Goh et al. (2010) are as follows,

$$\begin{aligned}\tau &= c' + [(\sigma - u_a) + (u_a - u_w)] \tan \phi' & \text{if } (u_a - u_w) \leq AEV \\ \tau &= c' + [(\sigma - u_a) + AEV] \tan \phi' + [(u_a - u_w) - AEV] b \Theta^\kappa \tan \phi' & \text{if } (u_a - u_w) > AEV\end{aligned}\quad \text{Equation 3-47}$$

$$\kappa = [\log(u_a - u_w) - \log(u_a - u_w)_b]^y \quad \text{Equation 3-48}$$

For the drying shear strength prediction,

$$y_d = 0.502 \ln(I_p + 2.7) - 0.387 \quad \text{Equation 3-49}$$

$$b_d = -0.245 \{ \ln[n_d(I_p + 4.4)] \}^2 + 2.114 \{ \ln[n_d(I_p + 4.4)] \} - 3.522 \quad \text{Equation 3-50}$$

For the wetting shear strength prediction,

$$y_w = 3.55 y_d - 3.00 \quad \text{Equation 3-51}$$

$$b_w = 0.542 b_d \left(\frac{n_d}{n_w} \right) + 0.389 \quad \text{Equation 3-52}$$

where,

τ = shear strength of soil

c' = effective cohesion

$(\sigma - u_a)$ = net normal stress

$(u_a - u_w)$ = matric suction

u_a = pore-air pressure

u_w = pore-water pressure

ϕ'	= effective internal friction angle of saturated soil
Θ	= normalized water content, θ_w/θ_s
θ_w	= volumetric water content at any matric suction
θ_s	= saturated volumetric water content
y_d	= parameter, y for drying shear strength prediction
y_w	= parameter, y for wetting shear strength prediction
b_d	= parameter, b for drying shear strength prediction
b_w	= parameter, b for wetting shear strength prediction
n_d	= fitting parameter, n , of Fredlund and Xing (1994) equation for drying SWCC
n_w	= fitting parameter, n , of Fredlund and Xing (1994) equation for wetting SWCC
AEV	= air-entry value
I_p	= plasticity index

CHAPTER 4 RESEARCH PROGRAMME

4.1 INTRODUCTION

A comprehensive research programme was planned to study the hysteresis effects on the shear strength, volume change and permeability of unsaturated soil. Different soils were used in the testing programme in order to have a better understanding on general behaviours of unsaturated soil under drying and wetting processes. A series of tests using different soils was planned and the results were compared with the prediction from the proposed equations. In this chapter, criteria for soil selection, methods and procedures for specimen preparation, development, modification and setup of testing equipments, various testing procedures as well as testing programme are elaborated.

4.2 OUTLINE OF RESEARCH PROGRAMME

Basically, the research programme of this study consisted of seven main parts:

Soil selection

The selection of suitable soils was conducted after a thorough review of literature as described in Section 2.3.1, using the software programme and following a set of criteria as described in Section 4.3.1.

Basic properties tests and standard properties tests

The basic properties tests and standard properties tests such as standard compaction test, saturated permeability test, saturated consolidated drained (CD) triaxial test, saturated consolidated undrained (CU) triaxial test and Soil-Water Characteristic Curve (SWCC) test were conducted on the selected soils.

Modification of testing apparatus

The modification of triaxial apparatus was done by incorporating modified ceramic disk, modified pedestal, modified top cap and thermal sensors to the modified triaxial apparatus for unsaturated soil testing, i.e., SWCC test, CD triaxial test and unsaturated permeability test. Tempe cells were also modified by incorporating a flushing system to the base of the Tempe cell for conducting wetting process of soil specimens.

SWCC tests

A series of multi-cycled drying and wetting SWCC tests using different soils were conducted by following the same incremental and decremental steps of matric suction.

Unsaturated CD and permeability triaxial test

Series of unsaturated CD triaxial tests and unsaturated permeability tests using different soils were performed under different matric suctions on multi-cycled drying and wetting paths.

Theoretical development and comparison study

The proposed shear strength equations were developed to predict drying and wetting shear strengths of soil. The comparison studies on various published equations from literatures and the proposed equations from this study were conducted using data from literatures and experimental data from this research

Analyses and uses of experimental results

Comprehensive analyses of experimental results were conducted. The measured shear strengths of different soils were used in the comparison study on shear strength equations.

4.3 SOIL CHARACTERIZATION AND PREPARATION

4.3.1 Criteria for Selecting Soil Type

In order to study the hysteresis effects on the mechanical behaviour of unsaturated soil, identical specimens were needed in this research programme to ensure all the results from different specimens can be related and comparable as well as used to establish the characteristics of the soil. Therefore, man-made soil specimens were chosen and used due to its reproducibility.

The selection criteria for the man-made soils specimen used in this study for SWCC test, triaxial test and permeability test were as follows:

1. The soil specimen must be homogeneous and uniform in soil composition, initial dry density and initial water content in order to have meaningful results for analyses and comparisons.
2. The soil specimen should have a distinct AEV, which is at low matric suction (20 - 60 kPa), to give a clear boundary between saturated and unsaturated conditions of soil along its SWCC.
3. The soil specimen should have a low residual matric suction, which is less than 400kPa, to avoid the use high pressure in testing equipments.
4. The soil specimen should have significant hysteresis in SWCC and distinct drying and wetting curves.
5. The coefficient of permeability of the soil specimen should be relatively high at low matric suctions and should not be too low at high matric suctions for air and water phases to be continuous during testing and reasonable time required for the testing is also a consideration. The time needed for most of the unsaturated soil testings are proportional to the unsaturated permeability of the soil. In addition, the unsaturated permeability should not decrease drastically due to an increase in matric suction especially in the transition zone of unsaturated soil.

6. The volume change of the soil specimen should not be too excessive, and ideally should be an “inert” material which has non-expansive and non-collapsing characteristics.

Mixtures between fine sand and coarse kaolin were selected for the experiments. Coarse kaolin (that contains mainly silt particles ($75 \sim 2 \mu\text{m}$) with liquid limit of 57.7 %, plastic limit of 34.6 % and plasticity index of 23.1 %) which is produced by Kaolin Malaysia SDN BHD (Malaysia) and ASTM Graded sand, Ottawa sand which is furnished by U.S. Silica Company were selected to be mixed to produce the ideal soil. Both Ottawa sand and coarse kaolin are considered as inert materials.

In order to have a complete study on the hysteresis of unsaturated soil, a wide range of soil types, i.e. from non-cohesive soils to cohesive soils, were used to determine the SWCCs under single and multi-cycled drying and wetting processes. On top of using the mixture of Ottawa sand and coarse kaolin, pure Ottawa sand was also used to prepare specimens for SWCC test in this research. In other words, sand-kaolin mixtures were used to prepare the soil specimens for SWCC test, triaxial test and permeability test while the Ottawa sand was used to prepare the soil specimens for SWCC test only.

4.3.1.1 Sand and Coarse Kaolin Ratio Selection for Soil Mixture Preparation

In order to prepare the soil specimens with identical particle size distribution for the research programme, Ottawa sand was sieved and categorized into four groups of different particle sizes, 600 - 1250 μm , 300 - 600 μm , 150-300 μm and 75 - 150 μm . The selection of suitable soil mixtures was based on the estimated SWCC from the grain size distribution of different soil mixtures (different ratios among the four groups of sands and coarse kaolin). Before the analysis, various SWCC estimation methods as listed in Section 3.3.2 were evaluated using ten sets of published SWCC data which have similar properties and particle size distributions with those of soils that might be used in this research. The evaluation was done by comparing the measured and the estimated SWCC from various methods. The ten sets of published SWCC data that include basic soil properties, grain size distributions and SWCCs were extracted from publications and Soil Vision database. Two SWCC estimation methods were selected based on the predictive

capability of the methods on the selected ten sets of data. Subsequently, a number of different sand-kaolin mixtures, with different percentages for each group of fine sands and coarse kaolin, were tested and the most suitable soil mixture was then selected. Reasonable assumptions that were made on the basic properties, e.g. total density, water content, specific gravity etc, were used in the Soil Vision programme during the selection. The soil mixture was then prepared according to the selected ratio and the basic properties tests were conducted. The SWCC of the selected soil mixture was re-estimated and re-examined whether the estimated SWCC fulfilled the criteria as described previously. Another soil mixture would be selected and the related procedures were repeated if the criteria were not fulfilled. The theories and equations used in Soil Vision programme for SWCC estimation are described in Chapter 3. The evaluation of SWCC estimation methods, soil selection results and related figures are shown and discussed in Section 5.2 and Section 6.2.

4.3.2 Basic Soil Properties Test

A series of basic soil properties tests which includes grain size analysis (dry sieving and hydrometer test), specific gravity tests, Atterberg limit tests and standard compaction test were conducted to determine the index properties of the soils. All the basic soil properties tests were based on ASTM soil testing standard.

Standard Proctor compaction test as given in ASTM D698 (1997) was conducted to determine the compaction curve for the soil. Different water contents of the soil were used in order to obtain different total densities and to investigate the relationship between water content and dry density of the soil.

After the full compaction curve was obtained (Figure 4-1), a particular dry density and a water content were selected for the specimen preparations. The soil specimen with the selected dry density and water content should satisfy the criteria as described in Section 4.3.1. Usually, soil at the dry optimum has more potential for volume change upon wetting and has a higher saturated permeability coefficient. Soil at the wet optimum is more compressible and has a lower saturated permeability coefficient. Therefore, soil at the optimum water content and maximum dry density condition was selected because it

Figure 4-2: Different sizes of static compaction moulds and plugs that used for soil specimen preparations in this research.

The stainless steel plugs consist of nine removable 10mm thickness disks. The disks can be attached to each other through the threaded screw that protrudes out from the centre of the face of one disk with the threaded hole drilled into the centre of the face of another disk. The screw is connected to the hole of the adjacent disk as illustrated in Figure 4-3. Therefore, the length of the plugs can be changed by adding or removing the disks accordingly to prepare a specimen layer by layer. All disks are designed in a T-shape with two different diameters, where the top part has a slightly larger diameter with a 5 mm thickness and the bottom part has a slightly smaller diameter with a 5 mm thickness (see Figure 4-3). The purpose of the T-shape design for the disk is to reduce the friction between the trapped soil particles in the gaps and along the wall of mould.

Table 4-1 shows the dimensions of disk and mould for each size of static compaction moulds. Coarse kaolin, different sizes of fine sands and water needed were calculated based on the selected ratio and mixed thoroughly and uniformly. All specimens were statically compacted to designated height with equal layers of 10 mm thickness. The mass of mixed soil needed for each layer was measured before placing into the mould for compaction. After a layer of specimen was compacted successfully, the mould was turned over so that the top plug would be at the bottom, and the bottom plug would be at the top. The height of the plug at the top was then reduced 10mm by removing one disk to allow space for the subsequent compaction layer. The compacted soil surface was scratched

consistently before placing the soil mixture inside the mould to ensure good and same bonding between layers. After placing the mixture into the mould, the top plug was inserted into the mould and axial load was applied at a constant rate until the additional layer was compressed to 10 mm. A fixed displacement rate of 1mm/min was applied using the compression machine (Figure 4-4) in order to prevent a high excess pore-water pressure built up and to obtain a uniform density throughout the whole specimen. The procedure was repeated until the designated height of the specimen was reached. The specimen was then extruded using the same compression machine.

Figure 4-3: (a) Stainless Steel plugs; (b) Connection between two adjacent disks; (c) Removable disk (after Ong, 1999).

Table 4-1: Specifications of the 50 mm, 63.5 mm and 71 mm static compaction moulds.

	50 mm static compaction apparatus	63.5 mm static compaction apparatus	71 mm static compaction apparatus
Mould inner diameter (mm)	50	63.5	71
Mould height (mm)	200	200	200
Disk thickness (mm)	10	10	10
Upper disk thickness (mm)	5	5	5
Lower disk thickness (mm)	5	5	5
Upper disk diameter (mm)	50	63.5	71
Lower disk diameter (mm)	40	53.5	61
Purposes of specimen	Consolidated Drained triaxial test and unsaturated permeability test in modified triaxial	SWCC test in Tempe cell and pressure plate	Unsaturated permeability test in triaxial permeameter

Figure 4-4: Static compaction machine used for sample preparation.

4.3.4 Saturated Consolidated Drained (CD) and Consolidated Undrained (CU) Triaxial Tests

Single stage and multistage saturated CD and CU triaxial tests were conducted using the triaxial apparatus shown in Figure 4-5. As shown in Figure 4-5, the confining pressure was applied and controlled through valve A, pore-water pressure was applied and controlled through valve B. A pressure transducer was installed at valve D to monitor the pore-water pressure of the soil specimen. Valve D was opened during flushing when de-aired distilled water was supplied from Valve C with 30 kPa water pressure.

Identical and homogenous statically compacted soil specimens of 50 mm in diameter and 100mm in height were used for CD and CU triaxial tests as a result of the size constrain of the triaxial cell. The soil specimen was saturated by applying back pressure and cell pressure. The procedures were the same as the saturation procedures for SWCC test and consolidated drained test using the modified triaxial apparatus as presented in Session 4.5.3. Upon saturation, the soil specimen was isotropically consolidated to the designated net confining pressure, $(\sigma_3 - u_w)$. During consolidation, the pore-water pressure the specimen was monitored by the pressure transducer at valve D while the pore-water

volume change of the specimen was measured by the volume change indicator. The completion of consolidation was defined as when there was no excess pore-water pressure and when the pore-water volume had reached equilibrium.

Figure 4-5: Schematic layout of triaxial for saturated CD, CU and permeability tests.

After consolidating the specimen to 50 kPa net confining pressure (first stage), the specimen was sheared by applying an axial load at a constant strain rate of 0.0009 mm/min which is the same as the strain rate used for the unsaturated CD triaxial test. Valve A and B were remained opened during the shearing stage. The pore-water pressure was monitored using the pressure transducer at valve D, the pore-water volume change of the specimen was measured by the volume change indicator through Valve B and the axial load and the axial displacement were measured through the load ring and dial gauges. Shearing was terminated when failure was imminent. This could be determined

by observing the deviator stress when it tends to a maximum value. The specimen was then unloaded and consolidated to a higher effective confining pressure and the same procedures were repeated. In the final stage (third stage), the specimen was sheared until the deviator stress reached a constant value or a significant shear plane in the specimen was observed. The maximum axial strain was limited to 20 %. The specimen was then unloaded and all the pressures were released. The soil specimen was dismantled from the pedestal and the rubber membrane was removed after the cell water was drained out. Subsequently, the final water content was determined.

For the multistage CU triaxial tests, constant strain rates of 0.05 mm/min and 0.009 mm/min were used in order to compare the effects of different strain rates on the shear strength of soil. All the procedures were similar to the multistage CD triaxial test except the Valve B was closed during the shearing stage since the tests were conducted under an undrained condition.

For the single stage CD triaxial tests, a constant strain rate of 0.0009 mm/min was used. All the set-up and procedures were the same as the multistage CD triaxial test, except the shearing of specimens under different net confining pressures were terminated after the deviator stress reached a constant value or a significant shear plane in the soil specimen was observed.

4.3.5 Saturated Permeability Test

The saturated permeability tests under different net confining pressures were conducted. Constant head permeability test procedure (D2434-68(1997)) as described in the ASTM (1997) soil testing standard was used. Saturated triaxial apparatus (Figure 4-5) and digital pressure and volume controller (DPVC) were used in the test. Three different net confining pressures were chosen for the saturated permeability test in order to compare and understand the effects of net confining pressure on the saturated permeability of the sand-kaolin mixture.

The saturation and consolidation of saturated permeability test were similar to the saturated CD and CU triaxial tests as described in Section 4.3.4. After consolidating the

soil specimens to the targeted effective confining pressure, valve C was opened. De-aired distilled water with a constant pressure of 10 kPa higher than the pore-water pressure inside the soil specimen was applied and controlled by DPVC through valve C while the back pressure with a constant pressure of 10 kPa lower than pore-water pressure inside the soil specimen was applied and controlled by the volume change indicator through valve B. Thus, water was made to flow upward through the column of soil specimen under the application of 20 kPa constant pressure difference. The test was stopped when the flow rate was constant for a given period of time. The soil specimen was then consolidated to a higher effective confining pressure and the same procedures were repeated in order to measure the permeability of the soil under higher effective confining pressures.

4.4 THE DEVELOPMENT AND SET UP OF EXPERIMENTAL EQUIPMENTS FOR UNSATURATED SOIL TESTING

In this research, Tempe cell and pressure plates were used for obtaining SWCC of soils under zero net confining pressure and the modified triaxial apparatus was used for obtaining SWCC under a constant net confining pressure, unsaturated CD tests and unsaturated permeability tests. Some modifications were done on the Tempe cell in order to conduct the wetting process of soil for SWCC test. The modified triaxial apparatus used in this research was similar to the modified triaxial apparatus for unsaturated soil testing as described by Fredlund and Rahardjo (1993a). Some advanced modifications were done on the modified triaxial apparatus. Thermal sensors were installed for monitoring the changes in ambient and water temperatures. New designs of ceramic disk and pedestal as well as the top cap were implemented in order to speed up the saturation and consolidation processes. In addition, unsaturated permeability and shear strength of a soil specimen can be measured using the same soil specimen in this improved design of the modified triaxial apparatus.

4.4.1 Tempe Cell and Pressure Plates

In this research, Tempe cell and pressure plate were used for obtaining the SWCC of soil under zero net confining pressure. Tempe cell is usually used for SWCC measurement up to 100kPa of matric suction, while the 5-bar and 15-bar pressure plates are used for SWCC measurement up to the matric suction of 500 kPa and 1500 kPa, respectively.

Tempe cell consists of a one bar air-entry value ceramic disk, three O-rings, cell cap, cell base, cell ring and three screws. The schematic cross section of an assembled Tempe cell is shown in Figure 4-6. The ceramic disk was pre-saturated in the vacuum chamber before it was used.

Figure 4-6: Schematic cross section of Tempe cell.

Figure 4-7: Set up of pressure plate apparatus for SWCC measurement (after Agus, 2001).

The layout of the pressure plate apparatus set up is shown in Figure 4-7. The pressure plate consists of a pressure chamber, a 5 bar (i.e., 500 kPa) air-entry value of ceramic disk and a rubber membrane beneath the ceramic disk. The space between the ceramic disk and the rubber membrane serves as a water compartment, which is connected to a burette line that is opened to atmosphere. The ceramic disk was saturated before starting the test

by pouring de-aired distilled water on the surface of the ceramic disk and applying an air pressure of 400 kPa. The valve of the burette was opened and exposed to atmosphere. The de-aired distilled water infiltrated through the ceramic disk and flowed out from pressure chamber due to the pressure difference. The ceramic disk was considered saturated after repeating these procedures for a few times and no more air bubble was observed. After saturation of the ceramic disk, the soil specimens were placed on the saturated ceramic disk. Matric suction was applied and the drying process of specimen was then started.

4.4.2 Modified Tempe Cell

A large Tempe cell was used for obtaining the SWCC of sand under zero net confining pressure in this research. Some modifications were done on the Tempe cell in order to conduct the wetting process of soil for SWCC test. Two holes were drilled through the cell base and two tube connectors were installed. The original water drain tube was blocked using rubber stopper. Therefore, the water tubes from burette and water tank can be connected with each other through the water compartment at the cell base. With these modifications, flushing process can be done easily to remove the air bubbles trapped inside the water compartment. Figure 4-8 shows the modified Tempe cells during the tests.

Figure 4-8: Two modified Tempe cells for drying and wetting SWCC test

4.4.3 Modified Triaxial Apparatus for Unsaturated Soil Testing

Figure 4-9 shows the modified triaxial apparatus that is used for CD triaxial tests, SWCC tests and unsaturated permeability tests of unsaturated soil. A transparent triaxial cell, modified pedestal, modified ceramic disks, modified top cap, constant rate compression machine, two linear variable differential transformers (LVDT), a diffused air volume indicator (DAVI), a digital pressure and volume controller (DPVC-1) for pore-water pressure, a digital pressure and volume controller (DPVC-2) for confining pressure, pore-air pressure control system, two thermal sensors, three pressure transducers, data acquisition system and a personal computer were parts of the triaxial equipment. The Axis-Translation technique (Hilf, 1956) was used in order to prevent cavitation (at -100kPa water pressure) (see Section 2.7.2) of water in the pore-water pressure line. Both pore-air pressure, u_a , and pore-water, u_w , pressures were controlled in order to apply matric suction, $(u_a - u_w)$ on a soil specimen. The layout of the modified triaxial apparatus for unsaturated soil testing is shown in Figure 4-10.

Figure 4-9: Modified triaxial apparatus for unsaturated soil testing.

Figure 4-10: Modified triaxial apparatus for unsaturated soil testing.

4.4.3.1 Compression Machine

A 50kN compression machine from Tritech was used to apply load at a constant strain rate on the soil specimen. The range of strain rate that this compression machine could apply is between 0.00001 mm/min to 9.9 mm/min. In this research, 0.0009 mm/min of strain rate was used to conduct the CD triaxial test.

4.4.3.2 Triaxial Cell

A small transparent triaxial cell was used for unsaturated soil testing. The maximum cell pressure that can be applied is 1700 kPa, which is much higher than the pressure required

in this research programme. Transparent cell wall was used in order to observe deformation of soil specimen.

4.4.3.3 Load Ring

In order to measure the applied load that was exerted to the soil specimen, a 2-kN load ring was used. The load ring LVDT was well-calibrated within the range of 0-2 mm (approximately 0-1.5 kN) with 0.05 mm interval before being installed to the top of triaxial cell.

4.4.3.4 Linear Variable Differential Transformer

In order to measure the axial deformation of specimen during testing, a linear variable differential transformer (LVDT) was attached to the loading ram as shown in Figure 4-10. The measuring range of the LVDT (displacement LVDT) was 25 mm while the maximum axial strain of the soil specimen was set to 20 % (approximately 20 mm). The sensitivity of the displacement LVDT was 0.01 mm. Another LVDT with measuring range of 10mm was attached to the load ring in order to digitalize and record load ring reading automatically and continuously. The sensitivity of the load ring LVDT was 0.002 mm. Both LVDTs were well-calibrated within the required range before the test started.

4.4.3.5 Pressure Transducers

Three pressure transducers were installed on the modified triaxial apparatus. They were attached to valve B for the pore-water pressure measurement, valve C for the pore-air pressure measurement and valve D for the cell pressure measurement, respectively. All transducers are able to measure pressures with accuracy of ± 0.3 kPa and up to 1000 kPa pressure. All of the pressure transducers were well-calibrated within the range of 0-600 kPa with 20 kPa intervals.

4.4.3.6 Thermal Sensor

Thermal sensor (LM35) from RS components Pte Ltd as shown in Figure 4-9 was installed at the outside of the triaxial cell (at middle height of the triaxial cell) for

monitoring the ambient temperature. The accuracy of the thermal sensor is $\pm 0.3^{\circ}\text{C}$. Another submersible thermal sensor from Wetec Pte Ltd was installed inside the triaxial cell for monitoring the cell water temperature. The accuracy of the thermal sensor is $\pm 0.2^{\circ}\text{C}$. Both thermal sensors were well-calibrated within the range of 20°C to 30°C .

4.4.3.7 Modified Pedestal, Ceramic Disk and Cell Base

For the unsaturated triaxial tests, the pedestal for 50 mm diameter specimen was used. In order to conduct unsaturated permeability test and CD triaxial test using the same modified triaxial apparatus with the same soil specimen, some advanced modification was done on the original ceramic disks as well as on the existing pedestal, top cap and triaxial base designs (Fredlund and Rahardjo, 1993a; Ong, 1999; Thu, 2005). With this improved design, the time required for saturation and consolidation can be shortened.

A circularly grooved water compartment in spiral design was constructed on the head of the pedestal. Two water pressure outlets are located on the base of the spiral groove, which are designed for applying pore-water pressure into the water compartment and subsequently into the soil specimen through the modified high air-entry ceramic disk. One water pressure line was connected to valve A and DPVC-1 through the triaxial base, while another water pressure line was connected to valve B through the triaxial base for flushing purposes. A protruded air pressure outlet was constructed on top of the grooves, which are designed for applying pore-air pressure by passing through the modified ceramic disk and then distribute it into soil specimen through the porous metal. The air pressure lines was connected to valve G through the triaxial base and subsequently connected to the pore-air pressure control system. The drawings of modified pedestal and modified triaxial cell base are shown in Figure 4-11 and Figure 4-12, respectively. As shown in Figure 4-12, there are 7 outlets at the modified triaxil cell base to be connected to valves A to G (Figure 4-10), as compared to the existing design of 4 outlets as described in Fredlund and Rahardjo (1993a). Figure 4-13 and Figure 4-14 show the modified top cap and pedestal with spiral grooves as well as the modified ceramic disk with porous metal.

Figure 4-11: Drawing of modified pedestal.

Figure 4-12: Drawing of modified triaxial cell base.

Figure 4-13: The modified top cap and modified pedestal with spiral grooves.

The key element for both measuring and controlling of pore-water pressure is the high air-entry ceramic disk (Fredlund and Rahardjo, 1993a). The separation of the water and air phases is achieved by using a ceramic disk with an air-entry value higher than the maximum applied matric suction of the test. Thus, if only using the original ceramic disk, it is impossible to apply and control both pore-air and pore-water pressures from one side of the soil specimen simultaneously. As a result, some modifications were designed and constructed on the original ceramic disk.

Figure 4-14: The modified pedestal, modified ceramic disks and modified top cap.

Figure 4-15: Modified ceramic disk and porous metal.

A 5-bar high air-entry ceramic disk with a thickness of 7.14 mm was chosen with the considerations of the applied matric suction range during the test and the saturated permeability of ceramic disk. Four equivalent grooves were constructed on top of the ceramic disk. Since a ceramic disk only allows water to pass through within the allowable matric suction range, an opening was constructed at the edge of the base of the grooves as shown in Figure 4-15. The opening allows air from the protruded air pressure outlet from the spiral grooves (Figure 4-13) of the pedestal, to pass through the ceramic disk. The opening of the ceramic disk was designed to be fitted with the protruded air pressure outlet from the spiral grooves. The spaces between the grooves of the ceramic disk were filled with porous metal which was directly connected to the opening of the protruded part from spiral grooves. The main purposes of the ceramic disk and the porous metal are to serve as a medium to distribute the water pressure and the air pressure, respectively, into the soil specimen uniformly as well as provide a flat surface of contact with the soil specimen for shearing test. The modified high air-entry ceramic disk was sealed on the modified pedestal by applying the slow setting epoxy glue along its circumference. During the test, air pressure was supplied from valve C, then passed through the air pressure lines in the protruded air pressure outlet from spiral grooves and subsequently applied and distributed uniformly into the soil specimen by the porous metal. Water pressure was supplied from valve A into water compartment, then was applied and distributed uniformly into soil specimen by the ceramic disk.

4.4.3.8 Modified Top Cap

Modified top cap (Figure 4-13 and Figure 4-16) was designed similar to the modified pedestal as described in Section 4.4.3.7 to apply and control water pressure during saturation and consolidation and to apply both water and air pressures simultaneously during other processes. Figure 4-16 shows the drawing of the modified top cap. It is made of aluminum in order to minimize the self-weight of the top cap on soil specimen. There are one air pressure line and two water pressure lines on the modified top cap which is the same as the modified pedestal. The modified ceramic disk with porous metal was sealed on the top cap by applying the slow setting epoxy glue along its circumference. The air pressure line at the top cap was connected to the valve C through the triaxial cell base. One of the water pressure lines was connected to valve E for flushing purposes while another water pressure line was connected to valve F for applying and controlling pore-water pressure through the triaxial base. During SWCC test and unsaturated CD test, valve A and valve F were joined and connected to DPVC-1 for applying and controlling pore-water pressure. During unsaturated permeability test, valve A and valve F were connected to DPVC-1 and DPVC-3, respectively, for applying and controlling different water pressures in order to create the water head difference for the permeability test.

Figure 4-16: Drawing of modified top cap.

The modified high air-entry ceramic disks were saturated before the test started. Distilled de-aired water was used to fill the triaxial cell and a cell pressure of 400 kPa was applied during the saturation of the modified ceramic disks. Valves B and E were opened and the distilled de-aired water flowed out continuously from the cell to the DAVI through the high air-entry ceramic disks for 12 hours. The water compartments were flushed every 2 hours by applying 30 kPa of water pressure to valves A and F and the water with trapped air bubbles was flushed out from valves B and E and collected at DAVI. Valves A and F were closed when there was no air bubble found during flushing. After saturation was finished, the triaxial cell remained pressurized until the triaxial test started.

4.4.3.9 Flushing System

Flushing system of the modified triaxial apparatus was used to remove the air bubbles which were trapped in the water compartments during the testing. Due to the high pore-air pressure in the specimen and long duration of the test, air diffused through the ceramic disks and turned into air bubbles in the water compartments. Air bubbles could affect the accuracy of the pore-water pressure and volume change measurements and impeded the flow of water into and from the specimen (Ho and Fredlund, 1982). Diffused Air Volume Indicator (DAVI) was connected to valves B and E to collect the air bubbles that were flushed out from the water compartments. During flushing, a 30 kPa of water pressure was applied to the water compartments from valves A and F and water was flushed out to DAVI which was kept open to atmospheric condition through valves B and E. Valves A, B, E and F were closed after flushing was completed.

4.4.3.10 Pore-water Pressure Control System

Two digital pressure and volume controllers (DPVC-1 and DPVC-2) manufactured by GDS Instruments Limited of England were used to apply and control pore-water pressure and confining pressure of specimen, respectively. The soil specimen was placed on the modified ceramic disk of the modified pedestal and modified top cap was placed on the top of soil specimen. During the SWCC and unsaturated CD triaxial tests, the water compartments in the modified pedestal and the modified top cap were connected to DPVC-1 through valves A and F. Water phase in the modified ceramic disks, soil specimen and the water compartments of the modified pedestal were in a continuous

condition. In other words, the pore-water pressure as well as pore-water volume in the soil specimen could be controlled and measured directly by DPVC-1. On the other hand, a pressure transducer was installed at valve B which was connected to the water compartment. The water pressure of the water compartment (equaled to the pore-water pressure of the soil specimen after equilibrium) was measured by the pressure transducer. During the unsaturated permeability test, valves A and F were connected to DPVC-1 and DPVC-3, respectively, in order to control different water pressures to the top and bottom of the specimen as well as to measure the flow of the water in the specimen accurately.

4.4.3.11 Pore-air Pressure Control System

In the modified triaxial apparatus, air pressure was applied into the specimen, in order to create matric suction in soil specimen. Air pressure was applied and distributed uniformly into the specimen through porous metal (Figure 4-15). The porous metal has a low AEV to allow air to go through and fill up the entire pores in the porous metal once the AEV is exceeded. The porous metals were connected with the air pressure lines through the modified top cap and modified pedestal and subsequently were connected to valves C and G. Both valves C and G were connected to the pore-air pressure control line and subsequently connected the air compressor. Therefore, the continuity of air voids in the soil specimen, porous metal, air pressure line and the pore-air control line could be formed for controlling the pore-air pressure in the soil specimen. Filter papers were placed in between specimen and porous metal in order to prevent fines particles being trapped inside the pores of porous metal.

4.4.3.12 Data Acquisition System

In order to computerize and automate the measurements of pore-water pressure, pore-air pressure, pore-water volume changes, total volume changes, temperature changes, displacement and applied load continuously, 8-channels data logger and a personal computer were used as part of the acquisition system. All the pressure transducers, thermal sensor and LVDTs were connected to the data logger that was connected to the personal computer while three DPVCs were directly connected to the personal computer for data collection and monitoring. The pressure transducers, thermal sensor and LVDTs were well-calibrated before the testing started. The calibration results are attached in

4.5 TESTING PROCEDURES

4.5.1 SWCC Tests Using Tempe Cell and Pressure Plate

Sand-kaolin specimen of 63.5 mm in diameter and 30 mm in height were prepared using the static compaction method as described in Section 4.3.3.1. The specimen was then cut into a specimen of 54 mm in diameter using the Tempe cell cutting mould. The cutting process was conducted using the same compression machine with a constant displacement rate of 1 mm/min in order to minimize the disturbance on the specimen due to the cutting. The specimen was then extruded from the Tempe cell cutting mould and placed into the Tempe cell ring.

Figure 4-18: Schematic layout of saturation set up for Tempe cell.

After placing the sand-kaolin specimen into the Tempe cell, the cell cap was tighten with the cell base. The specimen was then saturated from the bottom of the specimen through the pre-saturated ceramic disk in order to allow all the trapped air inside the soil specimen to flow upward and thus saturating the specimen efficiently. The layout of the saturation setup is shown in Figure 4-18.

After the saturation of sand-kaolin specimen in Tempe cell was completed, the specimen was taken out, weighed and transferred to the 5-bar pressure plate for SWCC measurement. In order to apply matric suction to the specimen, the axis-translation technique as proposed by Hilf (1956) was adopted. Air pressure of 10 kPa (equivalent to 100 cm water head) and water pressure of 9 kPa (equivalent to 90 cm water head) were firstly applied (see Figure 4-19). The equilibrated pore-water pressure (was kept constantly at 9 kPa) of the specimen was determined by the height difference between the specimen and the water level in the burette line, which was opened to atmosphere. In this case, the equilibrated pore-water pressure inside the specimen was considered to be equal to the applied water pressure since the specimen was in good contact with the saturated ceramic disk. Therefore, the difference between the air and water pressures applied was equal to the matric suction.

Figure 4-19: Illustration of applying low matric suction for pressure plate test.

The matric suction was applied from the lowest pressure of 1 kPa and subsequently increased to 2, 5, 10, 20, 50, 100, 200, 300 and 490 kPa. For matric suction range of below 10 kPa, the air pressure was applied and controlled using the set-up as illustrated in Figure 4-19. For matric suction range higher than 10 kPa, the air pressure was applied and controlled using the pressure regulator and pressure gauge. When the matric suction was applied, water from the soil specimen started to be drained out through the saturated ceramic disk. The mass of the specimen was weighed daily and consistently in order to have a close monitor of the equilibrium condition of the specimen. Once the specimen

reached equilibrium, the applied matric suction was then increased. When the matric suction was 50 kPa and above, a little of de-aired water was sprayed on the surface of ceramic disk in order to provide a good contact between the saturated ceramic disk and the specimen. The ceramic disk and water compartment were flushed every time after the readings were taken to ensure that the ceramic disk and the water compartment are in saturated condition.

After the drying curve of SWCC test was completed, wetting of the sand-kaolin specimen was conducted in order to obtain the wetting curve of SWCC. Air pressure was decreased from 490 kPa to 300 kPa and water started to flow into the specimen. All the procedures were the same as the procedures in the drying SWCC tests, except the air pressure was decreased after the mass of the specimen had reached equilibrium. The matric suction was decreased according to the designated decremental steps until matric suction of 1 kPa was reached. For multi-cycled drying and wetting SWCC, all procedures were repeated accordingly by following the designated incremental and decremental steps of matric suction. At the end of the test, the water content of specimen was then determined.

4.5.2 SWCC Tests Using Modified Tempe Cell

The sand SWCC specimen was prepared by placing the sand that was well-mixed from different particle sizes of fine sands into the Tempe-cell. The sand was placed and compacted in three layers with 10 mm thickness for each layer. The required amount of sand for each layer was calculated carefully in order to achieve the targeted dry density. After placing the calculated amount of the first layer of sand, the sand was compacted to the targeted dry density. Once the compaction was completed, the second layer of sand was placed and the procedures were repeated.

After the compaction was completed, the Tempe cell was placed into a vacuum chamber with a water level maintained at the ceramic disk of the Tempe cell. Some water was poured carefully on the surface of the sand specimen. A low vacuum pressure, i.e. 5kPa, was applied in order to accelerate the saturation process. After the sand specimen was saturated, excess water at the surface of sand was removed and the cell cap was tighten with the cell base. The Tempe cell was then transferred to the tray filled with water. The

water level was maintained at the height of the ceramic disk of the Tempe cell. Therefore, the equilibrated pore-water pressure at the ceramic disk was considered to be zero as the water level of the tray was opened to atmosphere.

The average of the negative hydrostatic pore-water pressure in the sand specimen was considered as the first matric suction applied on the sand specimen. The air pressure inlet tube was opened to atmosphere and water from the specimen started to drain out through the saturated ceramic disk. The mass of Tempe cell was measured and recorded at a designated time schedule. Once the mass of Tempe cell reached equilibrium, the lowest air pressure of 0.1 kPa was then applied using manometer and subsequently increased to 0.2, 0.5, 1.0, 2.0 and 5.0 kPa. After the specimen reached equilibrium under the air pressure of 5 kPa, the air pressure line was disconnected from manometer and connected to pressure gauge for higher pressure readings. The air pressure of 10 kPa was applied and subsequently increased to 20 and 40 kPa.

After the drying curve of SWCC test was completed, wetting of the sand specimen was conducted. The water tubes from burette and water tank were connected to the water compartment at the cell base. Water pressure of 4 kPa was applied and kept constant during the wetting process. Therefore, the equilibrated pore-water pressure at the ceramic disk was considered to be 4 kPa. Air pressure was decreased from 40 kPa to 24 kPa and water started to flow into the specimen. The specimen was weighed daily and consistently in order to monitor the equilibrium condition of the specimen. Once the equilibrium was reached, the applied matric suction was then decreased to 10 kPa and subsequently decreased to 5.0, 2.0, 1.0, 0.5, 0.2 and 0.1 kPa. Flushing was conducted every time after the readings were taken in order to remove the air bubbles that were trapped inside the water compartment. For multi-cycle of the drying and wetting SWCC, all procedures were repeated accordingly by following the designated incremental and decremental steps of matric suction.

4.5.3 SWCC Tests Using Modified Triaxial Apparatus

Statically compacted sand-kaolin specimen of 50 mm in diameter and 100 mm in height was used for SWCC tests using the modified triaxial apparatus. The specimen used for

SWCC tests was the same specimen used for unsaturated CD test and unsaturated permeability test. The specimen was placed on the modified ceramic disk which was sealed on top of the modified pedestal. Filter paper was placed in between the specimen and some parts of the modified ceramic disk to avoid fine soil particles being trapped inside the pores of the porous metal. In order to prevent leakage, vacuum grease was applied at the circumference of the modified top cap and the modified pedestal. An impermeable rubber membrane was put on the specimen and the modified top cap as well as the modified pedestal. A filter paper was also placed in between specimen and some parts of the modified ceramic disk of the top cap. Three O-rings were mounted on the modified pedestal and another three O-rings were mounted at the modified top cap. The SWCC tests using the modified triaxial apparatus consisted of three stages, saturation, consolidation as well as drying and wetting of soil specimen.

4.5.3.1 Saturation Stage

All specimens were saturated at the beginning of the testing in order to have a uniform initial condition and to reduce the matric suction to a low value. Before starting the saturation, valves C and G were joined and connected to back pressure line which was connected to DPVC-1 to supply water as well as to control water pressure into the specimen through porous metal from top and bottom of the soil specimen simultaneously. The porous metal has much higher permeability than the 5-bar high air-entry ceramic disk, thus the saturation time could be shortened. Cell pressure (confining pressure) was applied and controlled through valve D, which was connected to DPVC-2. Pore-water pressure was measured by the pressure transducers at valve B. Figure 4-10 shows the schematic diagram of the modified triaxial apparatus as discussed in Section 4.4.3.

The soil specimen was saturated by applying a cell pressure, σ_3 , and a back pressure, u_w , with a net confining pressure, $(\sigma_3 - u_w)$, of 10 kPa until the pore-water pressure parameter B value was larger than 0.95 as proposed by Head (1986). Pore-water pressure parameter, B value is defined as the ratio of a change in pore-water pressure to a change in the confining pressure as described by the following equation,

$$B = \frac{\Delta u}{\Delta \sigma_3} \quad \text{Equation 4-1}$$

where

Δu = Pore-water change after the increment of confining pressure

$\Delta \sigma_3$ = increment of confining pressure

In the beginning of saturation, 40 kPa of back pressure and 50 kPa of cell pressure were first applied to the soil specimen. After equilibrium, both valves C and G were closed and cell pressure (confining pressure) was increased to 100 kPa. So, the pore-water pressure of the specimen built up due to the increment of the confining pressure. After the pore-water pressure reached equilibrium, the pore-water pressure value was noted and used for calculating the B value. If the B value was lower than 0.95, back pressure was then increased to 90 kPa. After both pressures reached equilibrium, both valves C and G were closed and the cell pressure was then increased by 50 kPa, to 150 kPa. These procedures were repeated until the B value was larger than 0.95. The total volume change and pore-water volume change of the soil specimen were measured by DPVC-2 and DPVC-1, respectively.

4.5.3.2 Consolidation Stage

After the saturation was completed, the soil specimen was isotropically consolidated to the designated net confining pressure. During consolidation, the back pressure (pore-water pressure) line was connected to valve C and valve G while the cell pressure (confining pressure) line was connected to valve D. Valves A, B, E and F were remained closed. The pore-water pressure and the cell pressure were applied and controlled by DPVC-1 and DPVC-2, respectively, at the designated values. The pore-water pressure was monitored by the pressure transducer at valve B. The completion of consolidation was reached when there was no excess pore-water pressure and the pore-water volume reached equilibrium.

4.5.3.3 Drying and Wetting Stages

After the isotropic consolidation was completed, air pressure lines were connected to valves C and G while the pore-water pressure lines were connected to valves A and F. Confining pressure was kept constant throughout the test as the pressure applied during consolidation. Pore-water pressure was first decreased to a designated pressure while the valves C and G were remained closed. Valve C and valve G were opened and pore-air pressure was applied to the sand-kaolin specimen after approximately 20 % of the excess pore-water pressure, which was measured from the pressure transducer at valve B (see Figure 4-10), was dissipated. This approach was followed in order to avoid water from entering air pressure lines, causing a breakdown of the pressure regulator.

Water volume change and total volume change were measured by DPVC-1 and DPVC-2 while pore-water pressure and pore-air pressure were measured with pressure transducers at valve B and C, respectively (see Figure 4-10). Equilibrium was reached when excess pore-water pressure had been fully dissipated and water volume change was less than 0.04 % per day as suggested by Sivakumar (1993).

After the matric suction and water volume change reached equilibrium, matric suction was increased by decreasing the pore-water pressure while keeping the confining pressure and pore-air pressure constant. The increment of matric suction in a soil specimen implies that the soil specimen was in the drying stage while the decrement of matric suction implies that the soil specimen is in the wetting stage. During the wetting stage, the decrement of matric suction was achieved by increasing the pore-water pressure while maintaining the confining pressure and pore-air pressure constant. The test was completed after the designated number of cycles and matric suction value were reached. All the readings were taken and recorded using data acquisition system through a computer.

4.5.4 Unsaturated Consolidated Drained Triaxial Tests

Soil specimens of 50 mm in diameter and 100 mm in height were used for CD triaxial tests. All the set up and preparations before the test started were the same as SWCC tests as described in Section 4.5.3. The CD triaxial tests consisted of four stages: saturation, consolidation, drying or wetting or both and shearing of soil specimens.

4.5.4.1 Saturation, Consolidation, Drying and Wetting Stages

The saturation, consolidation as well as drying and wetting stages were conducted using the same procedures as the procedures used in the SWCC test (see Section 4.5.3.1 to Section 4.5.3.3). The designated incremental and decremental steps of matric suction were followed during the drying and wetting processes, respectively. Once the designated matric suction value and soil state (on drying or wetting path) were reached and equilibrium, the soil specimen was then sheared under a drained condition.

4.5.4.2 Shearing Stage

Valves A, C, D, F and G were remained open during the shearing stage. Both pore-air and pore-water phases were in a drained condition and were maintained at the same pressures as the pressures before the shearing started. A slow strain rate was applied such that excess pore-water pressure is zero to maintain the same pore-air and pore-water pressures throughout the shearing process. In Rahardjo et al. (2004) study, the strain rate of 0.0009 mm/min was used for the CD triaxial test of compacted residual soil from the Jurong Sedimentary Formation of Singapore. The same strain rate, 0.0009 mm/min, was reported to be used in the study of Thu et al. (2006) for the CD triaxial test of compacted kaolin. Since the sand-kaolin mixture used in this research had similar properties (i.e. saturated permeability, plasticity index, fine contents) as the soils used in both studies, the strain rate of 0.0009 mm/min was adopted for the CD triaxial tests in this research.

Shearing was terminated when the deviator stress reached a constant value or a significant shear plane in the soil specimen was observed. The maximum axial strain was limited to 20 %. The soil specimen was then unloaded and all the pressures were released. The soil specimen was dismantled from the triaxial cell, and the final water content of the soil specimen was obtained. All the measurements, e.g. pore-air pressure, pore-water pressure, total and water volume changes, applied load, displacement as well as cell and ambient temperatures were automatically recorded using the acquisition programme through the computer.

4.5.5 Unsaturated Permeability Tests

Soil specimens of 50 mm in diameter and 100 mm in height were used for unsaturated permeability test. All of the set up and preparations before the test started were the same as those in the SWCC tests as described in Section 4.5.3. The unsaturated permeability tests consisted of four stages, saturation, consolidation, drying or wetting or both and permeability testing of soil specimens.

4.5.5.1 Saturation, Consolidation, Drying and Wetting Stages

The saturation, consolidation as well as drying and wetting stages were conducted using the same procedures as those in the SWCC test (see Section 4.5.3.1 to Section 4.5.3.3). The designated incremental and decremental steps of matric suction were followed during the drying and wetting processes, respectively. Once the designated matric suction value and soil state (on drying or wetting path) were reached and at equilibrium, the unsaturated permeability test was then conducted by applying a constant water head difference using two DPVCs.

4.5.5.2 Unsaturated permeability testing

Once the designated soil state and matric suction were reached and at equilibrium, valve F was disconnected from DPVC-1 and connected to DPVC-3 while valve A was remained connected to DPVC-1. Two DPVCs were used in the unsaturated permeability test to apply and control different water pressures to the top and bottom of specimen in order to create a hydraulic head gradient for water flow in the specimen. Both DPVCs were also used to measure the flow of the water in the specimen accurately. The applied water pressure from DPVC-1 was a pressure of 15 kPa higher than the pore-water pressure of specimen while the applied water pressure from DPVC-3 was a pressure of 15 kPa lower than the pore-water pressure of specimen. Therefore, a water head difference of 30 kPa was created during the unsaturated permeability test. The same water head difference was used in all unsaturated permeability tests in this research.

Water was made to flow upward through the soil specimen under the application of 30 kPa constant water head difference. The inflow and outflow of water passing through the specimen within a given time were measured and monitored continuously. A graph of

water volume against time was plotted from the beginning of the test. The flow rate of water, Q (m^3/s), was calculated. A steady-state condition was established where the inflow and outflow rates were approximately the same. The flow rate was varied in the beginning, but it became constant after the water pressure difference was evenly distributed along the soil specimen. The test was stopped after the water flow under a constant flow rate for a given period of time. Valve F was then reconnected to DPVC-1. The matric suction was increased or decreased by following the designated incremental or decremental steps to the next targeted matric suction. The same procedures were repeated to measure the permeability of the soil under the targeted matric suction.

4.6 EXPERIMENTAL PROGRAMME

The experimental programme consisted of series of SWCC tests, CD triaxial tests and of permeability tests. Three different sand-kaolin mixtures and three different sands were used. Three different sand-kaolin mixtures with the ratio of 15:85, 35:65 and 55:45 by dry soil mass, as presented in Section 4.3.1 and Section 5.3.1, were used for the SWCC tests, CD triaxial tests as well as permeability tests following multi-cycled drying and wetting processes. On the other hand, three different sands with different particle size distributions as presented in Section 5.3.2, which were mixed from different combinations of fine sand groups, were used to determine the multi-cycle drying and wetting SWCCs of different sands. All the experimental programmes are discussed in this section. In order to simplify the naming convention for each test, sand-kaolin mixtures with the ratio of 15:85 (SK-5), 35:65 (SK-10) and 55:45 (SK-17) were named as soil “I”, “II” and “III”, respectively, while the three different sands were named as sand “IV”, “V” and “VI”.

4.6.1 Soil-Water Characteristic Curve Tests

In this research, SWCC tests were conducted using Tempe cell and 5-bar pressure plate for sand-kaolin specimens under zero net confining pressure, modified Tempe cell for sand specimens under zero net confining pressure and the modified triaxial apparatus for sand-kaolin specimens under a constant net confining pressure. The naming convention is adopted for each test as SWCC-c-xam.y.z-s, where the term, SWCC, means the test for obtaining soil-water characteristic curve; “c” indicates that the net confining pressure; “x” indicates the number of drying and wetting cycles that the soil specimen has experienced; “a” indicates the drying or wetting paths that the soil specimen on where the test is ended, “m” indicates the matric suction that the test is ended at, “y” indicates the maximum suction that the specimen has experienced; “z” indicates the name of the specimen for the particular set of SWCC test; “s” indicates the soil type of the specimen.

4.6.1.1 Tempe Cell and Pressure Plate for SWCC Tests

A series of SWCC tests under multi-cycled drying and wetting processes using Tempe cell and pressure plate were conducted. Three different sand-kaolin mixtures were used in

this research. The naming convention is explained in Section 4.6.1. For example, SWCC-0-5W10.500.2-III means the second sand-kaolin mixtures with the ratio of 55-45 specimen for the SWCC test under zero net confining pressure that has undergone five cycles of drying and wetting processes, experienced the highest matric suction of 500 kPa and ended at matric suction of 10 kPa on the 5th cycle wetting path. Table 4-2 shows the testing programme for the SWCC tests using Tempe cell and pressure plate.

Table 4-2: Testing programme of SWCC tests for sand-kaolin specimens without net confining pressure.

Soil type	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Test
Soil I (sand-kaolin mixture, SK-5)	0	3	SWCC-0-3W10.490.1-I
Soil II (sand-kaolin mixture, SK-10)	0	1	SWCC-0-1D800.800.1-II SWCC-0-1D800.800.2-II SWCC-0-1W1.490.1-II SWCC-0-1W1.490.2-II
		2	SWCC-0-2W10.490.1-II
Soil III (sand-kaolin mixture, SK-17)	0	3	SWCC-0-3W10.490.1-III SWCC-0-3W10.490.2-III

4.6.1.2 Modified Tempe Cell for SWCC Tests

A series of SWCC tests for sands under multi-cycled drying and wetting processes using modified Tempe cell were conducted. Three different sands with different particle size distributions, which were mixed from different combinations of fine sand groups, were used to determine the multi-cycled SWCCs under zero net confining pressure. The particle size distributions of the three different sands are presented in Section 5.3.2. The naming convention is explained in Section 4.6.1. For example, SWCC-0-2W1.40.1-IV means the first specimen of sand IV for SWCC test under zero net confining pressure that has undergone two cycles of drying and wetting processes, experienced the highest matric suction of 40 kPa and ended at matric suction of 1 kPa on the 2nd cycle wetting path. Table 4-3 shows the testing programme of SWCC tests for the sand specimens using modified Tempe cell.

Table 4-3: Testing programme of SWCC tests for sand specimens.

Soil type	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Test
Sand IV	0	4	SWCC-0-4D40.40.1-IV
Sand V	0	3	SWCC-0-3W0.3.40.1-V SWCC-0-1W0.2.40.1-V
Sand VI	0	4	SWCC-0-4D40.40.1-VI

Table 4-4: Testing programme of SWCC tests for sand-kaolin specimens under net confining pressure of 50 kPa.

Sand-kaolin ratio	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Matric suction, ($u_a - u_w$) (kPa)	
			100	300
15-85 (SK-5)	50	1	SWCC-50-1D100.100.1-I SWCC-50-1W100.440.1-I	SWCC-50-1D300.300.1-I SWCC-50-1W300.440.1-I
	50	2		SWCC-50-2D300.400.1-I SWCC-50-2W300.440.1-I
35-65 (SK-10)	50	1	SWCC-50-1D100.100.1-II SWCC-50-1W100.440.1-II	SWCC-50-1D300.300.1-II SWCC-50-1W300.440.1-II
	50	2	SWCC-50-2D100.440.1-II SWCC-50-2W100.440.1-II	SWCC-50-2D300.440.1-II SWCC-50-2W300.440.1-II
55-45 (SK-17)	50	1	SWCC-50-1D100.100.1-III SWCC-50-1W100.440.1-III	SWCC-50-1D300.300.1-III SWCC-50-1W300.440.1-III
	50	2	SWCC-50-2D100.440.1-III SWCC-50-2W100.440.1-III	SWCC-50-2D300.440.1-III SWCC-50-2W300.440.1-III
	50	3		SWCC-50-3D300.440.1-III SWCC-50-3W300.440.1-III

Table 4-5: Testing programme of SWCC tests for sand-kaolin specimens under net confining pressure of 100 kPa.

Sand-kaolin ratio	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Matric suction, ($u_a - u_w$) (kPa)	
			50	200
35-65 (SK-10)	100	1	SWCC-100-1D50.50.1-II SWCC-100-1W50.440.1-II	SWCC-100-1D200.200.1-II SWCC-100-1W200.440.1-II

4.6.1.3 Modified Triaxial Apparatus for SWCC Tests

The SWCC tests under a constant net confining pressure for the three different sand-kaolin mixtures using the modified triaxial apparatus were conducted. Similar to the

SWCC test as presented in Section 4.6.1, sand-kaolin specimens experienced multi-cycled drying and wetting processes. The net confining pressures used for each SWCC tests were selected to be the same as those used in the CD triaxial tests. The naming convention is described in Section 4.6.1. For example, SWCC-50-3D100.440.1-III means the first specimen of sand-kaolin type III for the SWCC test under a net confining pressure of 50 kPa that has undergone three cycles of drying and wetting processes, experienced the highest matric suction of 440 kPa and ended at matric suction of 100 kPa on the 3rd cycle drying path. Table 4-4 and Table 4-5 show the testing programme of SWCC tests for sand-kaolin specimens using the modified triaxial apparatus.

4.6.2 Consolidated Drained Triaxial Tests

In order to study the shear strength behaviour and volume change characteristics of the sand-kaolin mixture due to hysteresis, the shear strength characteristics of soils under single cycle and multi-cycled drying and wetting processes were determined. Series of CD triaxial tests were conducted at different matric suctions on drying and wetting paths under different net confining pressures, which are noted in the SWCC figures as presented in Section 5.4.2. Three different sand-kaolin mixtures with the ratio of 15:85, 35:65 and 55:45 by dry mass were selected as discussed in Section 4.3.1 and Section 5.2 and used for soil specimen preparations.

The naming convention adopted for each test as CD-c-nam-s, where the term CD means Consolidated Drained triaxial test; the term “c” represents the net confining pressure; “n” indicates the number of drying and wetting cycles that the soil specimen has experienced; “a” indicates the drying or wetting paths that the soil specimen on where the test is conducted at; “m” indicates the matric suction where the CD triaxial test is conducted at; “s” indicates the soil type of the specimen. For example, CD-50-2w100-II means the CD triaxial test for the sand-kaolin specimen with the ratio of 35:65 which was conducted at matric suction of 100 kPa on the wetting path, under a net confining pressure of 50 kPa after the soil specimen has experienced two cycles of drying and wetting. Table 4-6, Table 4-7 and Table 4-8 show the testing programmes for the CD triaxial tests. Figure 4-20 and Figure 4-21 illustrate the testing programme for the CD triaxial tests under a net confining pressure of 50 kPa on drying and wetting paths of the first and second cycles.

Table 4-6: Testing programme for CD triaxial tests under 50 kPa net confining pressure.

Sand-kaolin ratio	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Matric suction, ($u_a - u_w$) (kPa)		
			0	100	300
15-85 (SK-5)	50	1	CD-50-1D0-I	CD-50-1D100-I CD-50-1W100-I	CD-50-1D300-I CD-50-1W300-I
	50	2		N/A N/A	CD-50-2D300-I CD-50-2W300-I
35-55 (SK-10)	50	1	CD-50-1D0-II	CD-50-1D100-II CD-50-1W100-II	CD-50-1D300-II CD-50-1W300-II
	50	2		CD-50-2D100-II CD-50-2W100-II	CD-50-2D300-II CD-50-2W300-II
55-45 (SK-17)	50	1	CD-50-1D0-III	CD-50-1D100-III CD-50-1W100-III	CD-50-1D300-III CD-50-1W300-III
	50	2		CD-50-2D100-III CD-50-2W100-III	CD-50-2D300-III CD-50-2W300-III
	50	3		N/A N/A	CD-50-3D300-III CD-50-3W300-III

Table 4-7: Testing programme for CD triaxial tests under 100 kPa net confining pressure

Sand-kaolin ratio	Net confining Pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Matric suction, ($u_a - u_w$) (kPa)		
			0	50	200
15-85 (SK-10)	100	1	CD-100-1D0-I	N/A N/A	N/A N/A
35-65 (SK-10)	100	1	CD-100-1D0-II	CD-100-1D50-II CD-100-1W50-II	CD-100-1D200-II CD-100-1W200-II
35-65 (SK-17)	100	1	CD-100-1D0-III	N/A N/A	N/A N/A

Table 4-8: Testing programme for CD triaxial tests under 200 kPa net confining pressure.

Sand-kaolin ratio	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Matric suction, ($u_a - u_w$) (kPa)		
			0	50	200
15-85 (SK-5)	200	1	CD-200-1D0-I	N/A N/A	N/A N/A
35-65 (SK-10)	200	1	CD-200-1D0-II	N/A N/A	N/A N/A
55-45 (SK-17)	200	1	CD-200-1D0-III	N/A N/A	N/A N/A

Figure 4-20: Illustration of CD and permeability tests for sand-kaolin mixtures with ratio of 35:65 on first cycle drying and wetting paths under 50 kPa net confining pressure.

Figure 4-21: Illustration of CD and permeability tests for sand-kaolin mixtures with ratio of 35:65 on second cycle drying and wetting paths under 50 kPa net confining pressure.

4.6.3 Permeability Triaxial Tests

A series of saturated and unsaturated permeability tests were conducted at different matric suctions to determine the permeability characteristics of a soil under single and multi-cycle drying and wetting processes. The naming convention is adopted for each test as UP-c-nam-s, where the term UP means unsaturated permeability test; the term “c” represents the net confining pressure; “n” indicates the number of drying and wetting cycles that the soil specimen has experienced; “a” indicates the drying or wetting paths that the soil specimen on when the test is conducted; “m” indicates the matric suction where the test is conducted at; “s” indicates the soil type of the specimen. For example, UP-50-1W50-I means the unsaturated permeability triaxial test for the sand-kaolin specimen with the ratio of 15:85 which was conducted at matric suction of 50 kPa on the wetting path, under a net confining pressure of 50 kPa after the soil specimen has experienced one cycle of drying and wetting. Table 4-9, Table 4-10, Table 4-11 show the testing programmes for the unsaturated permeability triaxial tests. Figure 4-20 and Figure 4-21 illustrate the testing programme for the unsaturated permeability triaxial tests under 50 kPa net confining pressure on the drying and wetting paths of the first and second cycles, respectively.

Table 4-9: Testing programme for permeability tests under 50 kPa net confining pressure.

Sand-kaolin ratio	Net confining pressure ($\sigma_3 - u_a$) (kPa)	Number of cycles	Matric suction, ($u_a - u_w$) (kPa)		
			0	100	300
15-85 (SK-5)	50	1	UP-50-1D0-I	UP-50-1D100- I UP -50-1W100- I	UP -50-1D300- I UP -50-1W300- I
	50			UP -50-2D100- I N/A	UP -50-2D300- I UP -50-2W300- I
35-65 (SK-10)	50	1	UP-50-1D0-II	UP -50-1D100- II UP -50-1W100- II	UP -50-1D300- II UP -50-1W300- II
	50	2		UP -50-2D100- II UP -50-2W100- II	UP -50-2D300- II UP -50-2W300- II
55-45 (SK-17)	50	1	UP-50-1D0-III	UP -50-1D100- III UP -50-1W100- III	UP -50-1D300- III UP -50-1W300- III
	50	2		UP -50-2D100- III UP -50-2W100- III	UP -50-2D300- III UP -50-2W300- III
	50	3		UP -50-3D100- III N/A	UP -50-3D300- III UP -50-3W300- III

Table 4-10: Testing programme for permeability tests under 100 kPa net confining pressure.

Sand-kaolin ratio	Net confining pressure (σ_3-u_a) (kPa)	Number of cycles	Matric suction, (u_a-u_w) (kPa)		
			0	50	200
15-85 (SK-5)	100	1	UP -100-1D0-I	N/A N/A	N/A N/A
35-65 (SK-10)	100	1	UP -100-1D0- II	N/A N/A	N/A N/A
55-45 (SK-17)	100	1	UP -100-1D0- III	N/A N/A	N/A N/A

Table 4-11: Testing programme for permeability tests under 200 kPa net confining pressure.

Sand-kaolin ratio	Net confining pressure (σ_3-u_a) (kPa)	Number of cycles	Matric suction, (u_a-u_w) (kPa)		
			0	50	200
15-85	200	1	UP -200-1D0-I	N/A N/A	N/A N/A
35-65	200	1	UP -200-1D0- II	N/A N/A	N/A N/A
55-45	200	1	UP -200-1D0- III	N/A N/A	N/A N/A

CHAPTER 5 PRESENTATION OF RESULTS

5.1 INTRODUCTION

This chapter presents the results of the experiments that were conducted in this research. The soil selection results are presented first. Subsequently, the results of the basic soil properties, saturated consolidated drained (CD) and saturated consolidated undrained (CU) tests are presented. The results of SWCCs and CD triaxial tests under different matric suctions on different cycles of drying and wetting paths are presented in the subsequent section. The results of unsaturated permeability of the soils at different matric suctions are shown in the last part of this chapter.

5.2 SOIL SELECTION

5.2.1 Selection of SWCC Estimation Methods

As described in Section 4.3.1.1, a mixture between fine sand (Ottawa sand) and coarse kaolin (L2 grade kaolin) was selected for this research program in order to fulfill the criteria listed in Section 4.3.1. Before the percentage selection for each group of fine sands and coarse kaolin, the eight different SWCC estimation methods as listed in Section 3.3.2 were evaluated using ten sets of published soil data as described in Section 4.3.1.1. The grain size distributions of the ten sets of soil are shown in Figure 5-1 and Figure 5-3 as well as Figures C5.2.1-1, C5.2.1-4, C5.2.1-6 and C5.2.1-8 to C5.2.1-12 in APPENDIX C5.2. One set of SWCC estimations using the eight methods are illustrated in Figure C5.2.1-1 to Figure C5.2.1-3 of APPENDIX C5.2. It was found that the Fredlund and Wilson Estimation method (1997, 2002) and Scheinost Estimation method (1996) were the two methods that provide reasonable good agreement between the estimated and measured SWCCs for most of the selected soil data. Figure 5-1 to Figure 5-4 illustrate the estimation of SWCCs using Fredlund and Wilson estimation method (1997, 2002) and Scheinost estimation method (1996) for two sets of soil data. As a result, these two methods were selected for estimating the SWCCs of different sand-kaolin mixtures to determine the suitable sand-kaolin mixtures that were used in this research.

Figure 5-1: Grain size distribution of kaolin extracted from Nishimura et al. (2002).

Figure 5-2: Prediction of SWCCs using Fredlund and Wilson PTF and Scheinost PTF for kaolin extracted from Nishimura et al. (2002).

Figure 5-3: Grain size distribution of soil data (soil counter: 11282) extracted from Soil Vision.

Figure 5-4: Prediction of SWCCs using Fredlund and Wilson PTF and Scheinost PTF for soil data (soil counter: 11282) extracted from Soil Vision data base.

5.2.2 Selection of Suitable Soil Mixtures

The grain size distribution of the Ottawa sand as shown in Figure 5-5 was obtained using the dry sieving. Ottawa sand was sieved and separated into four groups of different soil particle sizes, >600 μm , 300~600 μm , 150~300 μm and 75~150 μm in order to control the particle size distribution of the soil mixture effectively and accurately. Figure 5-6 shows the grain size distribution of the coarse kaolin that was obtained using the hydrometer test. A number of sand-kaolin mixtures with different combinations of percentages needed from each group of fine sands and coarse kaolin were designed as listed in Table 5-1. The SWCCs of each designed mixture were estimated using Fredlund and Wilson Estimation (1997, 2002) and Scheinost Estimation (1996) methods. The estimated AEVs of all designed mixtures are summarized in Table 5-1.

Table 5-1: The summary of soil information of the designed sand-kaolin mixtures.

Soil name	Assumed dry density (Mg/m^3)	Kaolin (%)	Sand group of >600 μm	Sand group of 300 ~ 600 μm	Sand group of 150 ~ 300 μm	Sand group of 75 ~ 150 μm	AEV (kPa) (from estimation method 1)	AEV (kPa) (from estimation method 2)
SK-1	1.39	100.0	0.0	0.0	0.0	0.0	79.3	186.3
SK-2	1.43	95.0	0.0	0.0	2.5	2.5	74.3	127.4
SK-3	1.47	90.0	0.0	2.5	2.5	5.0	67.2	87.5
SK-4	1.51	85.0	0.0	2.5	5.0	7.5	61.4	68.0
SK-5	1.51	85.0	2.5	5.0	5.0	2.5	61.3	65.9
SK-6	1.51	85.0	0.0	5.0	7.5	2.5	61.4	68.6
SK-7	1.55	80.0	2.5	2.5	5.0	10.0	57.5	50.7
SK-8	1.59	75.0	2.5	5.0	7.5	10.0	54.6	38.9
SK-9	1.63	70.0	2.5	5.0	10	12.5	51.7	31.6
SK-10	1.67	65.0	2.5	7.5	10.0	15.0	47.2	25.5
SK-11	1.67	65.0	0.0	7.5	12.5	15.0	47.7	26.4
SK-12	1.67	65.0	0.0	0.0	15.0	20.0	47.8	26.6
SK-13	1.67	65.0	7.5	7.5	10.0	10.0	47.1	23.1
SK-14	1.71	60.0	5.0	7.5	12.5	15.0	43.6	20.6
SK-15	1.75	55.0	5.0	10.0	12.5	17.5	39.5	17.0
SK-16	1.79	50.0	5.0	10.0	15.0	20.0	48.0	14.6
SK-17	1.83	45.0	7.5	20.0	20.0	7.5	47.7	12.5
SK-18	1.83	45.0	10.0	17.5	22.5	5.0	49.8	12.0
SK-19	1.83	45.0	5.0	22.5	22.5	5.0	49.6	12.8
SK-20	1.87	40.0	5.0	15.0	20.0	20.0	19.4	10.9

Note: SWCC estimation 1 refers to Fredlund and Wilson Estimation Method while SWCC estimation 2 refers to Scheinost Estimation Method.

Figure 5-5: Grain-size distribution of Ottawa sand.

Figure 5-6: Grain-size distribution of coarse kaolin.

The mixture, SK-10, that comprises 35 % of sand and 65 % of kaolin with 2.5 %, 7.5 %, 10 % and 15 % for the sand groups of $>600\ \mu\text{m}$, $300\sim600\ \mu\text{m}$, $150\sim300\ \mu\text{m}$ and $75\sim150\ \mu\text{m}$, respectively, was firstly selected. Figure 5-7 and Figure 5-8 show the grain-size distribution and the estimated SWCCs of SK-10. All of the basic properties, permeability and SWCC tests were conducted on the selected soil mixtures. The experimental results are presented in Section 5.3.1. It was found that SK-10 fulfills the criteria as listed in Section 4.3.1 but it has a relatively high residual matric suction (higher than 1000 kPa).

In order to conduct a more general and comprehensive study on the hysteresis effects on mechanical behaviour of unsaturated soils, additional two mixtures were selected and used in this research. The mixture with a higher kaolin content, SK-5, which comprises 15 % of sand and 85 % of kaolin with 2.5 %, 5.0 %, 5.0 % and 2.5 % for the sand groups of $>600\ \mu\text{m}$, $300\sim600\ \mu\text{m}$, $150\sim300\ \mu\text{m}$ and $75\sim150\ \mu\text{m}$, respectively, as well as the mixture with a higher sand content, SK-17, which comprises 55 % of sand and 45 % of kaolin with 7.5 %, 20.0 %, 20.0 % and 7.5 % for the sand groups of $>600\ \mu\text{m}$, $300\sim600\ \mu\text{m}$, $150\sim300\ \mu\text{m}$ and $75\sim150\ \mu\text{m}$, respectively, were selected. Figure 5-9, Figure 5-10, Figure 5-11 and Figure 5-12 show the grain-size distributions and the estimated SWCCs of SK-5 and SK-17. Basic properties, permeability and SWCC tests were also conducted on the selected soil mixtures. The experimental results of the soil properties of the selected sand-kaolin mixtures are shown in Section 5.3.1. It was found that both mixtures, SK-5 and SK-17, were also suitable to be used in this research. Therefore, three different sand-kaolin mixtures were selected and used throughout of this research. All of the specimens used in this research programme were prepared using the selected percentages of coarse kaolin and fine sand groups as stated, with the same specimen preparation procedures in order to ensure all the specimens are identical with the same characteristics.

On the other hand, as described in Section 4.3.1, besides of using the three different mixtures of Ottawa sand and coarse kaolin, another three different Ottawa sands with different grain-size distribution were also used to prepare the specimens for the SWCC tests in this research. The experimental results of the soil properties of the selected Ottawa sands are shown in Section 5.3.2. In other words, sand-kaolin mixtures were used to prepare the soil specimens for SWCC test, triaxial test and permeability test while the Ottawa sands were used to prepare the soil specimens for SWCC test only.

Figure 5-7: Grain size distribution of the selected sand-kaolin mixture, SK-10.

Figure 5-8: Prediction of SWCC using Fredlund and Wilson PTF and Scheinost PTF for the selected sand-kaolin mixture, SK-10.

Figure 5-9: Grain-size distribution of the selected sand-kaolin mixture, SK-5.

Figure 5-10: Prediction of SWCC using Fredlund and Wilson PTF and Scheinost PTF for the selected sand-kaolin mixture, SK-5.

Figure 5-11: Grain-size distribution of the selected sand-kaolin mixture, SK-17.

Figure 5-12: Prediction of SWCC using Fredlund and Wilson PTF and Scheinost PTF for the selected sand-kaolin mixture, SK-17.

5.3 SOIL CHARACTERISTICS

5.3.1 Sand-Kaolin Mixture Characteristics

Three different sand-kaolin mixtures, SK-5, SK-10 and SK-17, were used in this study. The grain size distribution of SK-5, SK-10 and SK-17 are shown in Figure 5-13. SK-5 comprises 15 % of sand and 85 % of kaolin, SK-10 comprises 35 % of sand and 65 % of kaolin and SK-17 comprises 55 % of sand and 45 % of kaolin. The specific gravity of all mixtures, SK-5, SK-10 and SK-17, is 2.67. The plasticity index of SK-5, SK-10 and SK-17 are 19.9, 18.6 and 13.1, respectively. The SK-5, SK-10 and SK-17 are classified as silt with high plasticity (MH), clay with low plasticity (CL) and clay with low plasticity (CL), respectively, according to the Unified Soil Classification System (ASTM D2487-93, as shown in Figure 5-14. The compaction curves of SK-5, SK-10 and SK-17 are shown in Figure 5-15. The maximum dry density, ρ_{dmax} , of SK-5, SK-10 and SK-17 are 1.50 Mg/m³, 1.67 Mg/m³ and 1.86 Mg/m³, respectively, while the optimum water content, w_{opt} , of SK-5, SK-10 and SK-17 are 25 %, 19 % and 14 %, respectively. Table 5-2 summarizes the soil properties of SK-5, SK-10 and SK-17 that were used in this research programme.

Figure 5-13: Grain size distribution of sand-kaolin mixtures, SK-5, SK10 and SK-17.

Figure 5-14: Classification of SK-5, SK-10 and SK-17 in the Unified Soil Classification System plasticity chart (ASTM D 2487-93, 1997).

Figure 5-15: Compaction curves of SK-5, SK-10 and SK-17.

All sand-kaolin specimens used for SWCC test, permeability test and triaxial test were prepared at the maximum dry density and the optimum water content using static compaction method as described 4.3.3.1.

Table 5-2: Soil properties of SK-5, SK-10 and SK-17.

Characteristics of sand-kaolin mixture	SK-5	SK-10	SK-17
Specific gravity, G_s	2.66	2.67	2.66
Maximum dry density, ρ_{dmax} (Mg/m ³)	1.50	1.67	1.86
Optimum water content, w_{opt} (%)	25	19	14
Liquid limit, w_L (%)	51.1	41.9	29.3
Plastic limit, w_P (%)	31.2	23.3	16.2
Plasticity Index, I_p (%)	19.9	18.6	13.1
Mixture ratio (sand: kaolin)	15:85	35:65	55:45
Ottawa sand (%)	15	35	55
Kaolin (%)	85	65	45
Sand (>75 μ m)	15.0	35.0	55.0
Silt (2 ~75 μ m) (%)	73.6	44.5	39.0
Clay (<2 μ m) (%)	11.4	20.5	6.0
D_{60} (μ m)	12.5	10.0	203.0
D_{50} (μ m)	10.5	6.0	130.0
D_{30} (μ m)	6.0	2.7	12.0
D_{10} (μ m)	1.7	1.3	3.7
C_u (D_{60}/D_{10})	7.353	7.692	54.865
C_c ($((D_{30})^2/(D_{60}D_{10}))$)	1.694	0.561	0.192
Unified Soil Classification System, USCS	MH	CL	CL
k_s (m/s) @ $(\sigma - u_w) = 50$ kPa	6.02×10^{-9}	7.67×10^{-9}	1.63×10^{-8}
k_s (m/s) @ $(\sigma - u_w) = 100$ kPa	5.33×10^{-9}	4.85×10^{-9}	1.33×10^{-8}
k_s (m/s) @ $(\sigma - u_w) = 200$ kPa	4.70×10^{-9}	3.60×10^{-9}	1.22×10^{-8}
c' (kPa)	29.5	8.5	8.2
ϕ' (°)	26.8	26.9	33.2

A multi-stage saturated CD triaxial test and two multi-stage CU triaxial tests were conducted on the statically compacted SK-10 specimens. The SK-10 specimens were tested in the triaxial apparatus as described in Section 4.3.4 under different net confining pressures of 50 kPa, 100 kPa and 200 kPa. The multi-stage CD triaxial test results of SK-10 specimens are presented in Figure 5-16 and Figure C5.3.1-1. Figure 5-17 and Figure C5.3.1-2 show the multi-stage CU triaxial test results of SK-10 specimens with shearing

rate of 0.05 mm/min while Figure 5-18 and Figure C5.3.1-3 show the multi-stage CU triaxial test results with shearing rate of 0.009 mm/min. The Mohr circles of all saturated triaxial tests on the SK-10 specimens and the Mohr-Coulomb failure envelopes of the SK-10 specimens are summarized in Figure 5-96 at Section 5.6.1. As shown in Figure 5-96, the c' and ϕ' for the SK-10 specimens are 8.5 kPa and 26.9°, respectively.

Single-stage saturated CD triaxial tests were conducted on the SK-5 and SK-17 specimens with the shearing rate of 0.0009 mm/min. Figure 5-19 shows the stress–strain curves under different net confining pressures, i.e. 50 kPa, 150 kPa and 300 kPa, of the SK-5 specimens. Figure 5-20 shows the stress–strain curves under different net confining pressures, i.e. 50 kPa, 100 kPa and 150 kPa, of the SK-17 specimens. Figure C5.3.1-4 and Figure C5.3.1-5 present the relationship between q' and p' for the SK-5 and SK-17 specimens, respectively. Figure 5-95 and Figure 5-97 of Section 5.6.1 show the Mohr circles obtained from the CD triaxial tests and the Mohr-Coulomb failure envelopes for the SK-5 and SK-17 specimens, respectively. It was found that the c' and ϕ' for the SK-5 specimens are 29.5 kPa and 26.8°, respectively, while the c' and ϕ' for the SK-17 specimens are 8.2 kPa and 33.2°, respectively.

Figure 5-16: Stress-strain curves for SK-10 obtained from the multistage CD triaxial test with shearing rate of 0.0009 mm/min.

Figure 5-17: Stress-strain curves for SK-10 obtained from the saturated multistage CU triaxial test with shearing rate of 0.05 mm/min.

Figure 5-18: Stress-strain curves for SK-10 obtained from the saturated multistage CU triaxial test with shearing rate of 0.009 mm/min.

Figure 5-19: Stress-strain curves for SK-5 obtained from the saturated single-stage CD triaxial tests with shearing rate of 0.0009 mm/min.

Figure 5-20: Stress-strain curves for SK-17 obtained from the saturated single-stage CD triaxial tests with shearing rate of 0.0009 mm/min.

5.3.2 Sand Characteristics

Three different sands, Sand IV, Sand V and Sand VI were used for SWCC tests in this study. The grain-size distributions of Sand IV, V and VI are shown in Figure 5-21. Table 5-3 summarizes the properties of Sand IV, Sand V and Sand VI.

Figure 5-21: Grain-size distribution of Sand IV, V and VI.

Table 5-3: Soil properties of Sand IV, Sand V and Sand VI.

Characteristics of sand-kaolin mixture	Sand IV	Sand V	Sand VI
Specific gravity, G_s	2.66	2.66	2.66
Minimum void ratio, e_{min}	0.50*	0.50*	0.50*
Maximum void ratio, e_{max}	0.83*	0.83*	0.83*
D_{10} (mm)	0.125	0.150	0.232
D_{30} (mm)	0.202	0.235	0.324
D_{50} (mm)	0.300	0.325	0.367
D_{60} (mm)	0.375	0.377	0.382
Uniformity coefficient, C_u	3.00	2.51	1.66
Coefficient of curvature, C_c	0.87	0.98	0.18

Note: “ * ” denotes the averaged values from literatures due to the insufficient of the sand group of 75~150 μ m .

5.4 SOIL-WATER CHARACTERISTIC CURVE

Series of SWCC tests were conducted on six different soils, i.e. three different sand-kaolin mixtures and three different sands. The multi-cycled drying and wetting SWCCs of the six different soils were obtained and presented in this section. Generally, the SWCCs obtained in this research are categorized into SWCCs under zero net confining pressure as presented in Section 5.4.1 and SWCCs under a net confining pressure as presented in Section 5.4.2. All of the drying and wetting SWCCs were fitted using Fredlund and Xing (1994) equation with the correction factor, $C(\psi)$, was taken as 1, as suggested by Leong and Rahardjo (1997b). The fitting parameters of a_d , n_d and m_d for the 1st cycle drying SWCC of soil as well as the fitting parameters of a_w , n_w and m_w for the 1st cycle wetting SWCC of soil are also presented.

5.4.1 SWCC under Zero Net Confining Pressure

The SWCCs of the statically compacted SK-5, SK-10 and SK-17 specimens as well as the SWCCs of the sand IV, sand V and sand VI were obtained using Tempe cell, pressure plates and modified Tempe cell. Both experimental and fitted drying and wetting SWCCs of single and multi-cycle of the soils are presented in Sections 5.4.1.1 to 5.4.1.4.

5.4.1.1 SWCC of SK-5 (Sand-Kaolin Ratio of 15:85)

The 1st cycle drying and wetting SWCCs of the statically compacted SK-5 specimen are presented in Figure 5-22 and Figure 5-23, respectively. As shown, the AEV and residual matric suction, $(u_a - u_w)_r$, are 30 kPa and 1000 kPa, respectively. The saturated volumetric water content, θ_s , and the residual volumetric water content, θ_r , are 0.503 and 0.174, respectively. On the other hand, the wetting saturated point, $(u_a - u_w)_{bw}$, and water-entry value, $(u_a - u_w)_w$, are 11 kPa and 550 kPa, respectively. The wetting saturated volumetric water content, θ_{sw} , is 0.441 while the water-entry volumetric water content, θ_{rw} , is 0.208. The fitting parameters of a_d , n_d and m_d for the 1st cycle drying SWCC are 68.946 kPa, 1.412 and 0.793, respectively, while the fitting parameters of a_w , n_w and m_w for the 1st cycle wetting SWCC are 22.180 kPa, 1.284 and 0.520,

Figure 5-22: 1st cycle drying SWCC of SK-5 under zero net confining pressure. Figure 5-23: 1st cycle wetting SWCC of SK-5 under zero net confining pressure.

After the SK-5 specimen experienced the 1st cycle drying and wetting processes, the SWCC test was continued with the 2nd cycle drying and wetting processes. The 2nd cycle drying path was started from and ended at matric suction of 1 kPa and 490 kPa, respectively. The 2nd cycle wetting path was started from and ended at matric suctions of 490 kPa and 10 kPa, respectively. Figure 5-24 shows the 2nd cycle drying and wetting SWCCs of the SK-5 specimen with initial dry density of 1.50 Mg/m³.

Figure 5-24: 2nd cycle SWCC of SK-5 under zero net confining pressure.

The SWCC test was then continued with the 3rd cycle drying and wetting processes. Figure 5-25 presents the 3rd cycle drying and wetting SWCCs of the SK-5 specimen with initial dry density of 1.50 Mg/m³. The 3rd cycle drying path was started from and ended at matric suction of 10 kPa and 490 kPa, respectively. The 3rd cycle wetting path was started from and ended at matric suctions of 490 kPa and 10 kPa, respectively. Upon three cycles of drying and wetting processes completed, the specimen was measured and the water content of the specimen was determined. Figure C5.4.1-1 and Figure C5.4.1-2 (APPENDIX C5.4) show the multi-cycled SWCCs in term of gravimetric water content and degree of saturation, respectively, with respect to matric suction. Figure C5.4.1-3 (APPENDIX C5.4) shows the volume change of soil on the multi-cycled drying and wetting paths.

Figure 5-25: 3rd cycle SWCC of SK-5 under zero net confining pressure.

5.4.1.2 SWCC of SK-10 (Sand-Kaolin Ratio of 35:65)

Figure 5-26 and Figure 5-27 show the 1st cycle drying and wetting SWCCs, respectively, of the statically compacted SK-10 specimens. The soil starts to desaturate significantly when the matric suction exceeds the AEV of 41 kPa as shown in Figure 5-26. Meanwhile, the volumetric water content decreases gradually when matric suction exceeds the residual matric suction, $(u_a - u_w)_r$, of 1200 kPa. The saturated volumetric water content, θ_s , and residual volumetric water content, θ_r , on the drying path are 0.466 and 0.273, respectively. The wetting saturated point, $(u_a - u_w)_{bw}$, and water-entry value, $(u_a - u_w)_w$, are 13 kPa and 730 kPa, respectively, as shown in Figure 5-27. The wetting saturated volumetric water content, θ_{sw} , and water-entry volumetric water content, θ_{rw} , on the wetting path are 0.452 and 0.302, respectively. The fitting parameters of a_d , n_d and m_d for the 1st cycle drying SWCC are 81.829 kPa, 1.578 and 0.363, respectively. On the other hand, the fitting parameters of a_w , n_w and m_w for the 1st cycle wetting SWCC are 21.123 kPa, 1.282 and 0.266, respectively. Figure C5.4.1-4, Figure C5.4.1-5 and Figure C5.4.1-6 (APPENDIX C5.4) show the degree of saturation, gravimetric water content and volume change, respectively, of the SK-10 specimens, on the multi-cycled drying and wetting paths.

 Figure 5-26: 1st cycle drying SWCC of SK-10 under zero net confining pressure.

 Figure 5-27: 1st cycle wetting SWCC of SK-10 under zero net confining pressure.

Upon the 1st cycle drying and wetting processes were completed, the SWCC test was continued with the 2nd cycle drying and wetting processes. The 2nd cycle drying path was started from and ended at matric suction of 0.1 kPa and 490 kPa, respectively, while the 2nd cycle wetting path was started from and ended at matric suctions of 490 kPa and 10 kPa, respectively. Figure 5-28 shows the 2nd cycle drying and wetting SWCCs of the SK-10 specimen. After the two cycles of drying and wetting processes completed, the SWCC test was terminated and the water content of the specimen was determined.

Figure 5-28: 2nd cycle SWCC of SK-10 under zero net confining pressure.

5.4.1.3 SWCC of SK-17 (Sand-Kaolin Ratio of 55:45)

Statically compacted SK-17 specimens were prepared at different initial dry densities, i.e. maximum drying density (1.86 Mg/m³) and 95 % of maximum drying density of (1.77 Mg/m³), for the SWCC test under zero net confining pressure.

SWCC of SK-17 specimen with initial dry density of 1.86 Mg/m³

The 1st cycle drying and wetting SWCCs of the SK-17 specimens with initial dry density of 1.86 Mg/m³ are presented in Figure 5-29 and Figure 5-30, respectively. From the analysis, it was found that the AEV and residual matric suction, $(u_a - u_w)_r$, are 26 kPa

and 1000 kPa, respectively. The saturated volumetric water content, θ_s , and the residual volumetric water content, θ_r , are 0.359 and 0.135, respectively. On the other hand, the wetting saturated point, $(u_a - u_w)_{bw}$, and water-entry value, $(u_a - u_w)_w$, are 6.2 kPa and 340 kPa, respectively. The wetting saturated volumetric water content, θ_{sw} , is 0.325 while the water-entry volumetric water content, θ_{rw} , is 0.169 as shown in Figure 5-30. The fitting parameters of a_d , n_d and m_d for the first cycle drying SWCC are 64.757 kPa, 1.422 and 0.711, respectively, while the fitting parameters of a_w , n_w and m_w for the first cycle wetting SWCC are 11.150 kPa, 1.493 and 0.400, respectively. Figure 5-31 and Figure 5-32 show the drying and wetting SWCCs of 2nd cycle and 3rd cycle, respectively, of the SK-17 specimen. The ranges of the 2nd and 3rd cycles SWCCs of SK-17 specimens are similar to those of the 2nd and 3rd cycles SWCCs the SK-5 specimen as described earlier. Figure C5.4.1-7 and Figure C5.4.1-8 of APPENDIX C5.4 show the multi-cycled SWCCs of the SK-17 specimen in term of degree of saturation and gravimetric water content, respectively, with respect to matric suction. Figure C5.4.1-9 of APPENDIX C5.4 shows the volume change of the SK-17 specimen on the multi-cycled drying and wetting paths.

Figure 5-29: 1st cycle drying SWCC of SK-17 at initial dry density of 1.86 Mg/m³ under zero net confining pressure.

Figure 5-30: 1st cycle wetting SWCC of the SK-17 specimen at initial dry density of 1.86 Mg/m³ under zero net confining pressure.

Figure 5-31: 2nd cycle SWCC of the SK-17 specimen at initial dry density of 1.86 Mg/m³ under zero net confining pressure.

Figure 5-32: 3rd cycle SWCC of the SK-17 specimen at initial dry density of 1.86 Mg/m³ under zero net confining pressure.

SWCC of SK-17 specimen with initial dry density of 1.77 Mg/m³

The 1st cycle drying and wetting SWCCs of the SK-17 specimens with initial dry density of 1.77 Mg/m³ are presented in Figure 5-33. It was found that the saturated volumetric water content, θ_s and the wetting saturated volumetric water content, θ_{sw} , are 0.358 and 0.312, respectively, while the AEV and the wetting saturated point, $(u_a - u_w)_{bw}$, are 23 kPa and 5.4 kPa, respectively. The fitting parameters of a_d , n_d and m_d for the 1st cycle drying SWCC are 71.991 kPa, 1.246 and 0.933, respectively, while the fitting parameters of a_w , n_w and m_w for the 1st cycle wetting SWCC are 10.925 kPa, 1.537 and 0.441, respectively. The drying and wetting SWCCs of the 2nd cycle and 3rd cycle of the SK-17 specimen with initial dry density of 1.77 Mg/m³ are presented in Figure 5-34 and Figure 5-35, respectively. After the 3rd cycle wetting process, the SWCC test was terminated and the water content of the specimen was obtained. Figure C5.4.1-10 and Figure C5.4.11 show the multi-cycled SWCCs of the SK-17 specimen in term of degree of saturation and gravimetric water content, respectively, with respect to matric suction. Figure C5.4.1-12 shows the volume change of the SK-17 specimen on the multi-cycled drying and wetting paths.

Figure 5-33: 1st cycle drying and wetting SWCCs of SK-17 specimen at initial dry density of 1.77 Mg/m³ under zero net confining pressure.

Figure 5-34: 2nd cycle SWCC of the SK-17 specimen at initial dry density of 1.77 Mg/m³ under zero net confining pressure.

Figure 5-35: 3rd cycle SWCC of the SK-17 specimen at initial dry density of 1.77 Mg/m^3 under zero net confining pressure.

5.4.1.4 SWCC of Sand IV, Sand V and Sand IV

Three different sands with different grain-size distributions and different void ratios were used to obtain multi-cycled drying and wetting SWCCs under zero net confining pressure. The testing apparatus, set-up and procedures are elaborated in Chapter 4. The SWCC SWCC-0-4D40.40.1-IV and SWCC-0-4D40.40.1-VI, for the sand IV (initial void ratio, $e_o = 0.577$) and sand VI ($e_o = 0.590$), respectively, were ended after the 4th drying process of the soil specimens while the SWCC test, SWCC-0-3W1.40.1-V for sand V ($e_o = 0.588$) were ended after the 3rd wetting process of the specimen. Therefore, three complete cycles drying and wetting SWCCs with 4th cycle drying SWCC were obtained for sand IV and VI while three complete cycles drying and wetting SWCCs were obtained for the sand V. Besides, another SWCC test, SWCC-0-1W0.2.40.2-V, for Sand V ($e_o = 0.509$) was conducted in order to obtain the 1st cycle drying and wetting SWCC. All SWCCs of sand IV, sand V and sand IV are presented in Figure 5-36, Figure 5-37, Figure 5-38 and Figure 5-39.

Figure 5-36: Multi-cycled drying and wetting SWCCs of Sand IV ($e_o = 0.577$). Figure 5-37: Multi-cycled drying and wetting SWCCs of Sand V ($e_o = 0.588$).

Figure 5-38: Drying and wetting SWCCs of Sand V ($e_o = 0.509$). Figure 5-39: Multi-cycled drying and wetting SWCCs of Sand IV ($e_o = 0.590$).

5.4.2 SWCC under Net Confining Pressure

The SWCC of the statically compacted SK-5, SK-10 and SK-17 specimens under net confining pressure were obtained using the modified triaxial apparatus. The multi-cycled SWCCs of these soils are presented in Sections 5.4.2.1 to 5.4.2.3.

5.4.2.1 SWCC of SK-5 (Sand-Kaolin Ratio of 15:85) under Confining Pressure

1st cycle SWCC of SK-5 under 50 kPa net confining pressure

Figure 5-40 shows the 1st cycle SWCCs of the specimens. The AEV of the soils is 107 kPa while the wetting saturated point, $(u_a - u_w)_{bw}$, of the soils is 31 kPa. Figure 5-41, Figure C5.4.2-1 and Figure C5.4.2-2 show the void ratio, water content and degree of saturation, respectively, of the specimens on the 1st cycle drying and wetting paths.

2nd cycle SWCC of SK-5 under 50kPa net confining pressure

Figure 5-42 shows the 2nd cycle SWCC of the specimens. Figure 5-43, Figure C5.4.2-3 and Figure C5.4.2-4 show the void ratio, water content and degree of saturation, respectively, of the specimens on the 2nd cycle drying and wetting paths.

Figure 5-40: 1st cycle drying and wetting SWCCs of SK-5 specimens under a net confining pressure of 50 kPa.

Figure 5-41: The void ratio of the SK-5 specimens on the drying and wetting paths of the 1st cycle under a net confining pressure of 50 kPa.

Figure 5-42: 2nd cycle drying and wetting SWCCs of SK-5 specimens under a net confining pressure of 50 kPa.

Figure 5-43: The void ratio of the SK-5 specimens on the drying and wetting paths of the 2nd cycle under a net confining pressure of 50 kPa.

5.4.2.2 SWCC of SK-10 (Sand-Kaolin Ratio of 35:65) under Confining Pressure

1st cycle SWCC of SK-10 under 50 kPa net confining pressure

Figure 5-44 shows the 1st cycle SWCCs of the specimens while Figure 5-45 shows the void ratio of the specimens on the 1st cycle drying and wetting paths. The AEV and the wetting saturated point, $(u_a - u_w)_{bw}$, of the soils are 100 kPa and 29 kPa, respectively. The water content and degree of saturation of the specimens on the 1st cycle drying and wetting paths are shown in Figure C5.4.2-5 and Figure C5.4.2-6 of APPENDIX C5.4.

2nd cycle SWCC of SK-10 under 50 kPa net confining pressure

Figure 5-46 shows the 2nd cycle SWCC of the specimens while Figure 5-47 shows the void ratio of the specimens on the 2nd cycle drying and wetting paths. The water content and degree of saturation of the specimens on the 2nd cycle drying and wetting paths are shown in Figure C5.4.2-7 and Figure C5.4.2-8 of APPENDIX C5.4.

1st cycle SWCC of SK-10 under 100 kPa net confining pressure

Figure 5-48 shows the 1st cycle drying and wetting SWCCs of the specimens. Figure 5-49 shows the void ratio of the specimens on the 1st cycle drying and wetting paths. The AEV and the wetting saturated point, $(u_a - u_w)_{bw}$, of the soils are 133 kPa and 30 kPa, respectively. The water content and degree of saturation of the specimens on the 1st cycle drying and wetting paths are shown in Figure C5.4.2-9 and Figure C5.4.2-10 of APPENDIX C5.4.

Figure 5-44: 1st cycle drying and wetting SWCCs of SK-10 specimens under a net confining pressure of 50 kPa.

Figure 5-45: The void ratio of the SK-10 specimens on the drying and wetting paths of the 1st cycle under a net confining pressure of 50 kPa.

Figure 5-46: 2nd cycle drying and wetting SWCCs of SK-10 specimens under a net confining pressure of 50 kPa.

Figure 5-47: The void ratio of the SK-10 specimens on the drying and wetting paths of the 2nd cycle under a net confining pressure of 50 kPa.

Figure 5-48: Drying and wetting SWCCs of SK-10 specimens under a net confining pressure of 100 kPa.

Figure 5-49: The void ratio of the SK-10 specimens on the drying and wetting paths under a net confining pressure of 100 kPa.

5.4.2.3 SWCC of SK-17 (Sand-Kaolin Ratio of 55:45) under Confining Pressure

1st cycle SWCC of SK-17 under 50 kPa net confining pressure

Figure 5-50 shows the 1st cycle SWCCs of the specimens. The AEV of the soils is 95 kPa while the wetting saturated point, $(u_a - u_w)_{bw}$, of the soils is 25 kPa. The void ratio, water content and degree of saturation of the specimens on the drying and wetting paths are shown in Figure 5-51, Figure C5.4.3-11 and Figure C5.4.3-12, respectively.

2nd cycle SWCC of SK-17 under 50 kPa net confining pressure

Figure 5-52 shows the 2nd cycle SWCC of the specimens. The void ratio, water content and degree of saturation of the specimens on the drying and wetting paths are shown in Figure 5-53, Figure C5.4.3-13 and Figure C5.4.3-14, respectively.

3rd cycle SWCC of SK-17 under 50 kPa net confining pressure

Figure 5-54 shows the 3rd cycle SWCCs of the specimens. The void ratio, water content and degree of saturation of the specimens on the drying and wetting paths are shown in Figure 5-55, Figure C5.4.3-15 and Figure C5.4.3-16, respectively.

Figure 5-50: 1st cycle drying and wetting SWCCs of SK-17 specimens under a net confining pressure of 50 kPa.

Figure 5-51: The void ratio of the SK-17 specimens on the drying and wetting paths of the 1st cycle under a net confining pressure of 50 kPa.

Figure 5-52: 2nd cycle drying and wetting SWCCs of SK-17 specimens under a net confining pressure of 50 kPa.

Figure 5-53: The void ratio of the SK-17 specimens on the drying and wetting paths of the 2nd cycle under a net confining pressure of 50 kPa.

Figure 5-54: 3rd cycle drying and wetting SWCCs of SK-17 specimens under a net confining pressure of 50 kPa.

Figure 5-55: The void ratio of the SK-17 specimens on the drying and wetting paths of the 3rd cycle under a net confining pressure of 50 kPa.

5.5 CONSOLIDATED DRAINED TRIAXIAL TEST RESULTS

Series of unsaturated CD triaxial tests were conducted on the statically compacted sand-kaolin specimens using the modified triaxial apparatus. All specimens were initially saturated and consolidated under a net confining pressure as described in Section 4.5.3.1 and 4.5.3.2. Subsequently, the specimen was equalized to the targeted matric suction by following the designated matric suction incremental and decremental steps. Finally, the specimen was sheared at a constant rate of 0.0009 mm/min until the defined failure occurred (Section 4.5.4.2). The shearing of specimen was conducted at the designated matric suction on the drying or wetting paths that described in the SWCC figures of Section 5.4.2. The sign conventions for the volumetric strain that are expressed in all related figures are positive sign for dilation (volume increase) and negative sign for compression (volume decrease). Figure 5-56 shows some of the SK-5, SK-10 and SK-17 specimens at the end of the unsaturated CD triaxial tests.

Figure 5-56: (a) SK-5 specimen, (b) SK-10 and (c) SK-17 specimens after failure.

5.5.1 Soil Mixture with Sand-Kaolin Ratio of 15:85

A series of unsaturated CD triaxial tests were conducted on the statically compacted SK-5 specimens under a net confining pressure of 50 kPa. All of the SK-5 specimens reached saturated condition ($B > 0.95$) when the cell pressure of 400 kPa and back pressure of 390

kPa were applied. The experimental results of the SK-5 specimens from unsaturated CD triaxial tests are presented in the following Sections 5.5.1.1 to 5.5.1.4.

5.5.1.1 Characteristic of the SK-5 Specimens during Shearing on the 1st Cycle Drying Path under Net Confining Pressure of 50 kPa

Table 5-4 summarizes the soil conditions of the SK-5 specimens at each stage of CD-50-1D100-I and CD-50-1D300-I. Table 5-5 summarizes the stress state at failure of the specimens obtained from the CD triaxial tests. Figure 5-57 and Figure 5-58 show the stress-strain curves and volume change characteristics, respectively, of the specimens during shearing. Figure 5-59 presents the ambient and water temperatures changes during shearing of the specimen at matric suction of 100 kPa. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.1-1 and Figure 5.5.1-2 of APPENDIX C5.5, respectively.

Table 5-4: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-5 specimens on the 1st cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ ($u_a - u_w$)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
First drying/ 100 kPa	e	0.800	0.948	0.896	0.837	0.823	0.843
	S (%)	84.45	99.87	9.87	98.45	97.95	96.88
	w (%)	25.30	35.45	33.51	30.86	30.18	30.57
	θ_w	0.375	0.486	0.472	0.449	0.442	0.443
First drying/ 300 kPa	e	0.803	0.946	0.899	0.821	0.822	0.845
	S (%)	83.80	99.79	99.79	53.33	48.83	47.35
	w (%)	25.20	35.36	3.60	16.39	15.03	14.98
	θ_w	0.373	0.485	0.472	0.240	0.220	0.217

Table 5-5: Stress-strain results of the SK-5 specimens at different matric suctions on the drying path of the 1st cycle under a net confining pressure of 50 kPa.

($\sigma_3 - u_w$) (kPa)	($u_a - u_w$) (kPa)	ε_{yf} (%)	$\sigma_{1f} - \sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	7.80	322.89	50.00	372.89	211.45	161.45
50.00	300.00	6.70	433.62	50.00	483.62	266.81	216.81

Figure 5-57: Stress-strain curves of the SK-5 specimens at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5-58: Total volume change characteristics of the SK-5 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5-59: Temperatures during shearing of the SK-5 specimen at matric suction of 100 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

5.5.1.2 Characteristic of the SK-5 Specimens during Shearing on the 1st Cycle Wetting Path under Net Confining Pressure of 50 kPa

Table 5-6 summarizes the soil conditions of the SK-5 specimens at each stage of CD-50-1W100-I and CD-50-1W300-I. Table 5-7 summarizes the stress state at failure of the specimens obtained from the unsaturated CD triaxial tests. Figure 5-60 shows the stress-strain curves of the specimen during shearing while Figure 5-61 shows the volume change characteristics and of the specimens during shearing. The water volume change characteristics and degree of saturation of the specimens throughout the shearing stage are shown in Figure 5.5.1-3 and Figure 5.5.1-4 of APPENDIX C5.5, respectively.

Table 5-6: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-5 specimens on the 1st cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
First wetting/ 100 kPa	e	0.803	0.949	0.902	0.828	0.833	0.849
	S (%)	83.13	99.91	99.90	58.51	59.97	59.62
	w (%)	25.00	35.51	33.73	18.14	18.72	18.95
	θ_w	0.370	0.486	0.474	0.265	0.273	0.274
First wetting/ 300 kPa	e	0.801	0.949	0.897	0.814	0.819	0.848
	S (%)	84.35	99.83	99.82	44.04	42.71	41.34
	w (%)	25.30	35.48	33.53	13.42	13.11	13.13
	θ_w	0.375	0.486	0.472	0.198	0.192	0.190

Table 5-7: Stress-strain results of the SK-5 specimens at different matrix suctions on the wetting path of the 1st cycle under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	6.60	296.24	50.00	346.24	198.12	148.12
50.00	300.00	5.26	387.97	50.00	437.97	243.99	193.99

Figure 5-60: Stress-strain curves of the SK-5 specimens at matrix suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-61: Total volume change characteristics of the SK-5 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

5.5.1.3 Characteristic of the SK-5 Specimen during Shearing on the 2nd Cycle Drying Path under Net Confining Pressure of 50 kPa

Table 5-8 summarizes the soil conditions of the SK-5 specimen at each stage of CD-50-CD-50-2D300-I. Table 5-9 summarizes the stress state at failure of the specimen obtained from the CD triaxial test. Figure 5-62 shows the stress-strain curve of the specimen. Figure 5-63 shows the volume change characteristics of the specimen throughout the shearing stage. Figure 5-64 shows the ambient and water temperatures changes during shearing of the specimen at matric suction of 300 kPa. The water volume change characteristics and degree of saturation of the specimen during shearing are shown in Figure 5.5.1-5 and Figure 5.5.1-6 (APPENDIX C5.5).

Table 5-8: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-5 specimen on the 2nd cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Second drying/ 300 kPa	e	0.807	0.944	0.899	0.821	0.825	0.842
	S (%)	83.35	99.86	99.84	49.63	46.36	45.25
	w (%)	25.20	35.31	33.63	15.26	14.33	14.28
	θ_w	0.372	0.485	0.473	0.224	0.210	0.207

Table 5-9: Stress-strain results of the SK-5 specimen at matrix suction of 300 kPa on the drying path of the 2nd cycle SWCC under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	300.00	6.46	421.06	50.00	471.06	260.53	210.53

Figure 5-62: Stress-strain curve of the SK-5 specimen at matrix suction of 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5-63: Total volume change characteristics of the SK-5 specimen during shearing at matric suction of 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5-64: Temperatures during shearing of the SK-5 specimen at matric suction of 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

5.5.1.4 Characteristic of the SK-5 Specimen during Shearing on the 2nd Cycle Wetting Path under Net Confining Pressure of 50 kPa

Table 5-10 summarizes the soil conditions of the SK-5 specimen at each stage of CD-50-CD-50-2D300-I. Table 5-11 summarizes the stress state at failure of the specimen obtained from the CD triaxial test. Figure 5-65 shows the stress-strain curve of the specimen. Figure 5-66 presents the volume change characteristics of the specimen throughout the shearing stage. The water volume change characteristics and degree of saturation of the specimen during shearing are shown in Figure 5.5.1-7 and Figure 5.5.1-8 of APPENDIX C5.5, respectively.

Table 5-10: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-5 specimen on the 2nd cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
Second wetting/ 300 kPa	e	0.812	0.951	0.897	0.815	0.814	0.850
	S (%)	83.24	99.84	99.85	44.75	43.09	41.28
	w (%)	25.30	35.57	33.55	13.67	13.14	13.15
	θ_w	0.373	0.487	0.472	0.201	0.193	0.190

Table 5-11: Stress-strain results of the SK-5 specimen at matric suction of 300 kPa on the wetting path of the 2nd cycle under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	300.00	5.89	405.56	50.00	455.56	252.78	202.78

Figure 5-65: Stress-strain curve of the SK-5 specimens at matric suction of 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-66: Total volume change characteristics of the SK-5 specimen during shearing at matric suction of 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

5.5.2 Soil Mixture with Sand-Kaolin Ratio of 35:65

Two series of unsaturated CD triaxial tests were conducted on the statically compacted SK-10 specimens under different net confining pressures, i.e. 50 and 100 kPa. As shown in Figure 5-67, the SK-10 specimens reached saturated condition ($B > 0.95$) when the cell pressure of 450 kPa and back pressure of 440 kPa were applied. The following sections (Sections 5.5.2.1 to 5.5.2.4) present the experimental results of the SK-10 specimens from the unsaturated CD triaxial tests.

5.5.2.1 Characteristics of the SK-10 Specimens during Shearing on the 1st Cycle Drying Path under Net Confining Pressure of 50 kPa

Table 5-12 summarizes the void ratios, water contents, degrees of saturation and volumetric water contents of the SK-10 specimens at each stage of the CD triaxial tests while Table 5-13 summarizes the stress states of the specimens at the failure obtained from the CD triaxial tests, i.e. CD-50-1D100-II and CD-50-1D300-II. Figure 5-68 presents the stress-strain relationship at the shearing stage while Figure 5-69 presents volume change characteristics of the specimens at the shearing stage. The water volume change characteristics and degree of saturation of the specimens throughout the shearing stage are shown in Figure 5.5.2-1 and Figure 5.5.2-2 of APPENDIX C5.5, respectively.

Table 5-12: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-10 specimens on the 1st cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
First drying/ 100 kPa	e	0.660	0.807	0.741	0.685	0.673	0.699
	S (%)	76.45	99.74	99.70	98.42	96.38	95.17
	w (%)	18.90	30.14	27.65	25.27	24.28	24.92
	θ_w	0.304	0.445	0.424	0.400	0.388	0.392
First drying/ 300 kPa	e	0.661	0.796	0.741	0.647	0.622	0.657
	S (%)	77.39	99.79	99.77	89.92	86.98	82.00
	w (%)	19.16	29.75	27.70	21.77	20.26	20.19
	θ_w	0.308	0.442	0.425	0.353	0.334	0.325

Table 5-13: Stress-strain results of the SK-10 specimens at different matrix suctions on the drying path of the 1st cycle under a net confining pressure of 50 kPa.

$(\sigma_3 - u_w)$ (kPa)	$(u_a - u_w)$ (kPa)	ε_{yf} (%)	$\sigma_{1f} - \sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	8.91	272.47	50.00	322.47	186.24	136.24
50.00	300.00	10.34	502.19	50.00	552.19	301.10	251.10

Figure 5-67: The B value of SK-10 specimens during saturation process.

Figure 5-68: Stress-strain curves of the SK-10 specimens at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5-69: Total volume change characteristics of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

5.5.2.2 Characteristics of the SK-10 Specimens during Shearing on the 1st Cycle Wetting Path under Net Confining Pressure of 50 kPa

Table 5-14 summarizes the soil condition of the SK-10 specimens at each stage of the CD triaxial tests while Table 5-15 summarizes the stress states of the specimens at the failure obtained from the CD triaxial tests, i.e. CD-50-1W100-II and CD-50-1W300-II. Figure 5-70 shows the stress-strain relationships of the specimens. Figure 5-71 presents the volume change characteristics of the specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.2-3 and Figure 5.5.2-4 of APPENDIX C5.5, respectively. The ambient temperature and the cell water temperature during shearing are shown in Figure 5-72.

Table 5-14: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-10 specimens on the 1st cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
First wetting/ 100 kPa	e	0.671	0.800	0.742	0.662	0.675	0.703
	S (%)	76.35	99.86	99.85	89.78	89.57	89.54
	w (%)	19.18	29.93	27.73	22.26	22.63	23.59
	θ_w	0.307	0.444	0.425	0.358	0.361	0.370
First wetting/ 300 kPa	e	0.663	0.801	0.738	0.633	0.629	0.671
	S (%)	77.82	99.82	99.82	84.06	81.71	76.89
	w (%)	19.31	29.93	27.59	19.93	19.24	19.32
	θ_w	0.310	0.444	0.424	0.326	0.315	0.309

Table 5-15: Stress-strain results of the SK-10 specimens at different matric suctions on the wetting path of the 1st cycle under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	6.18	264.12	50.00	314.12	182.06	132.06
50.00	300.00	7.05	453.89	50.00	503.89	276.95	226.95

Figure 5-70: Stress-strain curves of the SK-10 specimens at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-71: Total volume change characteristics of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-72: Temperatures during shearing of the SK-10 specimen at matric suction of 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

5.5.2.3 Characteristics of the SK-10 Specimens during Shearing on the 2nd Cycle Drying Path under Net Confining Pressure of 50 kPa

The soil conditions of the two SK-10 specimens at each stage of the CD triaxial tests, i.e. CD-50-1D100-II and CD-50-1D300-II, are summarized in Table 5-16. The stress states at failure of the specimens obtained from the CD triaxial tests are summarized in Table 5-17. Figure 5-73 shows the stress-strain curves obtained from the shearing stage. Figure 5-74 presents the volume change characteristics of the specimens during shearing. The water volume change characteristics and degree of saturation of the specimens throughout the shearing stage are shown in Figure 5.5.2-5 and Figure 5.5.2-6 of APPENDIX C5.5, respectively.

Table 5-16: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-10 specimens on the 2nd cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ ($u_a - u_w$)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Second drying/ 100 kPa	e	0.658	0.796	0.740	0.674	0.680	0.707
	S (%)	77.89	99.79	99.77	94.39	94.51	94.44
	w (%)	19.21	29.77	27.66	23.84	24.07	24.99
	θ_w	0.309	0.442	0.424	0.380	0.383	0.391
Second drying/ 300 kPa	e	0.640	0.794	0.736	0.645	0.650	0.671
	S (%)	79.22	99.83	99.80	86.24	81.16	78.32
	w (%)	19.00	29.68	27.53	20.84	19.76	19.68
	θ_w	0.309	0.442	0.423	0.338	0.320	0.314

Table 5-17: Stress-strain results of the SK-10 specimens at different matrix suctions on the drying path of the 2nd cycle under a net confining pressure of 50 kPa.

($\sigma_3 - u_w$) (kPa)	($u_a - u_w$) (kPa)	ε_{yf} (%)	$\sigma_{1f} - \sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	6.89	269.40	50.00	319.40	184.70	134.70
50.00	300.00	8.24	483.53	50.00	533.53	291.77	241.77

Figure 5-73: Stress-strain curves of the SK-10 specimens at matrix suctions of 100 kPa and 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5-74: Total volume change characteristics of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

5.5.2.4 Characteristics of the SK-10 Specimens during Shearing on the 2nd Cycle Wetting Path of SWCC under Net Confining Pressure of 50 kPa

The soil conditions of the SK-10 specimens at each stage of CD-50-1W100-II and CD-50-1W300-II are summarized in Table 5-18. The stress states at failure of the specimens obtained from the CD triaxial tests are summarized in Table 5-19. The stress strain curves and the total volume change of the specimens during shearing are presented in Figure 5-75 and Figure 5-76, respectively. Figure 5-77 shows the ambient and cell water temperatures throughout the shearing stage. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.2-7 and Figure 5.5.2-8 of APPENDIX C5.5, respectively.

Table 5-18: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-10 specimens on the 2nd cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Second wetting/ 100 kPa	e	0.653	0.792	0.741	0.651	0.657	0.680
	S (%)	77.68	99.84	99.83	90.74	91.36	90.82
	w (%)	19.00	29.60	27.71	22.12	22.46	23.12
	θ_w	0.307	0.441	0.425	0.358	0.362	0.368
Second wetting/ 300 kPa	e	0.660	0.799	0.745	0.640	0.644	0.667
	S (%)	77.29	99.79	99.78	84.32	81.49	79.21
	w (%)	19.11	29.86	27.84	20.22	19.64	19.79
	θ_w	0.307	0.443	0.426	0.329	0.319	0.317

Table 5-19: Stress-strain results of the SK-10 specimens at different matrix suctions on the wetting path of the 2nd cycle under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	5.21	266.82	50.00	316.82	183.41	133.41
50.00	300.00	8.08	471.05	50.00	521.05	285.53	235.53

Figure 5-75: Stress-strain curves of the SK-10 specimens at matrix suctions of 100 kPa and 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-76: Total volume change characteristics of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-77: Cell water and ambient temperatures during shearing of the SK-10 specimens at matric suction of 300 kPa on 2nd cycle wetting path under net confining pressure of 50 kPa.

5.5.2.5 Characteristics of the SK-10 Specimens during Shearing on the 1st Cycle Drying Path under Net Confining Pressure of 100 kPa

Table 5-20 summarizes the soil conditions of the SK-10 specimens at each stage of CD-100-1D50-II and CD-100-1D200-II. Table 5-21 summarizes the stress state at failure of the specimens obtained from the CD triaxial tests. Figure 5-78 presents the stress-strain curves of the specimens obtained from the CD triaxial tests. Figure 5-79 shows the volume change characteristics of the specimens during shearing. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.2-9 and Figure 5.5.2-10 of APPENDIX C5.5, respectively.

Table 5-20: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-10 specimens on the 1st cycle drying path under a net confining pressure of 100 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
First drying/ 50 kPa	e	0.662	0.799	0.699	0.690	0.685	0.701
	S (%)	77.44	100.00	100.00	97.64	95.98	94.63
	w (%)	19.20	29.91	26.16	25.23	24.62	24.84
	θ_w	0.309	0.444	0.411	0.399	0.390	0.390
First drying/ 200 kPa	e	0.6450	0.791	0.692	0.643	0.615	0.623
	S (%)	78.65	100.00	100.00	96.82	94.80	94.00
	w (%)	19.00	29.61	25.93	23.31	21.84	21.92
	θ_w	0.308	0.442	0.409	0.379	0.361	0.361

Table 5-21: Stress-strain results of the SK-10 specimens at different matric suctions on the drying path of the 1st cycle under a net confining pressure of 100 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
100.00	50.00	12.00	278.02	100.00	378.02	239.01	139.01
100.00	200.00	11.55	477.85	100.00	577.85	338.93	238.93

Figure 5-78: Stress-strain curves of the SK-10 specimens at matric suctions of 50 kPa and 200 kPa on the 1st cycle drying path under a net confining pressure of 100 kPa.

Figure 5-79: Total volume change characteristics of the SK-10 specimens during shearing at matric suctions of 50 kPa and 200 kPa on the 1st cycle drying path under a net confining pressure of 100 kPa.

5.5.2.6 Characteristics of the SK-10 Specimens during Shearing on the 1st Cycle Wetting Path under Net Confining Pressure of 100 kPa

The void ratios, water contents, degrees of saturation and volumetric water contents of the SK-10 specimens at each stage of CD-100-1W50-II and CD-100-1W200-II are summarized in Table 5-22. The stress states at failure of the specimens are summarized in Table 5-23. Figure 5-80 shows the stress-strain relationships of the specimens. Figure 5-81 shows the volume change characteristics of the specimens at the shearing stage. Figure 5.5.2-11 and Figure 5.5.2-12 of APPENDIX C5.5 present the water volume change characteristics and degree of saturation of the specimens at the shearing stage. The ambient temperatures (room temperature) throughout the shearing stage of the specimens at matric suction of 200 kPa and 50 kPa on the 1st cycle wetting path are presented in Figure 5-82 and Figure 5.5.2-13 of APPENDIX C5.5, respectively.

Table 5-22: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-10 specimens on the 1st cycle wetting path under a net confining pressure of 100 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
First wetting/ 50 kPa	e	0.637	0.770	0.697	0.644	0.653	0.655
	S (%)	79.62	99.84	99.82	93.41	93.86	96.03
	w (%)	19.00	28.81	26.07	22.53	22.96	23.57
	θ_w	0.310	0.434	0.410	0.366	0.371	0.380
First wetting/ 200 kPa	e	0.658	0.782	0.685	0.625	0.629	0.646
	S (%)	79.40	99.89	99.85	88.60	87.12	85.74
	w (%)	19.56	29.26	25.63	20.73	20.53	20.73
	θ_w	0.315	0.438	0.406	0.341	0.336	0.336

Table 5-23: Stress-strain results of the SK-10 specimens at different matric suctions on the wetting path of the 1st cycle under a net confining pressure of 100 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
100.00	50.00	6.43	273.48	100.00	373.48	236.74	136.74
100.00	200.00	7.61	445.38	100.00	545.38	322.69	222.69

Figure 5-80: Stress-strain curves of the SK-10 specimens at matric suctions of 50 kPa and 200 kPa on the 1st cycle wetting path under a net confining pressure of 100 kPa.

Figure 5-81: Total volume change characteristics of the SK-10 specimens during shearing at matric suctions of 50 kPa and 200 kPa on the 1st cycle wetting path under a net confining pressure of 100 kPa.

Figure 5-82: Ambient temperature during shearing of SK-10 specimen at matric suction of 200 kPa on the 1st cycle wetting path under a net confining pressure of 100 kPa.

5.5.3 Soil Mixture with Sand-Kaolin Ratio of 55:45

A series of unsaturated CD triaxial tests were conducted on the statically compacted SK-17 specimens at different matric suctions on the drying and wetting paths of different cycles under a net confining pressure of 50 kPa. All SK-17 specimens reached saturated condition ($B > 0.95$) when the cell pressure of 400 kPa and back pressure of 390 kPa were applied. The experimental results of the SK-17 specimens obtained from the CD triaxial tests are presented in Sections 5.5.3.1 to 5.5.3.6.

5.5.3.1 Characteristics of the SK-17 Specimens during Shearing on the 1st Cycle Drying Path under Net Confining Pressure of 50 kPa

Table 5-24 summarizes the soil conditions of the specimens at each stage of CD-50-1D100-III and CD-50-1D300-III. Table 5-25 summarizes the stress states at failure of the specimens. Figure 5-83 and Figure 5-84 show the stress-strain relationships and volume change characteristics of the specimens during shearing. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.3-1 and Figure 5.5.3-2, respectively (APPENDIX C5.5).

Table 5-24: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-17 specimens on the 1st cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ ($u_a - u_w$)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
First drying/ 100 kPa	e	0.440	0.530	0.499	0.473	0.468	0.499
	S (%)	84.89	99.89	99.88	98.19	98.33	88.15
	w (%)	14.00	19.83	18.68	17.39	17.22	16.48
	θ_w	0.260	0.346	0.333	0.315	0.313	0.294
First drying/ 300 kPa	e	0.449	0.531	0.504	0.461	0.466	0.491
	S (%)	83.21	99.70	99.66	53.48	48.79	45.04
	w (%)	14.00	19.85	18.83	9.24	8.51	8.28
	θ_w	0.258	0.346	0.334	0.169	0.155	0.148

Table 5-25: Stress-strain results of the SK-17 specimens at different matrix suctions on the drying path of the 1st cycle under a net confining pressure of 50 kPa.

($\sigma_3 - u_w$) (kPa)	($u_a - u_w$) (kPa)	ε_{yf} (%)	$\sigma_{1f} - \sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	5.22	341.46	50.00	391.46	220.73	170.73
50.00	300.00	4.59	483.48	50.00	533.48	291.74	241.74

Figure 5-83: Stress-strain curves of the SK-17 specimens at matrix suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5-84: Total volume change characteristics of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

5.5.3.2 Characteristics of the SK-17 Specimens during Shearing on the 1st Cycle Wetting Path under Net Confining Pressure of 50 kPa

Another two SK-17 specimens were sheared at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path. The soil conditions of the specimens at each stage of CD-50-1W100-III and CD-50-1W300-III are summarized in Table 5-26. Table 5-27 summarizes the stress state at failure of the specimens obtained from the CD triaxial tests. Figure 5-85 shows the stress-strain curves of the specimens from both tests. Figure 5-86 shows the total volume change behaviours of the specimens during shearing. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.3-3 and Figure 5.5.3-4 of APPENDIX C5.5, respectively.

Table 5-26: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-17 specimens on the 1st cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ ($u_a - u_w$)		Before saturation	After saturation	After consolidation	After matric suction equalization	At peak of shearing	End of shearing
First wetting/ 100 kPa	e	0.446	0.532	0.505	0.468	0.476	0.518
	S (%)	84.46	99.88	99.88	58.23	59.00	55.76
	w (%)	14.10	19.89	18.88	10.22	10.53	10.82
	θ_w	0.260	0.347	0.335	0.186	0.190	0.190
First wetting/ 300 kPa	e	0.441	0.530	0.501	0.460	0.466	0.495
	S (%)	84.72	99.76	99.76	45.41	44.07	41.30
	w (%)	14.00	19.79	18.72	7.83	7.68	7.66
	θ_w	0.259	0.345	0.333	0.143	0.140	0.137

Table 5-27: Stress-strain results of the SK-17 specimens at different matric suctions on the drying path of the 1st cycle under a net confining pressure of 50 kPa.

($\sigma_3 - u_w$) (kPa)	($u_a - u_w$) (kPa)	ε_{yf} (%)	$\sigma_{1f} - \sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	3.97	322.80	50.00	372.80	211.40	161.40
50.00	300.00	3.91	389.37	50.00	439.37	244.69	194.69

Figure 5-85: Stress-strain curves of the SK-17 specimens at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-86: Total volume change characteristics of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressures of 50 kPa.

5.5.3.3 Characteristics of the SK-17 Specimens during Shearing on the 2nd Cycle Drying Path under Net Confining Pressure of 50 kPa

Two SK-17 specimens were sheared at matric suctions of 100 kPa and 300 kPa on the 2nd cycle drying path. Table 5-28 lists the soil conditions of the specimens at each stage of CD-50-2D100-III and CD-50-2D300-III. Table 5-29 summarizes the stress state of the specimens at the failure obtained from the CD triaxial tests. Figure 5-87 shows the stress-strain relationships of the specimens during shearing while Figure 5-88 shows the total volume change characteristics of the specimens during shearing. The water volume change characteristics and degree of saturation of the specimens during shearing are shown in Figure 5.5.3-5 and Figure 5.5.3-6 of APPENDIX C5.5, respectively.

Table 5-28: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-17 specimens on the 2nd cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ ($u_a - u_w$)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Second drying/ 100 kPa	e	0.448	0.531	0.501	0.470	0.476	0.490
	S (%)	84.94	99.86	99.85	68.12	65.68	63.97
	w (%)	14.25	19.87	18.73	11.98	11.70	11.74
	θ_w	0.263	0.347	0.333	0.218	0.212	0.210
Second drying/ 300 kPa	e	0.447	0.528	0.500	0.461	0.466	0.489
	S (%)	84.87	99.85	99.86	49.33	46.32	43.69
	w (%)	14.21	19.75	18.70	8.52	8.08	8.01
	θ_w	0.262	0.345	0.333	0.156	0.147	0.144

Table 5-29: Stress-strain results of the SK-17 specimens at different matrix suctions on the drying path of the 2nd cycle under a net confining pressure of 50 kPa.

($\sigma_3 - u_w$) (kPa)	($u_a - u_w$) (kPa)	ε_{yf} (%)	$\sigma_{1f} - \sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	3.60	338.56	50.00	388.56	219.28	169.28
50.00	300.00	4.41	466.19	50.00	516.19	283.10	233.10

Figure 5-87: Stress-strain curves of the SK-17 specimens at matrix suctions of 100 kPa and 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5-88: Total volume change characteristics of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

5.5.3.4 Characteristics of the SK-17 Specimens during Shearing on the 2nd cycle Wetting Path of under Net Confining Pressure of 50 kPa

Table 5-30 summarizes the soil conditions of the two SK-17 specimens at each stage of the unsaturated CD triaxial tests, i.e. CD-50-2D100-III and CD-50-2D300-III. Table 5-31 summarizes the stress state at failure of the specimens obtained from the CD triaxial tests. Figure 5-89 and Figure 5-90 show the stress-strain curves and the total volume change characteristics, respectively, of the specimens during shearing. The water volume change characteristics of the specimens during shearing are shown in Figure 5.5.3-7 of APPENDIX C5.5. Figure 5.5.3-8 of APPENDIX C5.5 shows the degree of saturations of the specimens at the shearing stage.

Table 5-30: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-17 specimens on the 2nd cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Second wetting/ 100 kPa	e	0.448	0.534	0.506	0.468	0.476	0.518
	S (%)	84.42	99.92	99.90	58.51	59.00	55.96
	w (%)	14.16	20.00	18.94	10.25	10.51	10.85
	θ_w	0.261	0.348	0.336	0.186	0.190	0.191
Second wetting/ 300 kPa	e	0.450	0.532	0.503	0.460	0.463	0.492
	S (%)	84.72	99.80	99.78	46.27	45.76	42.68
	w (%)	14.28	19.88	18.81	7.97	7.93	7.86
	θ_w	0.263	0.346	0.334	0.146	0.145	0.141

Table 5-31: Stress-strain results of the SK-17 specimens at different matrix suctions on the wetting path of the 2nd under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	100.00	3.83	331.67	50.00	381.67	215.84	165.84
50.00	300.00	3.96	449.29	50.00	499.29	274.65	224.65

Figure 5-89: Stress-strain curves of the SK-17 specimens at matrix suctions of 100 kPa and 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-90: Total volume change characteristics of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle wetting path under a net confining pressures of 50 kPa.

5.5.3.5 Characteristics of the SK-17 Specimen during Shearing on the 3rd Cycle Drying Path under Net Confining Pressure of 50 kPa

The soil conditions of the SK-17 specimen at each stage of CD-50-3D300-III are summarized in Table 5-32. Table 5-33 summarizes the stress state of the specimen at the failure obtained from the CD triaxial tests. Figure 5-91 shows the stress-strain curves of the specimen during shearing while Figure 5-92 shows the total volume change characteristics of the specimen during shearing. The water volume change characteristics of the specimen during shearing are shown in Figure 5.5.3-9 of APPENDIX C5.5. Figure 5.5.3-10 of APPENDIX C5.5 shows the degree of saturations of the specimen throughout the shearing stage.

Table 5-32: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-17 specimen on the 3rd cycle drying path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Third drying/ 300 kPa	e	0.445	0.532	0.506	0.462	0.467	0.493
	S (%)	85.73	99.93	99.92	50.04	47.41	44.24
	w (%)	14.30	19.92	18.92	8.67	8.29	8.17
	θ_w	0.264	0.347	0.336	0.158	0.151	0.146

Table 5-33: Stress-strain results of the SK-17 specimen at matrix suction of 300 kPa on the drying path of the 3rd cycle under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	300.00	4.26	463.61	50.00	513.61	281.81	231.81

Figure 5-91: Stress-strain curve of the SK-17 specimen at matrix suction of 300 kPa on the 3rd cycle drying path under a net confining pressure of 50 kPa.

Figure 5-92: Total volume change characteristics of the SK-17 specimen during shearing at matric suction of 300 kPa on the 3rd cycle drying path under a net confining pressure of 50 kPa.

5.5.3.6 Characteristics of the SK-17 Specimen during Shearing on the 3rd Cycle Wetting Path under Net Confining Pressure of 50 kPa

Table 5-34 summarizes the soil conditions of the SK-17 specimen at each stage of CD-50-3W300-III. Table 5-35 summarizes the stress state at failure of the specimen obtained from the CD triaxial tests. Figure 5-93 shows the stress-strain curves of the specimen during shearing. Figure 5-94 shows the total volume change characteristics of the specimen at the shearing stage. The water volume change characteristics of the specimen throughout the shearing stage are shown in Figure 5.5-11 of APPENDIX C5.5. The degree of saturation of the specimen during shearing is shown in Figure 5.5-12 of APPENDIX C5.5.

Table 5-34: Void ratio (e), water contents (w), degrees of saturation (S) and volumetric water contents (θ_w) at different stages of the SK-17 specimen on the 3rd cycle wetting path under a net confining pressure of 50 kPa from the CD triaxial tests.

Stress state/ (u_a-u_w)		Before saturation	After saturation	After consolidation	After matrix suction equalization	At peak of shearing	End of shearing
Third wetting/ 300 kPa	e	0.450	0.531	0.501	0.460	0.463	0.495
	S (%)	83.45	99.90	99.84	45.53	43.75	40.47
	w (%)	14.05	19.88	18.74	7.85	7.59	7.50
	θ_w	0.259	0.347	0.333	0.143	0.138	0.134

Table 5-35: Stress-strain results of the SK-17 specimen at matrix suction of 300 kPa on the wetting path of the 3rd cycle under a net confining pressure of 50 kPa.

(σ_3-u_w) (kPa)	(u_a-u_w) (kPa)	ε_{yf} (%)	$\sigma_{1f}-\sigma_{3f}$ (kPa)	σ'_{3f} (kPa)	σ'_{1f} (kPa)	p'_f (kPa)	q_f (kPa)
50.00	300.00	3.96	446.42	50.00	496.42	273.21	223.21

Figure 5-93: Stress-strain curve of the SK-17 specimen at matrix suction of 300 kPa on the 3rd cycle wetting path under a net confining pressure of 50 kPa.

Figure 5-94: Total volume change characteristics of the SK-17 specimen during shearing at matric suction of 300 kPa on the 3rd cycle wetting path under a net confining pressures of 50 kPa.

5.6 MOHR-COULOMB FAILURE ENVELOPE FOR SAND-KAOLIN SPECIMENS

The CD triaxial test results of the statically compacted specimens were analyzed by drawing Mohr circles and failure envelopes. The Mohr Coulomb failure envelopes for the saturated specimens are presented in Section 5.6.1 while the extended Mohr Coulomb failure envelopes for the unsaturated specimens are presented in Section 5.6.2.

5.6.1 Mohr Coulomb Failure Envelope for Saturated Specimens

A series of CD triaxial test were conducted on the SK-5 specimens. The stress-strain results of the SK-5 specimens are presented in Section 5.3. Figure 5-95 shows the Mohr circles and the Mohr-Coulomb failure envelope, which was constructed with a straight line drawn through all the points of tangency of the Mohr circles, of the SK-5 specimens. The interception of the failure envelope and shear stress axis is the effective cohesion, c' , which is 29.5 kPa and the slope of the failure envelope is the effective frictional angle, which is 26.8° .

Figure 5-95: Mohr-Coulomb failure envelope for SK-5 obtained from saturated single-stage CD triaxial tests with shearing rate of 0.0009 mm/min.

One multi-stage CD triaxial test and two multi-stage CU triaxial tests were conducted on the SK-10 specimens. The stress-strain results of the SK-10 specimens are presented in Section 5.3. The Mohr circles for all triaxial test results of SK-10 specimens are summarized in Figure 5-96. A Mohr-Coulomb failure envelope with the effective cohesion, c' , of 8.5 kPa and the effective frictional angle, ϕ' , of 26.9° is shown in Figure 5-96.

Figure 5-96: Mohr-Coulomb failure envelope for SK-10 obtained from a multi-stage CD triaxial test with shearing rate of 0.0009 mm/min and two multi-stage CU triaxial tests with shearing rate of 0.009 mm/min and 0.05 mm/min.

(Note: 0.05, 0.009 and 0.0009 in the label refer to the shearing rate of 0.05 mm/min, 0.009 mm/min and 0.0009 mm/min, respectively. Stages 1, 2 and 3 refer to the first, second and third stage, respectively, of the multi-stage triaxial test.)

A series of CD triaxial tests were conducted on the SK-17 specimens. The stress-strain results of the SK-17 specimens are presented in Section 5.3. The Mohr circles and the Mohr-Coulomb failure envelopes for the SK-17 specimens were drawn and shown in Figure 5-97. The effective cohesion, c' , and the effective frictional angle, ϕ' , were found to be 8.2 kPa and 33.2° , respectively.

Figure 5-97: Mohr-Coulomb failure envelope for SK-17 obtained from saturated single-stage CD triaxial tests with shearing rate of 0.0009 mm/min.

The ϕ' value as obtained from the Mohr-Coulomb failure envelope for saturated specimens were adopted for the development of the extended Mohr-Coulomb failure envelope for unsaturated specimens as presented in Section 5.6.2 and the unsaturated shear strength prediction and comparison as presented in Chapter 6.

5.6.2 Extended Mohr-Coulomb Failure Envelope for Unsaturated Specimens

Series of unsaturated CD triaxial tests were conducted on SK-5, SK-10 and SK-17 specimens. In order to obtain the cohesion intercepts of the specimens at different matric suctions on different cycles of drying and wetting paths, the extended Mohr-Coulomb failure envelope was constructed by a straight line with the effective friction angle, ϕ' , drawn through the point of tangency of Mohr circles. The interception of the failure envelope and shear stress axis is the cohesion intercept, c . The ϕ' values of 26.8°, 26.9° and 33.2° as presented in Section 5.6.1 were used to construct the failure envelopes for the unsaturated CD triaxial test results of SK-5, SK-10 and SK-17 specimens, respectively.

Extended Mohr-Coulomb Failure Envelope for SK-5 specimens

Figure 5-98 presents the Mohr circles, failure envelopes and cohesion intercepts for the specimens on the 1st cycle drying and wetting paths while Figure 5-99 presents those for the specimens on the 2nd cycle drying and wetting paths. All of the cohesion intercepts obtained from the analysis of the extended Mohr-Coulomb failure envelopes of the SK-5 specimens are listed in Table 5-36.

Table 5-36: Summary of the cohesion intercepts of SK-5 specimens.

1 st cycle SWCC & 50 kPa net confining pressure		2 nd cycle SWCC & 50 kPa net confining pressure	
Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)	Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)
Drying/ 100	74.1		
Drying/ 300	108.1	Drying/ 300	104.1
Wetting/ 100	65.9		
Wetting/ 300	94.1	Wetting/ 300	99.5

Figure 5-98: Mohr circles and cohesion intercepts, c , of the SK-5 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 5-99: Mohr circles and cohesion intercepts, c , of the SK-5 specimens on the 2nd cycle drying and wetting paths under a net confining pressure of 50 kPa.

Extended Mohr-Coulomb Failure Envelope for SK-10 specimens

Figure 5-100 presents the Mohr circles, failure envelopes and cohesion intercepts of the specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa while Figure 5-101 shows those of the specimens on the 2nd cycle drying and wetting paths under a net confining pressure of 50 kPa. Figure 5-102 shows the Mohr circles, failure envelopes and cohesion intercepts of the specimens on the 1st cycle drying and wetting paths under a net confining pressure of 100 kPa. The cohesion intercepts obtained from drawing the extended Mohr-Coulomb failure envelopes of the specimens are summarized in Table 5-37.

Table 5-37: Summary of the cohesion intercepts of SK-10 specimens.

1 st cycle SWCC & 100 kPa net confining pressure		1 st cycle SWCC & 50 kPa net confining pressure		2 nd cycle SWCC & 50 kPa net confining pressure	
Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)	Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)	Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)
Drying/ 50	34.6	Drying/ 100	58.3	Drying/ 100	57.3
Drying/ 200	96.0	Drying/ 300	128.8	Drying/ 300	123.1
Wetting/ 50	33.2	Wetting/ 100	55.7	Wetting/ 100	56.5
Wetting/ 200	86.0	Wetting/ 300	114.0	Wetting/ 300	119.2

Figure 5-100: Mohr circles and cohesion intercepts, c , of the SK-10 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 5-101: Mohr circles and cohesion intercepts, c , of the SK-10 specimens on the 2nd cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 5-102: Mohr circles and cohesion intercepts, c , of the SK-10 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 100 kPa.

Extended Mohr Coulomb Failure Envelope for SK-17 specimens

The Mohr circles, failure envelopes and cohesion intercepts of the specimens on the drying and wetting paths of the 1st, 2nd and 3rd cycles are presented in Figure 5-103, Figure 5-104 and Figure 5-105, accordingly. All of the cohesion intercepts obtained from drawing the extended Mohr Coulomb failure envelopes of the specimens are summarized in Table 5-38.

Table 5-38: Summary of the cohesion intercepts of SK-17 specimens.

1 st cycle SWCC & 50 kPa net confining pressure		2 nd cycle SWCC & 50 kPa net confining pressure		3 rd cycle SWCC & 50 kPa net confining pressure	
Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)	Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)	Stress state/ ($u_a - u_w$) (kPa)	Cohesion intercept (kPa)
Drying/ 100	59.6	Drying/ 100	58.8		
Drying/ 300	98.0	Drying/ 300	93.3	Drying/ 300	92.6
Wetting/ 100	54.5	Wetting/ 100	56.9		
Wetting/ 300	72.5	Wetting/ 300	88.7	Wetting/ 300	88.0

Figure 5-103: Mohr circles and cohesion intercepts, c , of the SK-17 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 5-104: Mohr circles and cohesion intercepts, c , of the SK-17 specimens on the 2nd cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 5-105: Mohr circles and cohesion intercepts, c , of the SK-17 specimens on the 3rd cycle drying and wetting paths under a net confining pressure of 50 kPa.

5.7 PERMEABILITY TEST RESULTS

A series of saturated and unsaturated permeability tests were conducted at different matric suctions on the drying and wetting paths of different cycles in order to determine the permeability characteristics of a soil under single and multi-cycled drying and wetting processes. The saturated permeability test results are presented in Section 5.3.1 while the unsaturated permeability test results are presented in Section 5.7.1.

5.7.1 Unsaturated Water Permeability of SK-5, SK-10 and SK-17

Table 5-39 summarizes the measured water coefficient of permeability of SK-5 specimens at different matric suctions on 1st and 2nd cycle drying and wetting paths using the modified triaxial apparatus as described in Section 4.4.3. Table 5-40 summarizes the measured water coefficient of permeability of SK-10 specimens at different matric suctions on the drying and wetting paths of the 1st and 2nd cycles. In addition, the water coefficient of permeability of SK-17 specimens at different matric suctions on the drying and wetting paths of the 1st, 2nd and 3rd cycles are listed in Table 5-41.

Table 5-39: Summary of the measured water coefficient of permeability of SK-5 specimens on the drying and wetting paths of the 1st and 2nd cycles.

Multi-cycled drying and wetting	SK-5		
	Matric suction (kPa)	Drying	Wetting
		Water coefficient of permeability, k_w (m/s)	
1 st cycle	100	1.763×10^{-9}	5.993×10^{-10}
	300	3.216×10^{-10}	1.717×10^{-10}
2 nd cycle	100	1.028×10^{-9}	N/A
	300	2.820×10^{-10}	1.801×10^{-10}

Table 5-40: Summary of the measured water coefficient of permeability of SK-10 specimens on the drying and wetting paths of the 1st and 2nd cycles.

Multi-cycled drying and wetting	SK-10		
	Matric suction (kPa)	Drying	Wetting
		Water coefficient of permeability, k_w (m/s)	
1 st cycle	100	2.025×10^{-9}	4.993×10^{-10}
	300	2.683×10^{-10}	1.289×10^{-10}
2 nd cycle	100	7.847×10^{-10}	3.905×10^{-10}
	300	2.140×10^{-10}	1.301×10^{-10}

Table 5-41: Summary of the measured water coefficient of permeability of SK-17 specimens on the drying and wetting paths of the 1st, 2nd and 3rd cycles.

Multi-cycled drying and wetting	SK-17		
	Matric suction (kPa)	Drying	Wetting
		Water coefficient of permeability, k_w (m/s)	
1 st cycle	100	1.671×10^{-9}	4.182×10^{-10}
	300	2.397×10^{-10}	8.570×10^{-11}
2 nd cycle	100	8.407×10^{-10}	3.754×10^{-10}
	300	1.800×10^{-10}	8.913×10^{-11}
3 rd cycle	100	7.898×10^{-10}	N/A
	300	1.514×10^{-10}	9.089×10^{-11}

CHAPTER 6 DISCUSSION OF THE RESULTS

6.1 INTRODUCTION

This chapter presents the discussions on the results obtained from this study. Firstly, the soil selection and the experimental results of various multi-cycled SWCCs are discussed. Subsequently, the saturated and unsaturated triaxial tests results are discussed. The mechanical behaviours of unsaturated soils on the different cycles of drying and wetting are compared and discussed. Lastly, the proposed shear strength equations are then verified and compared with various existing shear strength equations using experimental data obtained from this study as well as published data from literatures. The proposed shear strength equation is shown to be able to give the best prediction on shear strength of unsaturated soil under drying and wetting conditions with the least error.

6.2 SAND-KAOLIN MIXTURE SELECTION

As presented in section 5.2, in order to use the appropriate SWCC estimation methods to determine the suitable soil mixtures for the entire research programme as described in Section 4.3.1, eight SWCC estimation methods were evaluated using ten sets of published soil data. As a result, Fredlund and Wilson Estimation method (1997, 2002) and Scheinost Estimation method (1996) were found to be able to provide most of the selected soil SWCC with reasonable good comparison between the predicted and measured SWCCs. Therefore, both methods were used in the soil selection. Among twenty sets of different sand-kaolin mixtures, SK-5, SK-10 and SK-17 were selected to be used in the experimental programme to achieve the objective of this research.

6.2.1 SWCC Estimation Methods

As mentioned in section 5.2, Fredlund and Wilson estimation method (1997, 2002) and Scheinost estimation method (1996) were selected to determine the suitable soil mixtures that were used in this research. The predicted SWCCs, with relatively good agreement as

compared to others, over the entire range of matric suction with zero water content at matric suction of 1 GPa were obtained using Fredlund and Wilson estimation method (1997, 2002). Therefore, Fredlund and Wilson estimation method (1997, 2002) may be more suitable to be used to provide the predicted SWCC over the entire range of matric suction, or the predicted SWCC that covers high suction range, e.g. 100000 kPa. On the other hand, the predicted SWCCs with the trend of non-zero water content at matric suction of 1 GPa, with relatively close agreement to the measured data as compared to others, were given by Scheinost estimation method (1996). In addition, Scheinost method (1996) was found to be able to provide reasonably good prediction on the trend of low matric suction SWCC. As a result, Scheinost method (1996) may be more applicable to be used to obtain the predicted SWCC that covers low matric suction range, or with low AEV and low residual matric suction.

Figure 6-1: The comparison between the measured SWCC and the predicted SWCC using Fredlund and Wilson PTF and Scheinost PTF for SK-5.

However, both methods were found not able to predict all selected SWCCs effectively. Some of the SWCCs were not successfully predicted using both methods. This might be due to the fact that both methods are only suitable for certain soil types. Besides, there are other uncertainty factors e.g. SWCC apparatus used, compaction of specimen, saturation

method of specimen, initial moulding water content, stress state of specimen, historical stresses of specimen, specimen volume change during testing etc, which might affect the measured SWCC to be different from the predicted SWCC.

The measured and predicted SWCCs for the SK-5 are illustrated in Figure 6-1. The measured and predicted SWCCs for SK-10 and SK-17 are shown in Figure C6.2.1-1 and Figure C6.2.1-2, respectively, APPENDIX C6.2. As shown, both estimation methods are only able to provide moderately agreement in the SWCC predictions for the mixtures. Both SWCC estimation methods were found to be over-estimating the SWCC of SK-5 for the range of matric suction higher than AEV as illustrated in Figure 6-1. The predicted AEV from Fredlund and Wilson method (1997, 2002) was found to be an over-estimation while the predicted AEV from Scheinost method (1996) was found to be an under-estimation of the AEV from the experimental results. In addition, the slopes of the measured SWCCs were found to be slightly steeper than the predicted SWCCs as illustrated in Figure 6-1. Therefore, both SWCC estimation methods might not be suitable to be used to predict the SWCCs of statically compacted sand-kaolin specimens.

6.2.2 Selected Sand-Kaolin Mixtures

As described in Sections 5.2.2 and 6.2.1, sand-kaolin mixtures of SK-5, SK-10 and SK-17 were selected and used throughout of this research. The characteristics of soil mixtures are presented in Section 5.3.1. It was noted that the characteristics of SK-10 are out of the trend of the characteristics of SK-5 and SK-17 although the sand to kaolin ratio of SK-10, i.e. 35:65, is in between of the sand to kaolin ratios of SK-5 and SK-17, i.e. 15:85 and 55:45, respectively. This could be attributed mainly to the kaolin used in SK-10 was different from that used in SK-5 and SK-17. The L2 grade coarse kaolin (as described in Section 4.3.1) used in SK-10 was taken from a different bag from that used for SK-5 and SK-17, although they were delivered in the same batch. A higher clay content of 20.5 % was found in SK-10 as compared to the clay content of 11.4 % in SK-5 although the kaolin content of SK-10 (65 %) is lower than that of SK-5 (85 %). Therefore, SK-10 has higher plasticity index, lower effective internal friction angle and higher AEV than the expected values based on the kaolin contents in SK-5, SK-10 and SK-17. Therefore, SK-5, SK-10 and SK-17 were treated as three completely different soils since the characteristics

of three mixtures are distinctly different from each other. It should also be noted that no assumption was made based on the selected sand to kaolin ratio, in the analysis and comparison throughout of this research.

Static compaction was used to prepare all sand-kaolin specimens in this research. Identical specimens with the same soil composition and uniform dry density and water content were prepared for different tests in order to have meaningful results for analyses and comparisons. Figure 6-2 illustrates the consistency of soil water content at different layers of the specimen as prepared by the static compaction method. The specimens for CD triaxial tests and SWCC tests under a net confining pressure were saturated by applying a back-pressure. The main purposes of back-pressure saturation at the beginning of the test are to have the same and uniform initial condition and to reduce the matric suction induced during compaction to a low value for all specimens. Therefore, the test results for these specimens can be analyzed and compared to produce representative findings. Specimens were consolidated to the net confining pressures of 50 kPa or 100 kPa for testing. These pressures are similar to the overburden pressure of those soils at a few meters below the ground surface, where most of the residual soil slope failures often occurred.

Figure 6-2: Illustration of the water content of soil at different layers of the specimens.

6.3 SOIL-WATER CHARACTERISTIC CURVES

6.3.1 Hysteresis of Drying and Wetting SWCCs

Three different sand-kaolin mixtures, SK-5, Sk-10 and SK-17, and three different sands, Sand IV, Sand V and Sand VI, were used to obtain multi-cycled SWCC. All the multi-cycled SWCCs are presented in Figure 6-3, Figure 6-5, Figure 6-5, Figure 6-7 and Figure C6.3.1-1 to Figure C6.3.1-6 of APPENDIX C6.3. The details of the SWCCs are given in Figure 5-22 to Figure 5-55 of Section 5.4 and Figure C5.4.1-1 to Figure C5.4.2-16 of APPENDIX C5.4. All the multi-cycled SWCCs were fitted using Fredlund and Xing (1994) equation with the correction factor, $C(\psi)$, was taken as 1.

The SWCCs obtained in this research generally have distinct drying and wetting curves and exhibit hysteresis as illustrated in Figure 6-3, Figure 6-5, Figure 6-7 and Figure 6-11. The differences between the drying and wetting SWCCs were as a result of hysteresis effects on soil, which can be explained with the “ink bottle effect” (see Section 2.4).

Apart from the “ink bottle effect”, the other main reasons for the distinct difference between the 1st cycle drying and wetting SWCCs may be explained with the mechanism of swelling and shrinking of soil as well as the presence of entrapped air in soil. During the 1st cycle drying process, the soil experienced desorption of water and the volume of soil decreased (i.e. shrinking), which caused the changes in soil fabric and structure and pore-sizes of the soil. However, during the 1st cycle wetting process, although the soil experienced adsorption of water and the volume of soil increased (i.e. swelling), some part of the soil volume was not recovered. The inelastic volume change characteristic of an unsaturated soil, which depends on the drying and wetting history of the soil (Hillel and Mottas, 1966), was observed in all SWCCs obtained in this research (see Figure 6-4, Figure 6-6 and Figure 6-8). As a result, the available pore volume for water of the soil on the 1st cycle wetting path is lesser than that of the soil on the 1st cycle drying path. Therefore, the volumetric water content on the 1st cycle wetting SWCC is significantly lower than that on the 1st drying SWCC.

Figure 6-3: Multi-cycled SWCCs of SK-5 specimen under zero net confining pressure.

Figure 6-4: The void ratio of the SK-5 specimens on the drying and wetting paths of the first, second and third cycles SWCCs of under zero net confining pressure.

Figure 6-5: Multi-cycled SWCCs of SK-17 specimen under zero net confining pressure.

Figure 6-6: The void ratio of the SK-17 specimens on the drying and wetting paths of the first, second and third cycles SWCCs of under zero net confining pressure.

Figure 6-7: The multi-cycled SWCCs of SK-17 specimens under a net confining pressure of 50 kPa.

Figure 6-8: The void ratio of the SK-17 specimens on the drying and wetting paths of the first, second and third cycles SWCCs of under a net confining pressure of 50 kPa.

The volume recovery of the soils after the 1st cycle drying and wetting processes was found to be higher in the soil without net confining pressure, as compared to that under a net confining pressure (see Figure 6-6 and Figure 6-8). This might result in more significant difference between the 1st cycle drying SWCC and 1st cycle wetting SWCC of the soils under a net confining pressure as compared to that of the soils without net confining pressure. Due to the applied net confining pressure, the free-swelling of the soil was restricted. As a result, there was less “available pore-volume for water” in the soil on the 1st cycle wetting path under a net confining pressure as compared to that without net confining pressure.

The differences between the drying and wetting SWCCs of the 1st cycle were found to be more significant as compared to the differences between the drying and wetting SWCCs of the subsequent cycles, i.e. 2nd cycle, 3rd cycle (see Figure 6-3, Figure 6-5 and Figure 6-7, Figure 6-9). This could be attributed to the less difference between the volumes of the soils on the drying and wetting paths of the subsequent cycles as compared to that of the soils on the drying and wetting paths of the 1st cycle (see Figure 6-4, Figure 6-6 and Figure 6-8). The volumes of the soils during the 2nd cycle wetting process were found to recover relatively closer to the volumes of the soils during the 2nd cycle drying process as compared to those of the specimens during the 1st cycle drying and wetting processes (see points A1, B1, C1 and D1 in Figure 6-9). Similar findings were observed in the 3rd cycle SWCC of the soils. These findings are schematically illustrated in the Figure 6-10. As a result, the “available pore-volume for water” in the soil on the drying and wetting paths of the subsequent cycles was less difference, thus less difference between the drying and wetting SWCCs of the subsequent cycles were observed. In addition the characteristics of SWCCs of the subsequent cycles (i.e. 2nd, 3rd) were generally found to be relatively similar.

The differences between the drying and wetting SWCCs of the subsequent cycles under net confining pressure were found to be smaller than those of the specimens without net confining pressure (see Figure 6-5 and Figure 6-7). This might be due to the smaller difference in soil volume on the drying and wetting paths of the subsequent cycles as compared to the latter as a result of the effect of net confining pressure on swelling of soil during wetting process.

The distinct difference between the drying and wetting SWCCs of the 1st cycle could also be attributed to the presence of new entrapped air in soil during the wetting process, which decreased the water content of the soil on the wetting path. The volume of the entrapped air in a soil might be similar on the wetting paths of all cycles since the volume of the soil on the wetting paths of all cycles are similar. Therefore, the “entrapped-air” effect that caused the difference between the drying and wetting SWCCs might be more pronounced on the 1st cycle as compared to the subsequent cycles.

As described earlier, the drying SWCCs of the 2nd and 3rd cycles as well as the wetting SWCCs of the 1st, 2nd and 3rd cycles were found to be relatively similar in terms of general characteristics of water desorption and adsorption. However, it was found that there are small differences in term of volumetric water content on the curves during the same process (drying or wetting) of the different cycles (see Figure 6-9). The small differences in volumetric water content could be attributed to the inelastic volume change characteristics of the soils during cycles of drying and wetting processes (see Figure 6-10) and the presence of more new entrapped air in the soils. The soil experienced cycles of drying and wetting processes, the volume of soil shrank and swelled accordingly. The soil volume was not completely recovered during wetting processes. As illustrated in Figure 6-10, the void ratios at D1 and D2 on the 2nd cycle wetting path are slightly lower than those at B1 and B2 on the 1st cycle wetting path due to the inelasticity of soil during drying and wetting processes. Thus, the soil volume became slightly smaller as compared to that of the previous cycle. Therefore, the soil with smaller volume and smaller pore sizes resulted in more capability to retain water during drying process and had slightly lower volumetric water content at low matric suction range (see Figure 6-9). In addition, there might be more entrapped air appeared in the soil with smaller pore sizes on the drying and wetting paths of the subsequent cycles. Thus, the volumetric water contents of soils on the SWCCs of the subsequent cycles were generally found to be lower at low matric suction range but higher at high matric suction range as compared to that of soil on the SWCCs of the previous cycle. In other words, the SWCCs are flatter with increasing cycles of drying and wetting processes. Figure 6-9 and Figure 6-10 schematically illustrate the idealized multi-cycled SWCCs and the volume change of soil during the cycles of drying and wetting processes, respectively.

Figure 6-9: Idealized multi-cycled drying and wetting SWCCs.

Figure 6-10: Illustration of the volume change of soil during multi-cycles of drying and wetting processes.

6.3.2 SWCC of Soils with Different Grain-Size Distribution

The multi-cycled SWCCs of soils with different grain-size distribution were found to be different as shown in Figure 6-11 and Figure 6-12. The drying and wetting SWCCs of the 2nd and 3rd cycles are generally located in between the drying and wetting SWCCs of the 1st cycle. These SWCCs have the similar characteristics as the drying and wetting SWCCs of the 1st cycle of the corresponding soil specimen. Various studies, e.g. Fredlund et al. (1997), Yang et al. (2004), Fredlund (2006), reported that the 1st cycle drying and wetting SWCCs of a soil are closely related to the grain size distribution of the soil. Therefore, it appears that the multi-cycled SWCCs of a soil are also governed by the corresponding grain-size distribution. The multi-cycled SWCCs of the sands have a steeper slope of SWCC and a smaller range of transition zone than those of the sand-kaolin mixtures (see Figure 6-11 and Figure 6-12) as a result of the more uniform and smaller range of grain-size distributions of the sands as compared to those of the sand-kaolin mixtures (see Figure 6-13).

It was found that the gentle slope of grain-size distribution curve of a sand leads to the gentle slope of multi-cycled SWCCs of the sand, and vice versa. Similar findings were reported in Yang et al. (2004) and Gallage and Uchimura (2010). Among the three sands, Sand VI, which has the most uniform (steepest) grain size distribution curve, has the steepest slope of multi-cycled SWCCs (see Figure 6-11 and Figure 6-13). This could be explained with the uniformity of the pore-size distribution of the sand. The more uniform the grain-size distribution of a sand, the more uniform the pore-size distribution, resulting in the steeper the slope of the SWCC of the sand. In addition, Sand IV was found to have the highest AEV, residual matric suction, wetting saturated point and water-entry value among the three sands used (see Table 6-1 and Figure 6-14). This could be attributed to the higher content of finer sand, i.e. sand particle with a diameter smaller than 0.4 mm, of Sand IV as compared to those of Sand V and Sand VI (see Figure 6-13).

It was observed the characteristics of multi-cycled SWCCs of a sand-kaolin mixture are closely related to the grain-size distribution on the range of silt and clay of the sand-kaolin mixture (see Figure 6-12 and Figure 6-13). Among three sand-kaolin mixtures, SK-10 has the highest AEV and gentlest slopes of SWCCs. Besides, SK-10 also has the highest volumetric water content at matric suction range higher than AEV. These might

be due to SK-10 has the highest content of soil particles that are smaller than 0.01 mm (see Figure 6-13). In addition, SK-10 has the highest clay content (particle size smaller than 0.002 mm). Smaller soil particles, especially clay particles, in a soil govern the SWCCs of the soil. Similar findings were reported in Hillel (1998) and Yang et al.

Table 6-1: Summary of the AEV, residual matric suction, wetting saturated point and water-entry value of the soils used in this research.

Soil type	Drying SWCC		Wetting SWCC	
	$(u_a - u_w)_b$ (kPa)	$(u_a - u_w)_r$ (kPa)	$(u_a - u_w)_{bw}$ (kPa)	$(u_a - u_w)_w$ (kPa)
SK-5	30.00	1000.00	11.00	550.00
SK-10	41.00	1200.00	13.00	730.00
SK-17	26.00	1000.00	6.20	340.00
Sand IV	2.55	6.00	0.87	3.00
Sand V	2.35	4.60	0.83	2.50
Sand VI	2.05	3.70	0.70	2.00

Figure 6-11: Multi-cycled SWCCs of different sands under zero net confining pressure.

Figure 6-12: Multi-cycled SWCCs of SK-5, SK-10 and SK-17 under zero net confining pressure.

Figure 6-13: Grain-size distributions of all soils used in this research.

As described in Section 6.2.2, different mass percentages of the same kaolin and the sand were used to prepare SK-5 and SK-17 specimens for SWCC tests. It was found that SK-5 specimens have a higher AEV, a higher wetting saturated point and higher volumetric water content in all cycles of SWCCs than those of SK-17 specimens, due to the higher fines content, or the higher clay content, in the soil specimens. In addition, the characteristics of the differences between the drying and wetting SWCCs of the multi-cycles of SK-5 are similar to those between the drying and wetting SWCCs of the multi-cycles of SK-17. This might be attributed to the similar characteristics of grain-size distribution on the range of silt and clay of mixtures, which resulted in similar pore-size distribution that govern the drying and wetting processes of the mixtures.

6.3.3 SWCC of Soils with Different Initial Dry Densities

Soils with different initial dry densities were used to obtain multi-cycled SWCCs in this research. The SWCCs of the specimens with different initial dry densities were found to be different as illustrated in Figure 6-14, Figure 6-15 and Figure 6-16. It was found that the specimen with a higher initial dry density (or a lower initial void ratio) generally have a slightly higher AEV and a slightly higher wetting saturated point than those with a lower initial dry density (or higher initial void ratio) (see Figure 6-14 and Figure 6-15). At a given matric suction higher than AEV or wetting saturated point, the specimen with a higher initial dry density has a higher volumetric water content than that with a lower initial dry density (see Figure 6-14 and Figure 6-15). These results were in agreement with the results reported from Kawai et al. (2000), Yang et al. (2004) and Sun et al. (2006) studies, which used the similar type of soils as the soils used in this research.

The soils with a higher initial dry density most likely have a lower void ratio, a lower porosity and smaller pore sizes (macropores) than those with a lower initial density. With smaller pore sizes, the soils have more capability to retain water during the drying process, i.e. desorption of water, and has more capability to fill up the pores with water during the wetting process, i.e. absorption of water, as compared to the soil with larger pore sizes. This can be explained with the capillary phenomenon as illustrated in Figure 2-3.

Figure 6-14: First cycle drying and wetting SWCCs of the Sand V specimens with initial void ratio of 0.588 and 0.509 under zero net confining pressure.

Figure 6-15: First cycle drying and wetting SWCCs of the SK-17 specimens at initial dry density of 1.77 Mg/m^3 and 1.86 Mg/m^3 under zero net confining pressure.

Figure 6-16: First, second and third cycles drying and wetting SWCCs of the SK-17 specimens at initial dry density of 1.77 Mg/m^3 and 1.86 Mg/m^3 under zero net confining pressure.

The multi-cycled SWCCs of the same soils with different initial dry densities were found to be different in term of volumetric water content. This could be attributed to the different pore sizes of the specimens with different initial dry densities. However, the characteristics, in terms of water desorption and adsorption rates, of the drying and wetting SWCCs of the same soils with different initial dry densities were found to be similar. This might be due to similar characteristics of the pore-size distributions of both soils. All SWCCs of different cycles are governed by the pore-size distribution and pore sizes of the soil. Sun et al. (2006) suggests that SWCC of a soil shifts to the right when the soil becomes denser. However, from the results obtained from this research (see Figure 6-16), it might be suggested that the multi-cycled SWCCs of a compacted sand-kaolin specimen shift up and to the right when the specimen is prepared into a denser initial condition.

6.3.4 SWCC of Soils under Different Net Confining Pressures

The multi-cycled SWCCs under different net confining pressures of the sand-kaolin mixtures were obtained. It was found that the single-cycled and multi-cycled SWCCs under zero net confining pressure are different from those under a net confining pressure (see Figure 6-17 and Figure 6-18). The higher the applied net confining pressure, the lower the saturated volumetric water content and the higher the AEV of the soil. This may be explained by the consolidation of soil that results in the presence of a lower void ratio and a smaller average pore sizes in the soil under a net confining pressure than those in the soil under zero net confining pressure. The soil with a lower void ratio has a lower water content at low matric suctions. When the matric suction increases, the soil with smaller pore sizes has more capability to retain water in the pores under suction than the soil with larger pore sizes. Therefore, the soil under a higher net confining pressure has a higher AEV than that under zero net confining pressure. These findings agree with the findings from Vanapalli et al. (1996b), Ng and Pang (2000) and Thu et al. (2006) studies.

Figure 6-17: The first cycle drying and wetting SWCCs of SK-10 specimens under different net confining pressures.

Figure 6-18: Multi-cycled SWCCs of SK-17 specimens under different net confining pressures.

6.4 MECHANICAL BEHAVIOUR OF SAND-KAOLIN MIXTURES

Series of CD triaxial tests were conducted in order to study the characteristics of SK-5, SK-10 and SK-17 on multi-cycled drying and wetting paths. All results are presented in Section 5.3 and Section 5.5 as well as attached in APPENDIX C5.3 and APPENDIX C5.5. The discussions on the results are presented in this section.

6.4.1 Characteristics of Saturated Sand-Kaolin Mixtures

As presented in Section 5.3.1, Figure 5-16 to Figure 5-18 show the stress-strain characteristics of SK-10 specimens while Figure 5-19 and Figure 5-20 show the stress-strain characteristics of SK-5 and SK-17 specimens, respectively. In general, it can be observed that the higher the net confining pressure, the higher the peak deviator stress and the higher the stiffness of the specimen. This was due to the increase in net confining pressure that caused the specimen to become denser resulting in higher peak deviator stress and higher stiffness than those under a lower net confining pressure.

Similar stress-strain behaviours of the SK-10 specimens are shown in Figure 5-16 to Figure 5-18 although they were obtained from different test conditions. The shear strength parameters of SK-10, i.e. c' and ϕ' , that were obtained from the different tests, were found to be similar also. Therefore, all of the saturated triaxial test results of SK-10 were plotted in the same Mohr-Coulomb failure envelope for analysis in order to obtain the c' and ϕ' for SK-10 (Figure 5-96 of Section 5.6.1). The c' and ϕ' of SK-10 were used in the unsaturated shear strength analyses.

The results of single-stage CD triaxial test series of SK-5 and SK17, as presented in Section 5.3.1, were plotted in Mohr circles with failure envelopes as given in Figure 5-95 and Figure 5-97, respectively. The c' and ϕ' for both SK-5 and SK-17 were obtained from both Mohr-Coulomb failure envelope analyses. The tangent lines were managed to be drawn through all the points of tangency of the Mohr circles of SK-5 and SK-10 successfully. Thus, the c' and ϕ' of SK-5 and SK-17 were determined and used in unsaturated shear strength analyses.

6.4.2 Characteristics of Unsaturated Sand-Kaolin Mixtures

Figure 6-19 and Figure 6-20 illustrate the shearing characteristics and Mohr circles of SK-5 specimens, respectively. Generally, under the same net confining pressure, an unsaturated soil has a higher shear strength and a higher stiffness than a saturated soil. Figure 6-19 and Figure 6-20 show the significant differences between the saturated shear strength (at zero matric suction) and the unsaturated shear strengths (at matric suctions of 100 kPa and 300 kPa) of SK-5 specimens. The results show that the specimen at a given matric suction has a higher peak deviator stress and a higher stiffness than the specimen at zero matric suction. The higher the matric suction, the higher the shear strength and the higher the stiffness of a soil. In addition, as illustrated in Figure 5-58, the dilatancy of a soil under a low net confining pressure increases with an increase in the applied matric suction. The specimen at a higher matric suction has a lower void ratio, and thus, has the tendency to have a larger dilatancy than that at lower matric suction. In other words, the matric suction in a soil plays an important role in the volume change and stiffness of the soil as well as in contributing additional strength to the soil.

Figure 6-19: Stress-strain relationship of the saturated and unsaturated SK-5 specimens under a net confining pressure of 50 kPa.

Figure 6-20: Mohr circles and cohesion intercepts, c , of the saturated and unsaturated SK-5 specimens under a net confining pressure of 50 kPa.

It was noticed that the post-peak shear strengths of different specimens at different matric suctions are similar as shown in Figure 6-19. The effect of the applied matric suction on the shear strength appears to be eliminated after the post-peak strain softening, when the shear plane has fully developed in the specimen. This might be attributed to the fact that the post-peak shearing between the shear planes in the specimen is independent from the contribution of matric suction.

6.4.3 Characteristics of Sand-Kaolin Mixtures under Hysteresis Effects

This section discusses the characteristics of the sand-kaolin mixtures on the drying and wetting paths of different cycles. As designed and listed in 4.6.2, a total number of 28 unsaturated CD triaxial tests were conducted on multi-cycles of drying and wetting paths using SK-5, SK-10 and SK-17 statistically compacted specimens. The stress-strain curves and total volumetric strains during shearing of all specimens at different matric suction on different cycle drying and wetting paths are presented in Section 5.5 It should be noted that, for every designated matric suction and net confining pressure, only one test was conducted due to the limited time and available equipments since the test requires a long

time of testing and sophisticated equipments. Therefore, all the tests were conducted with good care and to the best knowledge of the author in order to produce reliable and good quality testing results. Some parts of the results and findings were published in Goh et al. (2010).

6.4.3.1 Shear Strength and Volume Change Characteristics during Shearing on the First Drying and Wetting Paths

Figure 6-21 illustrates the idealized stress-strain curves of sand-kaolin specimen as obtained from this research with the detailed behaviour of the specimens on the 1st drying and wetting paths. The stress-strain and total volume change characteristics of SK-5, SK-10 and SK-17 specimens on the 1st cycle drying and wetting paths obtained from the experiments are given in Figure 6-22 to Figure 6-31. In general, as illustrated in Figure 6-21, the specimens on the 1st cycle drying path (1st drying path) have a higher shear strength than those on the 1st cycle wetting path (1st wetting path) at a given matric suction. The difference appears to be more distinct for the specimens at high matric suctions (see Figure 6-22, Figure 6-26 etc), i.e. 200 or 300 kPa, which are in the transition stage as described in the physical model of Vanapalli et al. (1996b) (Figure 2-9), as compared to the specimens at low matric suctions, which probably are in or close the boundary effect stage of the physical model (Goh et al., 2010). The shear strengths of the specimens at low matric suctions, i.e. 50 or 100 kPa, on the 1st cycle drying path were found to be slightly higher than those on the wetting path. These findings agree with the findings reported by Han et al. (1995), Tse and Ng (2008).

However, as illustrated in Figure 6-22, it was noticed that the post-peak shear strengths of different specimens at different matric suctions (for the same type of soil) on the 1st cycle drying and wetting paths are generally similar, except for the SK-10 specimen on the 1st cycle drying path under a net confining pressure of 100 kPa that has no post-peak strain softening behaviour. The post-peak shear strengths of SK-10 specimens under different net confining pressures were found to be different as shown in Figure 6-26 and Figure 6-30. As illustrated in Figure 6-22, Figure 6-28 and Figure 6-30, the specimens (SK-17, SK-5 and SK-10), which were prepared using the different type of soils, have different post-peak shear strengths.

The specimens on the 1st cycle drying path generally have a higher peak shear strength, a lower stiffness, a higher axial strain at failure and a more ductility than those on the 1st cycle wetting path (as illustrated in Figure 6-21). A significant strain softening behaviour was observed in all SK-5 and SK-17 specimens. Besides, strain softening behaviour was found in SK-10 specimen at matric suction of 300 kPa under net confining pressure of 50 kPa (see Figure 6-30). Under a net confining pressure of 100 kPa, SK-10 specimen at matric suction of 200 kPa on the 1st cycle wetting path also exhibits distinct strain softening after the peak deviator stress was reached (see Figure 6-26).

In general, the specimens on the 1st cycle wetting path exhibit larger dilatancy during shearing as compared to those on the 1st cycle drying path. The specimens on the 1st cycle wetting path exhibit less contraction in the early stage of shearing and exhibit more dilation in the later as compared to those on the 1st cycle drying path at the same matric suction (see Figure 6-23). Furthermore, it was noticed that the specimens on the 1st cycle drying paths at higher matric suctions have a larger dilatancy than those at lower matric suctions (see Figure 6-23). However, on the 1st cycle wetting path, higher dilations were observed for the specimens at lower matric suction as compared to those at higher matric suctions (see Figure 6-23).

On the other hand, the specimens under a higher net confining pressure have more distinct difference in volume change behaviour during shearing as shown in Figure 6-27. Under a higher net confining pressure, the specimens on the 1st cycle wetting path dilate during shearing while the specimens on the 1st cycle drying path contract during shearing

As illustrated in Figure 6-24, the water content of the specimens at higher matric suctions decreases during shearing. In contrast, at the lower matric suctions, the water contents of the specimens on the 1st cycle drying path decreases in the early stage of shearing and increases in the later stage of shearing while the water contents of those on the 1st cycle wetting path increase in the whole shearing stage (see Figure 6-24).

Figure 6-25 illustrates the degree of saturation of the sand-kaolin specimens during shearing. In general, the degree of saturation of the specimens on the 1st cycle drying and wetting paths essentially remains constant or decreases gradually during shearing.

Figure 6-21: Illustration of the stress-strain curves of sand-kaolin specimen on the 1st cycle drying and wetting paths.

Figure 6-22: Stress-strain curves of the SK-17 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-23: Total volume change characteristics of the SK-17 specimens during shearing on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-24: Water volume change characteristics of the SK-17 specimens during shearing on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-25: Degree of saturation of the SK-17 specimens during shearing on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-26: Stress-strain curves of the SK-10 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 100 kPa (Goh et al., 2010).

Figure 6-27: Total volume change characteristics of the SK-10 specimens during shearing on the 1st cycle drying and wetting paths under a net confining pressure of 100 kPa (Goh et al., 2010).

Figure 6-28: Stress-strain curves of the SK-5 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-29: Total volume change characteristics of the SK-5 specimens during shearing on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-30: Stress-strain curves of the SK-10 specimens on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-31: Total volume change characteristics of the SK-10 specimens during shearing on the 1st cycle drying and wetting paths under a net confining pressure of 50 kPa.

The ductile behaviour and volume contraction (or smaller volume dilation under low net confining pressure) of the sand-kaolin specimens on the 1st cycle drying path during shearing are similar to the behaviour of a normally consolidated specimen (Melinda et al., 2004; Goh et al., 2010). The specimens on the 1st cycle drying path were initially saturated and consolidated to the designated net confining pressure. The specimens were subsequently subjected to the applied matric suctions by following the designated matric suction incremental steps. The specimens were then equalized at the final matric suctions (i.e. 50, 100, 200 or 300 kPa) before the shearing stage. The matric suction induced in the specimens during static compaction process was reduced to minimum during the saturation stage. Upon the consolidation and the drying processes, the specimen on the 1st cycle drying path could be considered similar to a normally consolidated soil due to the applied matric suction increases to a maximum magnitude to which the soil has never been subjected (Rahardjo and Fredlund, 2003). This stress path is analogous to the virgin compression curve associated with the conventional consolidation test on a soil (Rahardjo and Fredlund, 2003). As a result, the specimens on the drying path behave like a normally consolidated soil that exhibit ductility and contraction (or lower dilatancy) during shearing (Goh et al., 2010). In addition, it was observed that the shearing behaviour of a

specimen on the drying path under a higher net confining pressure is more alike to a normally consolidated soil than that under a lower net confining pressure.

Similar to those on the 1st cycle drying path, the specimens on the 1st cycle wetting path were initially saturated and subsequently consolidated to the designated net confining pressure. The specimens went through the drying path by following the designated matric suction incremental steps and subsequently went through part of the wetting path by following the designated matric suction decremental steps and subsequently equalized at the designated final matric suction (i.e. 50, 100, 200 or 300 kPa) before the shearing stage. After the matric suction decreased to the designated matric suction during the wetting process (similar to the unloading), the maximum matric suction that the soil has ever been subjected (i.e. 440 kPa) during the drying process could be considered similar to the pre-consolidation pressure of an over-consolidated soil. As a result, the sand-kaolin specimens on the wetting path behave like an over-consolidated soil which exhibit brittleness and dilation (or larger volume dilation as compared to the specimen on the drying path under the same low net confining pressure) during the shearing stage (Melinda et al., 2004; Goh et al., 2010). Furthermore, it was observed that the shearing behaviour of a specimen on the wetting path under a higher net confining pressure is more alike to an over-consolidated soil than that under a lower net confining pressure.

The degree of saturation of the specimens decreases gradually during shearing. The degree of saturation of a soil reflects directly the proportion of the wetted area of soil, which governs the shear strength contribution from matric suction. As a result, the gradual decrement of degree of saturation indicates that the shear strength of the soil decreases slowly as the matric suction contribution to shear strength decreases gradually.

The specimens on the 1st cycle drying path have a higher shear strength than those on the 1st cycle wetting path. This could be attributed to the hysteresis effects on the specimens. Similar findings were reported in various studies, e.g. Han et al. (1995), Rahardjo et al. (2004), Tse and Ng (2008), Goh et al. (2010), etc. At a given matric suction, the soil on the drying path has a higher volumetric water content than the soil on the wetting path. The difference in water content of the soils under drying and wetting processes could be explained with the mechanisms described in Section 6.3.1. The volumetric water content

of a soil is proportional to the wetted area of the soil, which contributes to the increase in shear strength of the soils when subjected to matric suction (Rahardjo et al., 2004). Therefore, the higher water content in the soil results in a larger wetted area of soils, contributing to the increase in shear strength of the soil.

The post-peak shear strengths of the specimens at different matric suctions on the 1st cycle drying and wetting paths are similar (see Figure 6-21). The effect of the applied matric suction, the highest matric suction that the soil has been subjected to during the drying process and the effect of hysteresis appear to be eliminated after the post-peak strain softening has occurred in the specimen, at the time when the shear plane has fully developed in the specimen. This might be attributed to the post-peak shearing between the shear planes in the specimen is independent from the contribution of matric suction. The post-peak shear strength of a specimen appears to be governed by the applied net confining pressure and the effective internal friction angle. The higher the applied net confining pressure on a specimen, the higher the post-peak shear strength of the specimen. In addition, the higher the effective internal friction angle, the higher the post-peak shear strength of the specimen.

On the 1st cycle wetting path, the specimens at lower matric suctions have larger dilatancy during shearing than those at higher matric suctions. The volumes of the specimens were not recovered (see Figure 6-8 and Figure 6-10) after the 1st cycle drying process due to the inelastic behaviour of soil and the effect of confinement on the soil. The volumes of the specimens at different matric suctions on the 1st cycle wetting path were similar (see Figure 6-8). Therefore, the matric suction in the soil governs the dilatancy of the specimens during shearing. Matric suction plays an important role in “holding” the structure of soil particles, and thus affects the strength and deformation of the soil (Fredlund and Rahardjo, 1993a). In this case, the specimens at the higher matric suctions on the 1st cycle wetting path have a greater resistance from dilation during shearing as compared to those at the lower matric suctions.

6.4.3.2 Shear Strength and Volume Change Behaviours during Shearing on the Second Drying and Wetting Paths

The general stress-strains and total volume change characteristics of the sand-kaolin specimens on the 2nd cycle drying and wetting paths are illustrated in Figure 6-32 and Figure 6-33. As illustrated in Figure 6-32, the difference in magnitude between the shear strengths of the specimens on the 2nd cycle drying and wetting paths is relatively smaller than that of the specimens on the 1st cycle drying and wetting paths (see Figure 6-22). The specimens on the 2nd cycle drying path generally have a slightly higher shear strength at a slightly higher axial strain at failure than those on the 2nd cycle wetting path at a given matric suction. Similar to the shear strength characteristics of the 1st cycle, the difference in magnitude between drying and wetting shear strengths at a given matric suction appears to be more significant for the specimens at higher matric suctions. The post-peak shear strength of the all specimens at different matric suctions on the 2nd cycle drying and wetting paths were found to be similar.

The specimens on the 2nd cycle drying and wetting paths generally exhibit similar stiffness, ductility and volume change behaviour during shearing (see Figure 6-32 and Figure 6-33). The specimens exhibit contraction in the early stage of shearing and exhibit dilation in the later stage of shearing. Similar to the specimens on the 1st cycle drying and wetting paths, the specimens on the 2nd cycle wetting path exhibit a slightly larger dilatancy during shearing as compared to those on the 2nd cycle drying path. Besides, specimens at lower matric suctions tend to dilate more than those at higher matric suction.

The specimens on the 2nd cycle drying path have a slightly higher shear strength than those on the 2nd cycle wetting path. The relatively small difference between the shear strengths on the 2nd cycle drying and wetting paths as compared to that on the 1st cycle drying and wetting, could be attributed to the hysteresis effects on the specimens. At a given matric suction, the soil on the 2nd cycle drying path has a slightly higher volumetric water content than the soil on the 2nd cycle wetting path. The difference in water content of a soil results in the difference in the amount of the wetted area of the soil that governs the unsaturated shear strength when subjected to matric suction. Therefore, the specimens on the 2nd cycle drying path have a slightly higher peak shear strength than those on the 2nd cycle wetting path.

Figure 6-32: Stress-strain curves of the SK-17 specimens on the 2nd cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-33: Total volume change characteristics of the SK-17 specimens during shearing on the 2nd cycle drying and wetting paths under a net confining pressure of 50 kPa.

The behaviour of the sand-kaolin specimens on the 2nd cycle during shearing is similar to the behaviour of an over-consolidated soil. Similar to the specimens on the 1st cycle wetting path, the specimens on the 2nd cycle drying and wetting paths were subjected to the highest matric suction (i.e. 440 kPa) that the specimens have ever experienced during the 1st cycle drying and wetting processes. Since the magnitude of the matric suctions applied to the specimens during the 2nd cycle did not exceed the maximum matric suction that the specimens ever experienced, the behaviour of the sand-kaolin specimens on the 2nd cycle drying and wetting paths during shearing are similar to those on the 1st cycle wetting path, which are similar to the behaviour of an over-consolidated soil. Therefore, the sand-kaolin specimens on the 2nd drying and wetting paths exhibit brittleness and dilation during the shearing stage that behaves like an over-consolidated soil.

The specimens on the 2nd cycle wetting path have a slightly larger dilatancy during shearing than those on the 2nd cycle drying path at a given matric suction. This could be contributed by the void ratio of the specimens on the 2nd cycle wetting path being slightly smaller than those on the 2nd cycle drying path (see Point C2 and D2 in Figure 6-10). Generally, the specimens on the 2nd cycle drying and wetting paths have similar characteristics of volume change during shearing to those on the 1st cycle wetting path.

The specimens have a similarly low void ratio (see Figure 6-8) at different matric suctions, i.e. 100 and 300 kPa on the 2nd cycle drying path or 2nd cycle wetting path. The specimens at lower matric suction tend to have larger dilatancy during shearing due to the smaller effect of matric suction in the specimens, and thus have lesser resistance to dilation as compared to those at higher matric suctions.

6.4.3.3 Shear Strength and Volume Change behaviours during Shearing on the Third Drying and Wetting Paths

The stress-strain and total volume change characteristics of the sand-kaolin specimens on the 3rd cycle drying path and 3rd cycle wetting path are illustrated in Figure 6-34 and Figure 6-35. As shown, the specimens on the 3rd cycle drying and wetting paths exhibit similar stiffness, ductility and volume change during shearing. The specimen on the 3rd cycle drying path has a slightly higher peak shear strength at a slightly higher axial strain

at failure than that on the 3rd cycle wetting path as shown in Figure 6-34. The difference in magnitude between the shear strengths of the specimens on the 3rd cycle drying and wetting paths are smaller than that of the specimens on the 2nd cycle drying and wetting paths at a given matric suction. The post-peak shear strengths of both specimens were found to be similar. As illustrated in Figure 6-35, both specimens exhibit contraction in the early stage of shearing and exhibit dilation in the later stage of shearing. The specimen on the 3rd cycle wetting path have a slightly larger dilatancy during shearing as compared to those on the 3rd cycle drying path.

The sand-kaolin specimens on 3rd cycle drying and wetting paths have a similar shearing behaviour as an over-consolidated soil. Analogous to the specimens on the 1st cycle wetting path, the specimens on 3rd cycle drying and wetting paths were subjected to the highest matric suction during the 1st cycle drying and wetting processes. The magnitude of the matric suctions applied to the specimens during the 2nd cycle and 3rd cycle did not exceed the maximum matric suction that the specimens ever experienced. Therefore, the sand-kaolin specimens on the 3rd cycle drying and wetting paths behave in a similar manner as an over-consolidated soil that exhibits brittleness and dilation during shearing. These characteristics are similar to those of the specimens on 1st cycle wetting path as well as 2nd cycle drying and wetting paths.

The specimen on the 3rd cycle drying path has a slightly higher shear strength than that on the 3rd cycle wetting path as a result of the hysteresis effects on the specimens. The relatively small difference between the volumetric water content of the specimens on the 3rd cycle drying and wetting paths results in the relatively small difference between the shear strengths on the 3rd cycle drying and wetting paths, as compared to those on the 1st cycle and 2nd cycle. The specimens on the 3rd cycle drying and wetting paths generally have a similar characteristic of volume change during shearing as those on the 1st cycle wetting path (see Figure 6-23 and Figure 6-35). However, the specimen on the 3rd cycle wetting path has a slightly larger dilatancy during shearing than that on the 3rd cycle drying path at a given matric suction. This could be attributed by the void ratio of the specimens on the 3rd cycle wetting path being slightly smaller than those on the 3rd cycle drying path (see Figure 6-10).

Figure 6-34: Stress-strain curves of the SK-17 specimens on the 3rd cycle drying and wetting paths under a net confining pressure of 50 kPa.

Figure 6-35: Total volume change characteristics of the SK-17 specimens during shearing on the 3rd cycle drying and wetting paths under a net confining pressure of 50 kPa.

6.4.3.4 Shear Strength and Volume Change behaviours during Shearing on the First, Second and Third Drying Paths

As illustrated in Figure 6-36 and Figure 6-37, the specimens on the 1st cycle drying path generally have a higher shear strength and a higher axial strain at failure as well as exhibit less dilation during shearing than those on the subsequent drying paths, i.e. 2nd and 3rd cycles. On the other hand, the shearing behaviours of the specimens on the 2nd and 3rd cycles drying path are generally similar. It was observed that the specimen on the 3rd cycle drying path has a slightly lower peak shear strength and a slightly smaller axial strain at failure as well as exhibit slightly larger dilatancy during shearing than that on the 2nd cycle drying path.

The specimens on the 1st cycle drying path have a higher shear strength than those on the subsequent drying paths could be due to the difference in the volumetric water contents of the specimens. The specimens on the 1st cycle drying path have a higher volumetric water content, a larger wetted area of soil, resulting in a higher shear strength than those on the subsequent cycle drying paths at the given matric suction.

Figure 6-36: Stress-strain curves of the SK-17 specimens on the multi-cycle drying paths under a net confining pressure of 50 kPa.

Figure 6-37: Total volume change characteristics of the SK-17 specimens during shearing on the multi-cycle drying paths under a net confining pressure of 50 kPa.

As described in Sections 6.4.3.1 to 6.4.3.3, the specimens on the 1st cycle drying path exhibit similar shearing behaviour as that of a normally consolidated soil while the specimens on the 2nd and 3rd cycles drying paths exhibit similar shearing behaviour as that of an over-consolidated soil. Besides, the specimens on the 2nd and 3rd cycles drying paths have a smaller void ratio prior to shearing than those on the 1st drying path (see Figure 6-8), which is likely due the reasons described in Section 6.3.1. As a result, the specimens on the 2nd and 3rd cycles drying paths have different dilation magnitude during shearing from those on the 1st cycle drying path. The difference is more significant in those specimens at lower matric suctions, which have a more distinct difference in the void ratio prior to shearing.

The difference in volumetric water contents of specimens on the 2nd and 3rd cycles drying paths is smaller as compared to the difference in those of the specimens on 1st and 2nd cycles drying paths (see Figure 6-7). In addition, the void ratios of the specimens on the 2nd and 3rd cycles drying paths are similar (see Figure 6-8). Therefore, similar shear strength and shearing behaviour of the specimens on the 2nd and 3rd cycles drying paths were observed in Figure 6-36. The small difference observed between the shear strength

and shearing behaviour might be mainly due to the small difference between volumetric water contents and void ratio of the specimens on the 2nd and 3rd cycles drying paths.

6.4.3.5 Shear Strength and Volume Change behaviours during Shearing on the First, Second and Third Wetting Paths

As shown in Figure 6-38 to Figure 6-41, the specimens on the 1st cycle wetting path were found to have a lower shear strength and a smaller axial strain at failure as well as exhibit less ductility during shearing than those on the subsequent cycle wetting paths. These results are similar to those from Tse and Ng (2008) study, which reported that the shear strength of CDT soil on the 1st cycle wetting path is lower than that on the 2nd cycle wetting path. The lower shear strength of the specimen on the 1st cycle wetting path appear to be related to the lower volumetric water content (i.e. smaller wetted area of soil) of the specimens on the 1st cycle wetting path as compared to those of the specimens on the subsequent wetting paths (see Figure 6-9 and Figure 6-10). The specimens on the 1st cycle wetting path have a less ductility and a smaller axial strain at failure than those on the subsequent cycle wetting paths might be also due to the smaller wetted area of soil. As a result, the particles in the soil have a weaker “holding” ability due to the smaller effect of matric suction in the soil.

The specimens on the wetting paths of more cycles have a greater effect of matric suction in soil that resist the soil from dilation than those on the wetting paths of fewer cycles. However, the void ratios of the specimens on the wetting paths of more cycles are also smaller (see Figure 6-10). As a result, the specimen has a greater ability to dilate during shearing due to the smaller void ratio of the soil. In this case, the tendency of dilation due to the small void ratio of a soil is likely to be counter-balanced by the effect of matric suction in the soil. Therefore, all specimens on different cycles wetting paths have similar dilation magnitude during shearing.

Figure 6-38: Stress-strain curves of the SK-17 specimens on the multi-cycle wetting paths under a net confining pressure of 50 kPa.

Figure 6-39: Total volume change characteristics of the SK-17 specimens during shearing on the multi-cycle wetting paths under a net confining pressure of 50 kPa.

Figure 6-40: Stress-strain curves of the SK-5 specimens on the multi-cycle wetting paths under a net confining pressure of 50 kPa.

Figure 6-41: Total volume change characteristics of the SK-5 specimens during shearing on the multi-cycle wetting paths under a net confining pressure of 50 kPa.

6.5 Unsaturated Permeability of Sand-Kaolin under Hysteresis Effect

Series of unsaturated permeability tests were conducted in order to study the characteristics of the sand-kaolin mixtures on multi-cycled drying and wetting paths. All results are presented in Section 5.7. Figure 6-42 and Figure 6-43 illustrate the water coefficient of permeability of SK-5 and SK-17 specimens on the drying and wetting paths of different cycles as well as the fitting curves using Leong and Rahardjo (1997a) permeability function.

It was found that the water coefficients of permeability of the sand-kaolin mixtures on the drying and wetting paths are different. The water coefficients of permeability of the specimens on the drying paths are higher than those of the specimens on the wetting paths. The differences between the coefficients of permeability on the 1st cycle drying and wetting paths were found to be larger than the differences between the coefficients of permeability on the subsequent cycle drying and wetting paths (see Figure 6-42 and Figure 6-43). The characteristics of permeability under drying and wetting were found to be analogous to the characteristic of the drying and wetting SWCCs. This is due to the fact that water in soil can only flow through soil voids that are filled with water in a continuous path (Fredlund and Rahardjo, 1993a). As a result, the permeability of the sand-kaolin specimens exhibited differently during drying and wetting due to hysteresis. Similar findings were reported in Fredlund and Rahardjo (1993a), Gan and Fredlund (2000), Agus et al. (2003) etc.

The measured water coefficients of permeability obtained from this study were fitted using Leong and Rahardjo (1996a) permeability function. The fitting parameter, q , for fitting the water coefficient of permeability of SK-5 on the 1st cycle drying and wetting paths are 4.230 and 4.483, respectively. On the other hand, the fitting parameter, q , for fitting the water coefficient of permeability of SK-17 on the 1st cycle drying and wetting paths are 6.920 and 6.923, respectively. These values of fitting parameter are within the range that was reported in Fredlund et al. (2001).

Figure 6-42: Measured water coefficient of permeability of the SK-5 specimens and fitting curves using Leong and Rahardjo (1997b) permeability function.

Figure 6-43: Measured water coefficient of permeability of the SK-17 specimens and fitting curves using Leong and Rahardjo (1997b) permeability function.

6.6 COMPARISONS BETWEEN DEVELOPED EQUATIONS AND OTHER PUBLISHED EQUATIONS

This section presents and discusses the comparisons between the descriptive and predictive capabilities of the proposed shear strength prediction equations (as elaborated in Section 3.7, Goh et al. (2010) equations) and the existing shear strength equations (as listed in Table 3-3. The experimental drying and wetting shear strength results were used for the comparisons. Meanwhile, two sets of the published shear strength data as listed in Table 3-2 were also used for the comparative analyses. It should be noted that both sets of published data were not used in the development of the shear strength prediction equations as proposed in Section 3.7.

The same statistical analyses as elaborated in Section 3.6 were performed to assess the comparisons of the shear strength equations. As defined in Section 3.6.2, the selected published shear strength equations were classified according to the nature of equation, i.e. prediction type shear strength equation and fitting type shear strength equation. The proposed equations were compared with various prediction type equations directly while the fitting type shear strength equations were used as the reference in the comparisons. The comparisons are presented by plotting the predicted shear strength envelopes, the predicted shear strengths against the measured shear strength as well as by listing the results of the statistical analyses on the predictions from the proposed equations and the published equations. Some parts of this section were published in Goh et al. (2010).

6.6.1 Comparisons using Experimental Results from This Study

The experimental drying and wetting shear strengths results of the statically compacted SK-5, SK-10 and SK-17 specimens were used in the comparisons. These experimental results were not used in the development of the proposed shear strength prediction equations. Due to the limited shear strength data points on each drying or wetting paths, the published fitting type equations were not included in the comparisons since unsaturated shear strength results are needed for developing the unsaturated shear strength envelope. The published prediction type equations, which were originally introduced for drying shear strength prediction, were assumed to be able to predict the

wetting shear strength using the available soil parameters in this research. Figure 6-44 to Figure 6-47, Figure 6-48 to Figure 6-51 as well as Figure 6-52 to Figure 6-55 present the comparisons while Table 6-2, Table 6-3 and Table 6-4 summarize the results of the statistical analyses on the comparisons using the measured drying and wetting shear strengths of the SK-5, SK-10 and SK-17 specimens, respectively.

Among all the equations, the predicted drying and wetting shear strengths from the proposed equations are generally the closest to the measured drying and wetting shear strengths as shown in the given respective figures (see Figure 6-44 to Figure 6-55). The proposed equations were found to be able to predict all measured drying and wetting shear strengths successfully, with giving the best agreement with the least errors for both drying and wetting shear strengths of all sand-kaolin mixtures. The predicted drying and wetting shear strengths envelopes were shown to be in a continuous form of nonlinear curves, demonstrating the nonlinear shear strength behaviour of an unsaturated soil. The predicted shear strength envelope shows a smooth transition from the matric suction range below AEV to that above AEV.

Figure 6-44: Comparison between the proposed equations (Goh et al. (2010) equation) and the selected published prediction type equations using the experimental drying shear strength results of SK-5 specimens from this study.

Figure 6-45: Comparison between the proposed equations (Goh et al. (2010) equation) and the selected published prediction type equations using the experimental wetting shear strength results of SK-5 specimens from this study.

Table 6-2: The comparisons on the predicted drying shear strengths using the proposed equation and various published prediction type equations for the SK-5 specimens.

Authors	Nature of equation	Shear strength prediction			
		Drying		Wetting	
		SSE_{norm}		SSE_{norm}	
Vanapalli et al. (1996b, Eq.2)	Prediction	0.18995	[D]	0.35000	[D]
Oberg and Sallfors (1997)	Prediction	0.00709 (2)	[A]	0.02162 (2)	[A]
Bao et al. (1998)	Prediction	0.08420 (4)	[A]	0.23862	[D]
Khalili and Khabbaz (1998)	Prediction	0.17289	[D]	0.28615	[D]
Tekinsoy et al. (2004)	Prediction	0.01568 (3)	[A]	0.02208 (3)	[A]
Garven and Vanapalli (2006)	Prediction	0.26485	[D]	0.17119 (4)	[D]
Goh et al. (2010)	Prediction	0.00135 (1)	[A]	0.01070 (1)	[A]

Note: SSE_{norm} value is between zero to infinity, which a smaller value representing a better predicted results. Value in parentheses at the SSE_{norm} column is the best-prediction ranking for the selected published equations on every set of data. Value of 1 indicates the best prediction for the data set.

The [A] and [D] denote “agreement” and “discrepancy”, respectively, which are defined in text.

Figure 6-46: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts on the first drying path of SK-5 specimens obtained from this research.

Figure 6-47: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts on the first wetting path of SK-5 specimens obtained from this research.

Figure 6-48: Comparison between the proposed equations (Goh et al. (2010) equation) and the selected published prediction type equations using the experimental drying shear strength results of SK-10 specimens from this study.

Figure 6-49: Comparison between the proposed equations (Goh et al. (2010) equation) and the selected published prediction type equations using the experimental wetting shear strength results of SK-10 specimens from this study.

Table 6-3: The comparisons on the predicted drying shear strengths using the proposed equation and various published prediction type equations for the SK-10 specimens.

Authors	Nature of equation	Shear strength prediction			
		Drying		Wetting	
		SSE_{norm}		SSE_{norm}	
Vanapalli et al. (1996b, Eq.2)	Prediction	0.48715	[D]	1.24745	[D]
Oberg and Sallfors (1997)	Prediction	0.03946 (2)	[A]	0.04816 (2)	[A]
Bao et al. (1998)	Prediction	0.37363	[D]	0.88292	[D]
Khalili and Khabbaz (1998)	Prediction	0.62221	[D]	1.32749	[D]
Tekinsoy et al. (2004)	Prediction	0.04147 (3)	[A]	0.09740 (3)	[A]
Garven and Vanapalli (2006)	Prediction	0.25603 (4)	[D]	0.30080 (4)	[D]
Goh et al. (2010)	Prediction	0.00839 (1)	[A]	0.01437 (1)	[A]

Note: SSE_{norm} value is between zero to infinity, which a smaller value representing a better predicted results. Value in parentheses at the SSE_{norm} column is the best-prediction ranking for the selected published equations on every set of data. Value of 1 indicates the best prediction for the data set.

The [A] and [D] denote “agreement” and “discrepancy”, respectively, which are defined in text.

Figure 6-50: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts on the first drying path of SK-10 specimens obtained from this research.

Figure 6-51: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts on the first wetting path of SK-10 specimens obtained from this research.

Figure 6-52: Comparison between the proposed equations (Goh et al. (2010) equation) and the selected published prediction type equations using the experimental drying shear strength results of SK-17 specimens from this study.

Figure 6-53: Comparison between the proposed equations (Goh et al. (2010) equation) and the selected published prediction type equations using the experimental wetting shear strength results of SK-17 specimens from this study.

Table 6-4: The comparisons on the predicted drying shear strengths using the proposed equation and various published prediction type equations for the SK-17 specimens.

Authors	Nature of equation	Shear strength prediction			
		Drying		Wetting	
		SSE_{norm}		SSE_{norm}	
Vanapalli et al. (1996b, Eq.2)	Prediction	0.22804	[D]	1.08087	[D]
Oberg and Sallfors (1997)	Prediction	0.08799 (3)	[A]	0.40780 (4)	[D]
Bao et al. (1998)	Prediction	0.09408 (4)	[A]	0.87652	[D]
Khalili and Khabbaz (1998)	Prediction	0.27056	[D]	0.66801	[D]
Tekinsoy et al. (2004)	Prediction	0.07396 (2)	[A]	0.20292 (3)	[D]
Garven and Vanapalli (2006)	Prediction	0.19354	[D]	0.17439 (2)	[D]
Goh et al. (2010)	Prediction	0.00776 (1)	[A]	0.07379 (1)	[A]

Note: SSE_{norm} value is between zero to infinity, which a smaller value representing a better predicted results. Value in parentheses in the SSE_{norm} column is the best-prediction ranking for the selected published equations on every set of data. Value of 1 indicates the best prediction for the data set.

The [A] and [D] denote “agreement” and “discrepancy”, respectively, which are defined in text.

Figure 6-54: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts on the first drying path of SK-17 specimens obtained from this research.

Figure 6-55: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts on the first wetting path of SK-17 specimens obtained from this research.

Oberg and Sallfors (1997) equation provides good agreement between the predicted and the measured drying and wetting shear strengths of SK-5 and SK-10. On the other hand, Tekinsoy et al. (2007) equation was found to be able to provide reasonable good predictions on most of the drying and wetting shear strengths of SK-5 and SK-10. However, it was found that the predicted shear strengths from the Tekinsoy et al. (2007) equation are relatively far away from the measured wetting shear strengths of SK-10 as compared to those from the proposed equations. Both equations were found to be able to provide reasonable predictions on the drying and wetting shear strengths only for SK-17 at low matric suctions. Bao et al. (1998) equation is only able to predict some of the drying shear strengths of the soils.

Figure 6-56, Figure 6-57 and Figure 6-58 show the comparison between the measured and the predicted values for SK-5, SK10 and SK-17, respectively, using the proposed equations. It can be observed that the proposed equations slightly under-estimate most of the drying and wetting shear strengths for these soils. This could be attributed to the effect of net confining pressure on soils that is not considered and included in the proposed equations.

Figure 6-56: Measured versus predicted cohesion intercept of SK-5 specimens obtained from this research using the proposed equations (Goh et al. (2010) equation).

Figure 6-57: Measured versus predicted cohesion intercept of SK-10 specimens obtained from this research using the proposed equations (Goh et al. (2010) equation).

Figure 6-58: Measured versus predicted cohesion intercept of SK-17 specimens obtained from this research using the proposed equations (Goh et al. (2010) equation).

6.6.2 Comparisons using Data from Literature

The published shear strength data from Vanapalli et al. (1996a) and Kayadelen et al. (2007) were used in the comparisons. Both sets of data were not used in the development of the proposed equations as elaborated in 3.7. Figure 6-59 and Figure 6-60 present the comparisons of the predicted shear strength envelopes from the proposed equations and various published prediction type equations using the shear strength data from Vanapalli et al. (1996a) and Kayadelen et al. (2007), respectively. Table 6-5 summarizes the statistical analyses of the comparisons between the proposed equations and the published equations using Kayadelen et al. (2007) and Vanapalli et al. (1996a) data.

Table 6-5: The comparisons between the proposed equation and the published equations using the shear strength data from Kayadelen et al. (2007) and Vanapalli et al. (1996a).

Authors	Nature of equation	Kayadelen et al. (2007) study		Vanapalli et al. (1996a) study	
		SSE_{norm}		SSE_{norm}	
Shen and Yu (1996, Eq. 1)	<i>Fitting</i>	0.0050 (2)	[A]	0.0315 (3)	[A]
Shen and Yu (1996, Eq. 2)	<i>Fitting</i>	1.4013	[D]	4.6379	[D]
Vanapalli et al. (1996b, Eq.1)	<i>Fitting</i>	0.0327 (4)	[A]	0.0125 (1)	[A]
Vanapalli et al. (1996b, Eq. 2)	Prediction	0.1857	[D]	0.0380 (4)	[A]
Oberg and Sallfors (1997)	Prediction	0.2414	[D]	1.0308	[D]
Bao et al. (1998)	Prediction	0.2296	[D]	0.0725	[A]
Khalili and Khabbaz (1998)	Prediction	0.4067	[D]	1.5767	[D]
Rassam and Cook (2002)	<i>Fitting</i>	0.0400	[A]	0.4860	[A]
Tekinsoy et al. (2004)	Prediction	0.0528	[A]	0.3116	[A]
Lee et al. (2005)	<i>Fitting</i>	0.0030 (1)	[A]	0.0381 (5)	[A]
Garven and Vanapalli (2006)	Prediction	0.0372 (5)	[A]	0.0441	[A]
Vilar (2006)	<i>Fitting</i>	0.2084	[D]	0.3304	[A]
Proposed equation (Goh et al., 2010)	Prediction	0.0184 (3)	[A]	0.0174 (2)	[A]

Note: SSE_{norm} value is between zero to infinity, which a smaller value representing a better predicted results. Value in parentheses at SSE_{norm} column is the best-prediction ranking for the selected published equations on every set of data. Value of 1 indicates the best prediction for the data set.

The [A] and [D] denote “agreement” and “discrepancy”, respectively, which are defined in text.

Figure 6-59: Comparison between predicted shear strength envelopes from the proposed equations and the selected published prediction type equations using the experimental soil data (Indian Head till) from Vanapalli et al. (1996a) study (Goh et al., 2010).

Figure 6-60: Comparison between predicted shear strength envelopes from the proposed equations and the selected published prediction type using the experimental soil data (Diyarbakir residual soil) from Kayadelen et al. (2007) study (Goh et al., 2010).

As shown in Figure 6-59 and Figure 6-60, the predicted shear strength envelopes from the proposed equations agreed closely with the measured shear strength values from both studies. The proposed equations were found to be able to provide the least prediction errors (the lowest SSE_{norm}) among all of the prediction type equations and the errors are close to those from the fitting type equations, i.e. the predictions from Vanapalli et al. (1996b, Eq.1) equation on Vanapalli data and the predictions from Lee et al. (2005) equation on Kayadelen data. Furthermore, the predicted shear strength envelopes from the proposed equations are in a continuous form of nonlinear curve, demonstrating the nonlinear shear strength behaviour of an unsaturated soil, and show a smooth transition from the matric suction range below AEV to that above AEV.

The predicted shear strength envelopes from Garven and Vanapalli (2006) equation are reasonably close to the measured data points from both studies. Vanapalli et al. (1996b, Eq. 2) equation provides a good agreement between the predicted shear strengths and the experimental shear strength data from Vanapalli et al. (1996a) study; however, it only provides moderate predictions on the experimental shear strength data from the Kayadelen et al. (2007) study. The predictions from Bao et al. (1998) equation were found to be similar as the predictions from Vanapalli et al. (1996b, Eq. 2) equation. Tekinsoy et al. (2004) equation is only able to provide moderately agreement between predicted and measured shear strength from both studies.

As illustrated in Figure 6-61, the predicted shear strength envelopes from the proposed equations seem to under-estimate slightly the measured shear strengths at higher matric suction from both Vanapalli et al. (1996a) and Kayadelen et al. (2007). Figure 6-61 shows the comparisons between the predicted shear strength values from the proposed equations and the measured shear strength values from Vanapalli et al. (1996a) and Kayadelen et al. (2007).

In summary, the proposed shear strength prediction equations developed in this research are generally able to predict the drying and wetting shear strengths of different soil successfully. Similar findings were reported in Goh et al. (2010).

Figure 6-61: Predicted cohesion intercepts from the proposed equations (Goh et al. (2010) equation) versus measured cohesion intercepts from Vanapalli et al. (1996a) and Kayadelen et al. (2007) studies.

6.7 PRACTICAL SIGNIFICANCE OF RESEARCH FINDINGS

Series of SWCC, unsaturated shear strength and permeability test result and analyses from this research contribute significantly to the unsaturated soil mechanics database and literature for both researchers and practical engineers. These results and findings will help engineers to understand and estimate the conditions and behaviours of an unsaturated soil such as compacted soils under hysteresis effects associated to cyclic drying and wetting processes. The observed behaviors of the multi-cycle drying and wetting SWCCs are important for practical significances. The relatively insignificant differences between the volumetric water content and volume of soil on the drying and wetting paths after several cycles as compared to those on the first cycle, can assist the engineers to use the suitable SWCC parameters for unsaturated soil functions in the implementation of unsaturated soil mechanics in geotechnical designs.

The findings on the drying and wetting shear strength characteristics of soil under the initial cycle are important for engineers in designing and modeling geotechnical structures. The understanding on the differences between the drying and wetting shear strengths helps engineers to use the appropriate characteristics and parameters for solving geotechnical problems such as slope stability. The differences between the shear strengths on the drying and wetting paths under several cycles have been shown to be less significant as compared to those under the first cycle. This provides important information to engineers when faced with geotechnical problems associated to the cyclic drying and wetting due to climate changes.

The proposed prediction type shear strength equations developed in this research can be used to predict the drying and wetting shear strengths of unsaturated soil under the initial cycle of drying and wetting. The unsaturated shear strengths of soil on the initial drying and wetting paths have been shown to be different in this study and literature. Engineers may use the proposed equations to predict the initial wetting shear strength of soil, which is crucial for newly constructed slopes and other geotechnical structures during rainfalls. In addition, the estimation on the shear strengths after several cycles may also be predicted using the proposed equations since the differences between the drying and wetting shear strengths of soil after several cycles is relatively insignificant. These

information are useful in the evaluation and design of the geotechnical structures associated to cyclic climate changes.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

This chapter presents the conclusions made from this study. Subsequently, the recommendations for future study are also presented in this chapter.

7.1 CONCLUSIONS

The conclusions made from this study are summarized as follows:

1. The SWCC experimental results show that the drying and wetting SWCCs of soils were different due to hysteresis. The difference between the drying and wetting SWCCs of the first cycle were found to be larger than those of the subsequent cycles. This could be attributed to the inelasticity of soil during drying and wetting processes. However, the difference between the drying and wetting SWCCs of the second and third cycles were found to be less significant due to the similar volume change characteristics of soil in both cycles.
2. The multi-cycled SWCC test results show that the volume of soils on the subsequent cycles of drying and wetting was lower than that of soils on the previous cycles of drying and wetting. The soils with smaller volume and pore sizes had more capability to retain water during drying process and had a slightly lower volumetric water content at low matric suction range than the soils with larger volume and pore sizes. As a result, the volumetric water content of soils on the subsequent cycles of drying and wetting was lower at low matric suction range but higher at high matric suction range than those of soil on the previous cycles of drying and wetting. In other words, SWCCs are flatter with increasing cycles of drying and wetting processes.
3. The SWCC test results show that the net confining pressures, initial dry densities and grain-size distributions of soils affected the multi-cycled SWCCs of the soils. The drying and wetting SWCCs in subsequent cycles were shown to be located in between the first drying and wetting SWCCs and had similar characteristics as those of the first drying and wetting SWCCs. The gentler the slope of grain-size distribution of a soil,

the gentler the slope of multi-cycle SWCCs of the soil. On the other hand, the higher the applied net confining pressure of a soil, the higher the AEV and the higher the wetting saturated point of the soil. In addition, the higher initial dry density of a soil results in a higher AEV and a higher wetting saturated point of the soil as compared to a soil with the lower initial dry density.

4. The series of unsaturated CD triaxial test results show that the sand-kaolin specimens on the drying paths had a higher shear strength than those on the wetting paths. This could be attributed to the hysteresis effects on the soil during drying and wetting processes. As compared to the specimens on the wetting paths, the specimens on the drying paths had a higher volumetric water content, resulting in a larger wetted area of soil that contributes to the increase in shear strength of the soil when subjected to matric suction.
5. It was found that the sand-kaolin specimens on the first cycle drying path had a higher axial strain at failure and exhibited less stiffness, more ductility and contraction (or less dilation) as compared to those on the first cycle wetting path during shearing. The specimens on the 1st cycle drying path generally had a similar shearing behaviour as that of a normally consolidated soil where the applied matric suction increases to a maximum magnitude to which the soil had never been subjected to prior to shearing. This stress path was similar to the virgin compression curve associated with the conventional consolidation test on a soil.
6. The sand-kaolin specimens on the first cycle wetting path, which had a lower axial strain at failure, exhibited more stiffness, more brittleness and dilation than those on the first cycle drying path during shearing, generally had a similar shearing behaviour as that of an over-consolidated soil. The sand-kaolin specimens were subjected to the maximum matric suction prior to the rewetting to the designated matric suction for shearing. This maximum matric suction which the soil ever experienced could be considered analogous to the pre-consolidation pressure of an over-consolidated soil.
7. The sand-kaolin specimens on the subsequent drying and wetting paths generally exhibited similar behaviour as those on the first cycle wetting path during shearing.

Since the magnitude of the matric suctions applied to the specimens in the subsequent cycles did not exceed the maximum matric suction that the specimens ever experienced, the shearing behaviour of the sand-kaolin specimens on the subsequent cycle of drying and wetting during shearing was similar to those on the 1st cycle wetting path.

8. The post-peak shear strengths of the specimens at different matric suctions on different cycles drying and wetting paths were similar. This could be attributed to fact that the post-peak shearing between the shear planes in the specimens is independent from the hysteresis effects and the contribution of matric suction. The post-peak shear strength of a specimen might be governed by the applied net confining pressure and the effective internal friction angle of the specimen. The results show that the higher the applied net confining pressure on a specimen or the higher the effective internal friction angle of a soil, the higher the post-peak shear strength of the specimen.
9. The shear strengths of the sand-kaolin specimens on the drying paths of the second and third cycles were found to be slightly lower than those of the sand-kaolin specimens on the drying path of the first cycle. On the other hand, the shear strengths of the specimens on the second and third wetting paths were found to be higher than those of the specimens on the first wetting path. These could be attributed to the inelasticity of the volume change characteristics of soil during the cycles of drying and wetting processes. The differences in the volumetric water content on the drying and wetting paths of different cycles resulted in the differences in shear strength of the specimens.
10. The unsaturated permeability test results show that the permeability of sand-kaolin specimens on the drying paths were higher than that of the sand-kaolin specimens on the wetting paths. This could be attributed due to the hysteresis effects on the soils during drying and wetting processes. The characteristics of permeability of the sand-kaolin specimens were found to be analogous to the characteristics of drying and wetting SWCCs of the sand-kaolin specimens. This is due to the fact that water in unsaturated soil can only flow through soil voids that are filled with water in a continuous path.

11. In this research, shear strength equations were categorized according to the nature of equation whether it was fitting or prediction type equations. Eleven sets of shear strength data, which consists of seven sets of shear strength data on the initial drying path and four sets of shear strength data on the initial drying and wetting paths, were selected for the comparison study. Among the fitting type equations, Vanapalli et al. (1996, Eq. 1) equation and Lee et al. (2005) equation were found to be able to provide good estimations on all selected drying and wetting shear strength data. On the other hand, among all the prediction type equations, Garven and Vanapalli (2006) equation was found to be able to predict most of the selected drying shear strength data. However, none of the selected prediction type shear strength equations were able to predict both drying and wetting shear strengths effectively.
12. Prediction type shear strength equations for drying and wetting shear strengths of unsaturated soils were developed successfully. The comparative study showed that the proposed shear strength equations were able to provide the best prediction on the first cycle drying and wetting shear strengths of the three different sand-kaolin mixtures investigated in this research. The predicted drying and wetting shear strength envelopes from the proposed equations demonstrated the nonlinear shear strength behaviour of an unsaturated soil with a continuous and smooth transition from the matric suction range below AEV to that above AEV. In addition, the proposed equations were also shown to predict the experimental shear strength data from literature successfully with the smallest error among all of the prediction type equations.

7.2 RECOMMENDATIONS

The recommendation for future studies of this research can be divided into two categories namely laboratory works and theoretical development.

Laboratory works

The multi-cycled SWCCs of a soil is directly governed by the volume change of the soil. Therefore, the volume change characteristics under multi-cycled drying and wetting processes should be further investigated and studied. Devices for accurate volume change measurements of specimen should be developed and studied.

Different types of soils should be used for further investigation on the hysteresis effects on shear strength of soil. Factors that affect the magnitude of hysteresis of a soil on different cycles of drying and wetting should be further investigated.

Theoretical development

The shear strength equations for predicting the unsaturated shear strength of soil on the drying and wetting paths may be further modified by incorporating the effects of net confining pressure on soil. In addition, the shear strength equations could be further improved by relating or including more soil properties into the shear strength equations.

Hysteresis index should be developed to quantify the magnitude of hysteresis of a soil. It is expected that the hysteresis index can be estimated from the given soil properties easily in order to provide a good first approximation of the magnitude of the hysteresis of a soil.

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APPENDIX

This section (APPENDIX) includes all the secondary information for this research, i.e. figures and tables, which are relatively less important as compared to those included in the main test. A naming convention is adopted for the figures and tables in the appendix in order to ease the way of referring in the main text. The naming convention adopted for figure in appendix as Figure Cx.y.z-n, where the term C means Chapter; “x.y.z” indicates the section numbers; “n” indicates the figure number. For example, “Figure C5.4.2-3” in the main text is referred to the third figure used for Section 5.4.2 (Chapter 5) that is attached in Appendix.

APPENDIX C3.6

Table C3.6.1-1: The soil information for the selected data used in comparison study.

Authors	Material	LL	PL	I_p	G_s	Clay Content (%)	Silt Content (%)	ρ_{dmax} (Mg/m ³)	w_{opt} (%)	AEV (kPa)	$(u_a - u_w)_r$ (kPa)
Thu et al. (2006)	Compacted Kaolin	51	35.6	15.4	2.65	15	85	1.35	22	47	N/A
Melinda et al. (2004)	Bukit Timah Granitic Residual Soil	47	29	18	2.66	26	25	1.747	14.5	60	N/A
Vanapalli et al. (1996b)	Indian Head Glacial Till	35.5	16.8	18.7	2.7	30	42	1.8	16.3	35	3000
Kayadelen et al. (2007)	Diyarbakir Residual Soil	77	32	45	2.67	66	27	1.481	N/A	40	25000
Escario and Juca (1989)	Madrid Clay Sand	28	20	8	2.71	6	7	1.96	N/A	30	12000
Miao et al. (2002)	Nanyang Expansive Soils	58.3	26.5	31.8	2.7	24.8	68.5	1.5	17	25	1500
Lee et al. (2005)	Korea Weathered Granitic Residual Soil	N/A	N/A	NP	2.65	4	9	1.864	10	2.3	12.9
Han et al. (1995)	Bukit Timah Granitic Residual Soil	47	29	18	2.66	26	25	1.75	14.4	10	N/A
Escario and Juca (1989)	Madrid Grey Clay	71	36	35	2.64	72	27	1.29	N/A	110	40000
Escario and Juca (1989)	Red Silty Soil	33	19.4	13.6	2.66	35	48	1.8	N/A	40	33000
Rahardjo et al. (2004)	Jurong Sedimentary formation residual soil	36.4	21.6	14.8	2.72	42	24	1.86	14.2	100	N/A

APPENDIX C3.7

Figure C3.7.2-1: Trend of SSE_{norm} corresponding to the number of fitting parameters used for Madrid clay sand from Escario and Juca (1989) shear strength data.

Figure C3.7.2-2: Trend of SSE_{norm} corresponding to the number of fitting parameters used for Korea weathered granitic soil from Lee et al. (2005) shear strength data.

Figure C3.7.2-3: Trend of SSE_{norm} corresponding to the number of fitting parameters used for Bukit Timah granitic residual soil from Han (1996) drying shear strength data.

Figure C3.7.2-4: Trend of SSE_{norm} corresponding to the number of fitting parameters used for Bukit Timah granitic residual soil from Han (1996) wetting shear strength data.

C3.7.2-5: Trend of SSE_{norm} corresponding to the number of fitting parameters used for Jurong sedimentary residual soil from Rahardjo et al. (2004) wetting shear strength data.

Table C3.7.2-1: SSE_{norm} of two parameters for drying shear strength equations.

SSE_{norm} for 2 parameters (drying shear strength)						
Data source Combination No.	Thu et al. (2006)	Han (1996)	Escario and Juca (1989)	Miao et al. (2002)	Lee et al. (2005)	Sum of SSE_{norm}
1	0.00649	0.00922	0.01038	0.00028	0.00761	0.03397
2	0.00988	2.40670	0.32568	0.01192	0.04980	2.80398
3	0.00486	0.00483	0.00161	0.00086	0.00751	0.01966
4	0.00649	0.00922	0.01038	0.00028	0.00761	0.03397
5	0.00649	0.00922	0.01038	0.00028	0.00761	0.03397
6	0.00605	0.00400	0.01035	0.00028	0.00759	0.02827
7	0.00605	0.00400	0.01035	0.00028	0.00759	0.02827
8	0.00649	0.13086	0.01046	0.00028	0.00896	0.15705
9	0.00321	0.00497	0.00143	0.00035	0.00653	0.01649
10	0.01208	0.59092	0.11392	0.00123	0.11347	0.83162
11	0.01208	0.59092	0.11392	0.00123	0.11347	0.83162
12	0.00503	0.03003	0.00967	0.00029	0.00789	0.05291
13	0.00503	0.03003	0.00967	0.00029	0.00789	0.05291
14	0.00486	0.00483	0.00161	0.00086	0.00751	0.01966
15	0.00649	0.00922	0.01038	0.00028	0.00761	0.03397
16	0.00649	0.00922	0.01038	0.00028	0.00761	0.03397
17	0.00605	0.00400	0.01035	0.00028	0.00759	0.02827
18	0.00611	0.11835	0.00977	0.00029	0.00857	0.14309
19	0.00321	0.00497	0.00143	0.00035	0.00653	0.01649
20	0.00321	0.00497	0.00143	0.00035	0.00653	0.01649
21	0.00315	0.00512	0.00171	0.00032	0.00424	0.01454
22	0.00315	0.00512	0.00171	0.00032	0.00424	0.01454
23	0.01208	0.59092	0.11392	0.00123	0.11347	0.83162
24	0.00503	0.12032	0.00968	0.00029	0.00881	0.14413
25	0.00503	0.12032	0.00968	0.00029	0.00881	0.14413
26	0.00503	0.03003	0.00967	0.00029	0.00789	0.05291
27	0.00503	0.12032	0.00968	0.00029	0.00881	0.14413
28	0.01170	0.22451	0.05690	0.00043	0.16356	0.45711
Average	0.00632	0.18561	0.03202	0.00086	0.02590	0.25071

The constant parameter, y_d was found to be closely related to plasticity index (I_p) of soils in natural logarithm ($\ln$) form, while the constant parameter, b_d was found to be closely related to I_p and the fitting parameter, n_d , from Fredlund and Xing (1994) equation in natural logarithm form. However, the relationships become invalid when I_p equals to zero since natural logarithm of zero or negative real number are undefined in mathematics. Therefore, two positive constant values, namely g and j , were used in y_d and b_d relationships, respectively, in order to generate the relationships that are valid for all I_p values. Five sets of published data which include the published data set with non-plastic soil (I_p equals to zero) from Lee et al. (2005), were used in relating the y_d and b_d to I_p as well as to I_p and n_d , respectively.

The value of g was determined using the best-fit analysis. Various values of g were used for estimating the y_d for the five data sets and SSE_{norm} for each g was determined as shown in Figure C3.7.2-6. The value of 2.7 that results in the lowest SSE_{norm} for predicting y_d was chosen based the results of analysis. Therefore, the constant parameter y_d can be related to I_p using in term of $\ln(I_p + 2.7)$. As a result, the equation for y_d is as follows,

$$y_d = 0.502\ln(I_p + 2.7) - 0.387$$

The value of j was determined using best-fit analysis. Based on the results of analysis (Figure C3.7.2-7), the value of 4.4 for the constant parameter, j gives the minimum SSE_{norm} for estimating the b_d for the five data sets. Therefore, it is selected for the proposed equation. The relationship between the constant parameter, b_d and I_p and n_d (fitting parameter of Fredlund and Xing (1994) equation) can be represented using the following form,

$$b_d = -0.245\{\ln[n_d(I_p + 4.4)]\}^2 + 2.114\{\ln[n_d(I_p + 4.4)]\} - 3.522$$

Figure C3.7.2-6: The trend of SSE_{norm} with respect to j constant.

Figure C3.7.2-7: The trend of SSE_{norm} with respect to g constant.

Table C3.7.2-2: SSE_{norm} of two parameters for wetting shear strength equations.

SSE_{norm} for 2 Fitting Parameters (wetting shear strength)				Sum of SSE_{norm}
Data name	Thu et al. (2006)	Han (1996)	Rahardjo et al. (2004)	
Combination No.	Compacted Kaolin	Bukit Timah Granitic Residual Soil	Jurong Sedimentary Formation Residual Soil	
1	0.16251	0.00529	0.00036	0.16816
2	N/A	3.19879	0.22263	3.42142
3	0.21561	0.00778	0.00104	0.22442
4	0.01407	0.00529	0.00036	0.01972
5	0.16251	0.00529	0.00036	0.16816
6	0.00189	0.00328	0.00034	0.00550
7	0.00189	0.00328	0.00034	0.00550
8	0.00119	0.08573	0.00104	0.08796
9	0.01836	0.00692	0.00029	0.02557
10	0.05830	0.57288	0.02178	0.65295
11	0.05830	0.57288	0.02178	0.65295
12	0.00305	0.01358	0.00048	0.01711
13	0.00305	0.01358	0.00048	0.01711
14	0.04199	0.00778	0.00104	0.05080
15	0.00119	0.00529	0.00036	0.00684
16	0.00119	0.00529	0.00036	0.00684
17	0.00189	0.00328	0.00034	0.00550
18	N/A	0.07151	0.00065	0.07216
19	0.01836	0.00692	0.00029	0.02557
20	0.01836	0.00692	0.00029	0.02557
21	0.00627	0.00602	0.00006	0.01235
22	0.00627	0.00602	0.00006	0.01235
23	0.05830	0.57288	0.02178	0.65295
24	0.00305	0.07301	0.00076	0.07682
25	0.00305	0.07301	0.00076	0.07682
26	0.00305	0.01358	0.00048	0.01711
27	0.00305	0.07301	0.00076	0.07682
28	0.00656	0.17029	0.00181	0.17866

APPENDIX C4.4

Table C4.4.3-1: Calibration data for LVDT and pore-water pressure transducer

Displacement (mm)	LVDT Reading (V)	Displacement (mm)	LVDT Reading (V)	Pressure (kPa)	Transducer reading (V)
0	0.504902	15	0.529337	0	0.498211
1	0.506497	16	0.531001	9	0.500068
2	0.508121	17	0.532571	19	0.502416
3	0.509753	18	0.534211	29	0.504739
4	0.511397	16	0.535890	39	0.507032
5	0.513045	20	0.537500	48	0.509367
6	0.514663	21	0.539100	100	0.521249
7	0.516263	22	0.540769	199	0.544088
8	0.517898	23	0.542410	299	0.567160
9	0.519531	24	0.544020	499	0.613279
10	0.521154	25	0.545647		
11	0.522808	26	0.547309		
12	0.524465	27	0.548914		
13	0.526046	28	0.550525		
14	0.527684	29	0.552170

Figure C4.4.3-1: Calibration of LVDT.

Figure C4.4.3-2: Calibration of pore-water pressure transducer.

Table C4-2: Calibration data for pore-air pressure transducer, load cell and thermal sensor.

Pressure (kPa)	Transducer reading (V)	Pressure (kPa)	Transducer reading (V)	Weight load (kPa)	Load cell reading (V)	Temperature (°C)	Thermal sensor reading (V)
0	0.504096	250	0.561515	174.65	0.514518	22.305	0.58873
9	0.505946	300	0.573020	349.3	0.515352	23.579	0.593765
19	0.508281	349	0.584335	523.95	0.516197	24.435	0.597784
29	0.510608	400	0.595995	698.6	0.517074	25.854	0.602768
39	0.512896	450	0.607645	873.25	0.517963	25.854	0.602768
49	0.515218	499	0.618787	1047.9	0.518795	26.225	0.603541
59	0.517481			1222.55	0.519696	27.806	0.609609
69	0.519860			1397.2	0.520536	28.495	0.612474
80	0.522477			1571.85	0.521388	29.852	0.618889
90	0.524722			1746.5	0.522277		
100	0.526990			1921.15	0.523183		
150	0.538582			2095.8	0.524026		
200	0.549962

Figure C4.4.3-3: Calibration of pore-air pressure transducer.

Figure C4.4.3-4: Calibration of 5kN load cell.

Figure C4.4.3-5: Calibration of thermal sensor.

APPENDIX C5.2

Figure C5.2.1-1: Grain size distribution of soil data (soil counter: 11408) extracted from Soil Vision.

Figure C5.2.1-2: Prediction of SWCC using Arya and Paris PTF, Fredlund and Wilson PTF, Gupta and Larson PTF and Scheinost PTF for soil data (soil counter: 11408) extracted from Soil Vision data base.

Figure C5.2.1-3: Prediction of SWCC using Aubertin PTF, Rawls and Brakensiek PTF, Tyler and Wheatcraft PTF and Vereecken PTF for soil data (soil counter: 11408) extracted from Soil Vision data base.

Figure C5.2.1-4: Grain size distribution of soil data (soil counter: 11326) extracted from Soil Vision data base.

Figure C5.2.1-5: Prediction of SWCC using Fredlund and Wilson PTF and Scheinost PTF for soil data (soil counter: 11326) extracted from Soil Vision data base.

Figure C5.2.1-6: Grain size distribution of coarse kaolin (specification from factory).

Figure C5.2.1-7: Prediction of SWCC using Fredlund and Wilson PTF and Scheinost PTF for coarse kaolin extracted from Thu et al. (2006).

Figure C5.2.1-8: Grain size distribution of Botkin silt extracted from Vanapalli et al. (2000).

Figure C5.2.1-9: Grain size distribution of soil data (soil counter: 11056) extracted from Soil Vision.

Figure C5.2.1-10: Grain size distribution of soil data (soil counter: 10834) extracted from Soil Vision data base.

Figure C5.2.1-11: Grain size distribution of soil data (soil counter: 11539) extracted from Soil Vision data base.

Figure C5.2.1-12: Grain size distribution of soil data (soil counter: 11278) extracted from Soil Vision data base.

APPENDIX C5.3

Figure C5.3.1-1: The relationship between q' and p' for SK-10 obtained from the saturated multistage CD triaxial test with shearing rate of 0.0009mm/min.

Figure C5.3.1-2: The relationship between q' and p' for SK-10 obtained from the saturated multistage CU triaxial test with shearing rate of 0.05mm/min.

Figure C5.3.1-3: The relationship between q' and p' for SK-10 obtained from the saturated multistage CU triaxial test with shearing rate of 0.009mm/min.

Figure C5.3.1-4: The relationship between q' and p' for SK-5 obtained from the saturated multistage CD triaxial test with shearing rate of 0.0009mm/min.

Figure C5.3.1-5: The relationship between q' and p' for SK-17 obtained from the saturated multistage CD triaxial test with shearing rate of 0.0009mm/min.

APPENDIX C5.4

Figure C5.4.1-1: Multi-cycled SWCCs (degree of saturation vs. matric suction) of SK-5 under zero net confining pressure.

Figure C5.4.1-2: Multi-cycled SWCCs (gravimetric water content vs. matric suction) of SK-5 under zero net confining pressure.

Figure C5.4.1-3: The void ratio of SK-5 during multi-cycled drying and wetting processes.

Figure C5.4.1-4: Multi-cycled SWCCs (degree of saturation vs. matric suction) of SK-10 under zero net confining pressure.

Figure C5.4.1-5: Multi-cycled SWCCs (gravimetric water content vs. matric suction) of SK-10 under zero net confining pressure.

Figure C5.4.1-6: The void ratio of SK-10 during multi-cycled drying and wetting processes.

Figure C5.4.1-7: Multi-cycled SWCCs (degree of saturation vs. matric suction) of the SK-17 specimen with initial dry density of 1.86 Mg/m^3 under zero net confining pressure.

Figure C5.4.1-8: Multi-cycled SWCCs (gravimetric water content vs. matric suction) of the SK-17 specimen with initial dry density of 1.86 Mg/m^3 under zero net confining pressure.

Figure C5.4.1-9: The void ratio of the SK-17 specimen with initial dry density of 1.86 Mg/m^3 during multi-cycled drying and wetting processes.

Figure C5.4.1-10: Multi-cycled SWCCs (degree of saturation vs. matric suction) of the SK-17 specimen with initial dry density of 1.77 Mg/m^3 under zero net confining pressure.

Figure C5.4.1-11: Multi-cycled SWCCs (gravimetric water content vs. matric suction) of the SK-17 specimen with initial dry density of 1.77 Mg/m^3 under zero net confining pressure.

Figure C5.4.1-12: The void ratio of the SK-17 specimen with initial dry density of 1.77 Mg/m^3 during multi-cycled drying and wetting processes.

SWCC under net confining pressure

Figure C5.4.2-1: 1st cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-5 specimens under a net confining pressure of 50kPa.

Figure C5.4.2-2: 1st cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-5 specimens under a net confining pressure of 50kPa.

Figure C5.4.2-3: 2nd cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-5 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-4: 2nd cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-5 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-5: 1st cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-10 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-6: 1st cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-10 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-7: 2nd cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-10 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-8: 2nd cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-10 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-9: 1st cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-10 specimens under a net confining pressure of 100 kPa.

Figure C5.4.2-10: 1st cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-10 specimens under a net confining pressure of 100 kPa.

Figure C5.4.2-11: 1st cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-17 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-12: 1st cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-17 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-13: 2nd cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-17 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-14: 2nd cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-17 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-15: 3rd cycle drying and wetting SWCCs (gravimetric water content vs. matric suction) of SK-17 specimens under a net confining pressure of 50 kPa.

Figure C5.4.2-16: 3rd cycle drying and wetting SWCCs (degree of saturation vs. matric suction) of SK-17 specimens under a net confining pressure of 50 kPa.

APPENDIX C5.5

Unsaturated CD triaxial test results of SK-5 under 50kPa net confining pressure

Figure 5.5.1-1: Change of water content of the SK-5 specimens during shearing at matric suctions of 100 and 300 kPa on the 1st drying path under 50 kPa net confining pressure.

Figure 5.5.1-2: Degree of saturation of the SK-5 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.1-3: Change of water content of the SK-5 specimens during shearing at matric suctions of 100 and 300 kPa on the 1st cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.1-4: Degree of saturation of the SK-5 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5.5.1-5: Change of water content of the SK-5 specimen during shearing at matric suction of 300 kPa on the 2nd cycle drying path under 50 kPa net confining pressure.

Figure 5.5.1-6: Degree of saturation of the SK-5 specimens during shearing at matric suction of 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.1-7: Change of water content of the SK-5 specimen during shearing at matric suction of 300 kPa on the 2nd cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.1-8: Degree of saturation of the SK-5 specimens during shearing at matric suction of 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Unsaturated CD triaxial test results of SK-10 under 50kPa net confining pressure

Figure 5.5.2-1: Change of water content of SK-10 specimens during shearing at matric suctions of 100 and 300 kPa on the 1st drying path under 50 kPa net confining pressure.

Figure 5.5.2-2: Degree of saturation of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.2-3: Change of water content of SK-10 specimens during shearing at matric suctions of 100 and 300 kPa on the 1st cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.2-4: Degree of saturation of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5.5.2-5: Change of water content of SK-10 specimens during shearing at matric suctions of 100 and 300 kPa on the 2nd cycle drying path under 50 kPa net confining pressure.

Figure 5.5.2-6: Degree of saturation of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.2-7: Change of water content of SK-10 specimens during shearing at matric suctions of 100 and 300 kPa on the 2nd cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.2-8: Degree of saturation of the SK-10 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Unsaturated CD triaxial test results of SK-10 under 100kPa net confining pressure

Figure 5.5.2-9: Change of water content of SK-10 specimens during shearing at matric suctions of 50 and 200 kPa on the 1st cycle drying path under 100 kPa net confining pressure.

Figure 5.5.2-10: Degree of saturation of the SK-10 specimens during shearing at matric suctions of 50 kPa and 200 kPa on the 1st cycle drying path under a net confining pressure of 100 kPa.

Figure 5.5.2-11: Change of water content of SK-10 specimens during shearing at matric suctions of 50 and 200 kPa on the 1st cycle wetting path under 100 kPa net confining pressure.

Figure 5.5.2-12: Degree of saturation of the SK-10 specimens during shearing at matric suctions of 50 kPa and 200 kPa on the 1st cycle wetting path under a net confining pressure of 100 kPa.

Figure 5.5.2-13: The trends of ambient temperature change of SK-10 specimen during shearing at matric suction of 50 kPa on the 1st cycle wetting path under 100 kPa net confining pressure.

Unsaturated CD triaxial test results of SK-17 under 50kPa net confining pressure

Figure 5.5.3-1: Change of water content of SK-17 specimens during shearing at matric suctions of 100 and 300 kPa on the 1st cycle drying path under 50 kPa net confining pressure.

Figure 5.5.3-2: Degree of saturation of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.3-3: Change of water content of SK-17 specimens during shearing at matric suctions of 100 and 300 kPa on the 1st cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.3-4: Degree of saturation of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Figure 5.5.3-5: Change of water content of SK-17 specimens during shearing at matric suctions of 100 and 300 kPa on the 2nd cycle drying path under 50 kPa net confining pressure.

Figure 5.5.3-6: Degree of saturation of the SK-17 specimens during shearing at matric suctions of 100 kPa and 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.3-7: Change of water content of SK-17 specimens during shearing at matric suctions of 100 and 300 kPa on the 2nd cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.3-8: Degree of saturation of the SK-17 specimens during shearing at matrix suctions of 100 kPa and 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

Figure 5.5.3-9: Change of water content of SK-17 specimen during shearing at matrix suction of 300 kPa on the 3rd cycle drying path under 50 kPa net confining pressure.

Figure 5.5.3-10: Degree of saturation of the SK-17 specimens during shearing at matric suctions of 300 kPa on the 3rd cycle drying path under a net confining pressure of 50 kPa.

Figure 5.5.3-11: Change of water content of SK-17 specimen during shearing at matric suction of 300 kPa on the 3rd cycle wetting path under 50 kPa net confining pressure.

Figure 5.5.3-12: Degree of saturation of the SK-17 specimens during shearing at matric suctions of 300 kPa on the 3rd cycle wetting path under a net confining pressure of 50 kPa.

Fluctuated Total Volume Measurement

The ambient temperature and triaxial cell water temperature were monitored using the thermal sensors during the unsaturated triaxial test. As illustrated in **Error! Reference source not found.** and **Error! Reference source not found.**, the total volumetric strain of the specimen, the ambient temperature (room temperature) and the triaxial cell water temperature were found to fluctuate periodically at the shearing stage. In contrast, the water volumetric strain measurement of the specimen was found to be a smooth curve.

From the observation, it was found that the fluctuation of the total volume of specimen was closely related to the fluctuation of the ambient temperature and the cell water temperature during the test. Both temperatures and the total volume measurement fluctuated in a similar trend but the ambient temperature curve moved slightly in advance as compared to the cell water temperature curve and the total volumetric strain curve. As the ambient temperature increases, the cell water temperature and the total volumetric strain of the specimen also increase, and vice versa. This might be due to the fact that the rates of expansions and contractions ($\text{mm}^3/\text{°C}$) of the triaxial cell and the cell water are different.

The measurement of the water volume of soil was not affected by the fluctuation of room temperature. This might be due to the fact that the soil water is considered insulated from room temperature and the amount of soil water volume measurement involved is relatively small as compared to the cell water.

The fluctuated total volume measurement with respect to the room temperature was not corrected since the fluctuation was considered as minor. The general trend of total volume change of the specimen was not affected. In addition, the effect of the fluctuated value on the analyses and calculations involved total volume data was found to be insignificant.

Temperature and volume changes during shearing of SK-10 specimens at matric suction of 300 kPa on the 1st cycle wetting path under a net confining pressure of 50 kPa.

Temperature and volume changes during shearing of SK-5 specimen at matric suction of 300 kPa on the 2nd cycle drying path under a net confining pressure of 50 kPa.

Temperature and volume changes during shearing of SK-10 specimen at matric suction of 200 kPa on the 1st cycle wetting path under a net confining pressure of 100 kPa.

Temperature and volume changes during shearing of SK-10 specimen at matric suction of 50 kPa on the 1st cycle wetting path under a net confining pressure of 100 kPa.

Temperature and volume changes during shearing of SK-5 specimen at matric suction of 100 kPa on the 1st cycle drying path under a net confining pressure of 50 kPa.

Temperature and volume changes during shearing of SK-10 specimens at matric suction of 300 kPa on the 2nd cycle wetting path under a net confining pressure of 50 kPa.

APPENDIX C6.2

Figure C6.2.1-1: The comparison between the measured SWCC and the predicted SWCC using Fredlund and Wilson PTF and Scheinost PTF for SK-10.

Figure C6.2.1-2: The comparison between the measured SWCC and the predicted SWCC using Fredlund and Wilson PTF and Scheinost PTF for SK-17.

APPENDIX C6.3

Figure C6.3.1-1: Multi-cycle SWCCs of SK-5 specimens under a net confining pressure of 50kPa.

Figure C6.3.1-2: Multi-cycle SWCCs of SK-10 specimens under zero net confining pressure.

Figure C6.3.1-3: Multi-cycle SWCCs of SK-10 specimens under a net confining pressure of 50kPa.

Figure C6.3.1-4: SWCCs of SK-10 specimens under a net confining pressure of 100kPa.

Figure C6.3.1-5: Multi-cycle SWCCs of SK-17 specimens at initial dry density of 1.77 Mg/m^3 under zero net confining pressure.

Figure C6.3.1-6: Multi-cycle SWCCs of SK-17 specimens under a net confining pressure of 50kPa.

Figure C6.3.2-1: The multi-cycle SWCCs of SK-17 specimens under different net confining pressures.

Figure C6.3-1: Multi-cycled SWCCs of SK-17 specimen in normalized volumetric water content under zero net confining pressure.

Figure C6.3-2: The multi-cycled SWCCs of SK-17 specimens in normalized volumetric water content under a net confining pressure of 50 kPa.