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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**COMPRESSION, SHRINKAGE AND WETTING-INDUCED VOLUME
CHANGE OF UNSATURATED SOILS**

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

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**COMPRESSION, SHRINKAGE AND WETTING-INDUCED VOLUME
CHANGE OF UNSATURATED SOILS**

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Summary

The saturated soil mechanics approach has been widely accepted as the basis for geotechnical engineering design. While the saturated soil mechanics approach in most cases gives conservative estimate in terms of shear strength, it is unable to explain drying and wetting-induced volume changes such as shrinkage, swelling and collapse. The understanding of shrinkage and wetting-induced volume change requires the understanding of the loading history of soils. Thus, the understanding of unsaturated soil compression behaviour becomes paramount.

The soil-water characteristic curve is an important property of unsaturated soils as it can be used to estimate many engineering properties of unsaturated soils. The soil-water characteristic curve is affected by the change in volume of the unsaturated soils due to compression, drying or wetting. Current equations to describe the soil-water characteristic curve have limitations. Thus a universal equation which can be used to fit curves with any number of linear segments was proposed. The parameters in the equation can be directly obtained from the soil-water characteristic curve plot. The proposed equation can also be used to curve fit the shrinkage curve. An equation was also proposed to estimate the soil-water characteristic curve of a soil with different initial dry densities or void ratios.

A series of oedometer tests was conducted in this study in order to understand the compression behaviour of unsaturated soil. The oedometer was modified in order to incorporate a high-capacity tensiometer to measure the change in negative pore-water pressure of the unsaturated soil during loading under constant water content condition. The change in negative pore-water pressure provided an understanding on the matric suction decreases during the constant water content test and thus is referred to as mechanical wetting. However, as the suction measurement range of the high-capacity tensiometer is

limited (<1500 kPa), soil specimens which have very low water content (and very high matric suction) were tested in the conventional oedometer without any suction measurement. A series of constant rate of strain tests was also conducted in order to observe the effect of strain rate on unsaturated soils. The effect of strain rate appears to be negligible for unsaturated soils.

Some of the specimens were also subjected to wetting tests in order to observe the wetting-induced behaviour under constant net normal stress or constant volume condition. Based on the observation, a model was proposed to estimate the change in volume due to the change in net normal stress under constant water content condition and due to wetting or drying under different net normal stresses. The proposed model was verified with tests conducted in this study and also test results from the literature. The model was shown to perform well provided that the parameters can be accurately determined. Notwithstanding the accuracy of the parameters, it gives a good qualitative estimate of the volumetric response of the unsaturated soil under different combinations of loading, drying and wetting. The proposed model does not require suction measurement or suction control apparatus and requires only the conventional oedometer to perform constant water content oedometer test. Thus, the proposed model can be applied more readily in practice.

List of Publications

The author published/submitted several journal and conference papers during his Ph.D. candidature as below:

Journal:

1. Wijaya, M., Leong, E. C. & Rahardjo, H. (2015) Effect of shrinkage on air-entry value of soils. *Soils and Foundations* 55(1):166-180.
2. Leong, E. C. & Wijaya, M. (2015) Universal soil shrinkage curve equation. *Geoderma* 237–238(0):78-87.
3. Wijaya, M. and Leong, E.C. (2016). Swelling and Collapse of Unsaturated Soils Due to Inundation Under One-Dimensional Loading, *Indian Geotechnical Journal*, 46(3): 239–251. DOI 10.1007/s40098-015-0172-4
4. Wijaya, M. & Leong, E. C. (2016) Equation for unimodal and bimodal soil–water characteristic curves. *Soils and Foundations* 56(2):291-300
5. Wijaya, M. & Leong, E. C. (2016) Performance of high-capacity tensiometer in constant water content oedometer test. *International Journal of Geo-Engineering* 7(1):13.
6. Wijaya, M. & Leong, E. C. (2016) Modelling the effect of density on soil-water characteristic curve (**accepted in Géotechnique**).

Conference:

1. Wijaya, M., Leong, E. C. & Rahardjo, H. (2014) CRS compression tests of an unsaturated kaolin with pore-water pressure measurement In *Proceedings of Unsaturated soils: Research & Applications* (Khalili, N., Russell, A. R., and Khoshghalb, A. (eds)). CRC Press Taylor & Francis vol. 2.
2. Wijaya, M. & Leong, E. C. (2014) Modelling shrinkage behaviour of soft soils. In *Softsoils 2014*. (Rahardjo, P., and Bakar, I. (eds)), Bandung, vol. 2.

3. Wijaya, M. & Leong, E. C. (2015) Estimation of Soil Shrinkage Curve In Proceedings of Unsaturated soils: Research & Applications. CRC Press/Balkema vol. 1, pp. 785-789.

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List of Symbols

a_f, n_f, m_f	:	Fredlund and Xing (1994) curve fitting parameters
$a_G, b_G, e_r, m_G, \phi_G$	:	Gould et al. (2011) curve fitting parameters
AE	:	Air entry point which is the water content at air entry value
AEV	:	Air entry value
Case I	:	Condition where the unsaturated specimen becomes saturated due to the compression prior to reaching NCL
Case II	:	Condition where the unsaturated specimen remains unsaturated after reaching NCL
C_c	:	Slope of the NCL
c_g	:	Compressibility of grains
C_r	:	Generic term for the slope of unloading line which can be either $C_{r,d}$ or $C_{r,s}$
$C_{r,d}$	:	Slope of the unloading line under constant water content condition
$C_{r,d}(S_c)$	:	Slope of the unloading line under constant water content condition as a function of S_c
c_i	:	Curvature parameter for the soil-water characteristic curve equation
C_i	:	Slope of the initial compression line where its value varies from 0 to C_r
c_s	:	Compressibility of the granular skeleton
$C_{r,s}$	:	Slope of the unloading line under saturated condition
CRS	:	Constant rate of strain
CV	:	Constant volume test
CWC	:	Constant water content compression test without pore-water pressure measurement

CWC-P	:	Constant water content compression test with pore-water pressure measurement
D_r	:	Relative density
e	:	Void ratio
e_c	:	Preconsolidation void ratio
e_c'	:	Preconsolidation void ratio to calibrate $C_{r,d}$
$e_{c,d}$	:	Preconsolidation void ratio prior to inundation
$e_{c,s}$	:	Preconsolidation void ratio after inundation
$e_{d,i}$	:	Void ratio before inundation at inundation pressure
$e_{s,i}$	:	Void ratio after inundation at inundation pressure
e_{j-1}	:	Previously calculated void ratio
e_m	:	microstructural void ratio
e_{max}	:	Maximum void ratio of the coarse grained soils, correspond to very dense condition
e_{min}	:	Minimum void ratio of the shrinkage curve
e_{min-n}	:	Natural minimum void ratio
e_{min}^*	:	Minimum void ratio of the coarse grained soils, correspond to very loose condition
e_{NCL}	:	Void ratio along the NCL
$e_{N,s}$	:	Void ratio of the NCL at net normal stress equals to 1 kPa
$e_{N,d}$	:	Void ratio of the UCL at degree of saturation equals to 0 and at net normal stress equals to 1 kPa
$e_N(S_c)$	:	Void ratio on the UCL at constant preconsolidation degree of saturation and at apparent preconsolidation pressure equals to 1 kPa
$e_N(S_{c,i})$	:	Void ratio on the UCL at arbitrary constant preconsolidation degree of saturation and at apparent preconsolidation pressure equals to 1 kPa

which is determined by using void ratio on the unsaturated compression line with degree of saturation equals to $S_{c,i}$

e_0	:	Initial void ratio
EOP	:	End of primary consolidation
e_0'	:	Initial void ratio to calibrate $C_{r,d}$
$e_0(w)$	:	Initial void ratio as a function of water content where its value is defined by the shrinkage curve
e_{UCL}	:	Void ratio along UCL
$e(w)$	:	Void ratio as a function of water content which is obtained from the shrinkage curve
$e(v)$	:	Void ratio as a function of moisture ratio which is obtained from the shrinkage curve
$f(x)$	:	A function of multi-linear segments curve
$f(x_1)$	:	The value of $f(x)$ at $x = x_1$ which gives the intercept of the function
$f_1(s)$	:	Dimensionless SWCC function which corresponds to the macropores
$f_2(s)$	:	Dimensionless SWCC function which corresponds to the micropores
G_s	:	Specific gravity
HAE	:	High air-entry
HCT	:	High-capacity tensiometer
$H(x, x_i)$	:	Heaviside unit step function which is equals to 0 at $x < x_i$ and 1 at $x > x_i$
$H(x, x_i, k_i)$	:	Approximate Heaviside unit step function which tends to 0 at $x < x_i$ and tends to 1 at $x > x_i$
h_i'	:	Height of the specimen at the inundation pressure prior to inundation
h_i''	:	Height of the specimen at the inundation pressure after inundation
i_{mp}	:	Collapsing coefficient

k_i	:	Curvature parameter between segment i and segment $i-1$ of the shrinkage curve equation
LL	:	Liquid limit
LVDT	:	Linear variable differential transformer
m_i	:	Slope of linear segment i
n	:	Number of linear segments
n_i	:	Instantaneous porosity
n_0	:	Initial porosity
NCL	:	Normal compression line, which is compression line after the compression curve reached saturated preconsolidation pressure
p	:	Net normal stress
P_c	:	preconsolidation pressure which can be either $P_{c,s}$ or $P_{c,d}$
$P_{c,d}$	:	Apparent preconsolidation pressure / preconsolidation pressure under unsaturated condition
$P_{c,d}(s)$	:	Apparent preconsolidation pressure / preconsolidation pressure under unsaturated condition as a function of matric suction
$P_{c,d}(S_c)$	:	Apparent preconsolidation pressure / preconsolidation pressure under unsaturated condition as a function of preconsolidation degree of saturation
$P_{c,s}$	:	Saturated preconsolidation pressure / preconsolidation pressure under saturated condition
P_i	:	Inundation pressure
PI	:	Plasticity index
PL	:	Plastic limit
P_0	:	Net normal stress at $e = e_0$
P_s	:	Post-wetting net normal stress

P_T	:	Transition pressure which is the net normal stress that indicates whether swelling or collapse will occur
r, β_a	:	Curve fitting parameter for $\lambda(s)$ in BBM
$R_i(x, x_i, k_i)$	:	Ramp function
R^2	:	Coefficient of determination
s	:	Matric suction
S_c	:	Degree of saturation at preconsolidation void ratio
S_e	:	Effective degree of saturation
S_f	:	Degree of saturation after inundation and usually equals to 100%
s_i	:	Matric suction at the intersection between segment i and segment i-1
s_{i-}	:	Matric suction which serves as the point of departure between segment i and segment i-1
s_{i+}	:	Matric suction which serves as the point of convergence between segment i and segment i-1
S_0	:	Initial degree of saturation
S_r	:	Degree of saturation
s_r	:	Residual matric suction
$S_{r,i}$	:	Arbitrary degree of saturation on the UCL which is used to determine $e_N(S_{r,i})$
$S_r(s)$	:	Degree of saturation as a function of matric suction which can be obtained from soil-water characteristic curve based on degree of saturation
$S_r(s, e)$	:	Degree of saturation as a function of matric suction and void ratio
SCT	:	Saturated compression test
SL	:	True shrinkage limit

SL'	:	Apparent shrinkage limit or the intersection between zero shrinkage line and saturation line at at the shrinkage curve
S_m	:	Matric suction at the merging point of bimodal SWCC
s_2	:	Matric suction at the intersection between virgin drying line and the initial drying line
$s_{2,0}$	:	Measured s_2 when w_s equals to $w_{s,0}$
$s_{2,f}$	:	Estimated s_2 when w_s equals to $w_{s,f}$
SWCC	:	Soil-water characteristic curve
SWCC-S	:	Soil-water characteristic curve based on degree of saturation
SWCC-w	:	Soil-water characteristic curve based on gravimetric water content
SWCC- θ	:	Soil-water characteristic curve based on volumetric water content
s'	:	Effective suction
s^*	:	Modified suction
$T_i(x, x_i, m_i, m_{i-1}, k_i)$	:	Transition function
u_a	:	Pore-air pressure
$u_a - u_w$	:	Matric suction
u_w	:	Pore-water pressure
UCL	:	Unsaturated compression line
w	:	Gravimetric water content
w_i	:	Gravimetric water content at the intersection between segment i and segment i-1 on the shrinkage curve
w_{i-}	:	Gravimetric water content which becomes the point of departure between segment i and segment i-1
w_{i+}	:	Gravimetric water content which becomes the point of convergence between segment i and segment i-1
w_M	:	Saturated water content of the macropores

w_m	:	Saturated water content of the micropores
w_0	:	Initial gravimetric water content
w_s	:	Saturated gravimetric water content
$w_{s,0}$	:	Measured saturated gravimetric water content when s_2 equals to $s_{2,0}$
$w_{s,f}$	:	Estimated saturated gravimetric water content when s_2 equals to $s_{2,f}$
$w(s)$	:	Gravimetric water content as a function of matric suction which can be obtained from soil-water characteristic curve based on gravimetric water content
$w(s,e_0)$	:	Gravimetric water content as a function of matric suction and initial void ratio
$w_1(s)$	:	SWCC equation corresponding to the macropores
$w_2(s)$	:	SWCC equation corresponding to the micropores
x_i	:	x value at the intersection between segment i and segment i-1
x_{i-}	:	Point of departure
x_{i+}	:	Point of convergence
α, β, η	:	Curve fitting parameters of Leong et al. (2013) equation
$\alpha_s(s)$	:	Proportionality coefficient which is bounded from 0.79 to 1
$\delta_i(x, x_i, k_i)$	:	Dirac delta function
$\delta_{\max}$	:	Maximum Dirac delta function
Δe	:	Wetting-induced void ratio change, can be either swelling or collapse settlement
χ	:	Bishop's effective stress parameter
γ_t	:	Total unit weight
κ	:	Slope of the unloading line with natural log as the x axis
μ_1	:	A function which relates total stress and state variables with the constitutive stress

μ_2	:	A function which relates state variables and matric suction with modified suction
$\lambda(0)$	:	Slope of the normal compression line with natural log as the x axis
$\lambda(s)$	:	Slope of the unsaturated compression line compression line with natural log as the x axis
ρ_{dry}	:	Dry density
$\rho_{\text{dry-max}}$	:	Maximum dry density
$\rho_{\text{dry-min}}$	:	Minimum dry density
ρ_t	:	Total density
u	:	Moisture ratio
u_i	:	Moisture ratio at the intersection point between segment i and segment i-1 on the shrinkage curve
u_{i-}	:	Moisture ratio which becomes the point of departure between segment i and segment i-1
u_{i+}	:	Moisture ratio content which becomes the point of convergence between segment i and segment i-1
θ	:	Volumetric water content
θ_s	:	Saturated volumetric water content
$\theta(s)$	:	Volumetric water content as a function of matric suction which can be obtained from the soil-water characteristic curve based on volumetric water content
σ	:	Total stress
σ_3	:	Confining pressure
σ'	:	Effective stress
σ^*	:	Average skeleton stress
$\sigma^{*'} $	:	Constitutive stress

$\sigma - u_a$	:	Net normal stress
ξ_m	:	Ratio of the microstructural void ratio to the void ratio

Chapter 1. Introduction

1.1. Background

The degree of saturation has long been known to affect the mechanical behaviour of soils. When the soil becomes unsaturated, the pore-water pressure becomes negative due to the development of pore-water meniscus which then leads to a less compressible and permeable material with higher shear strength. The term matric suction (pore-water pressure referenced to the pore-air pressure) is commonly used to represent the negative pore-water pressure in the unsaturated soils.

However, incorporating the degree of saturation and matric suction to estimate the mechanical behaviour of unsaturated soils is complicated. Thus, it is quite common to assume that the soil is fully saturated. Such an approach is commonly accepted by practitioners as assuming the soil to be fully saturated gives lower shear strength and the soil becomes more compressible with respect to the change in net normal stress (total stress with reference to the pore-air pressure). Unfortunately, assuming that the soil is fully saturated ignores the fact that soil also undergoes change in volume during drying and wetting. Three notable volume changes due to drying and wetting are shrinkage, swelling and collapse.

Shrinkage is associated with a decrease in volume due to drying (increase in matric suction or decrease in water content) while swelling and collapse are due to wetting (decrease in matric suction or increase in water content). Swelling and shrinkage may cause excessive heave and settlement, respectively. Expansive soils contain montmorillonite clay mineral and swell excessively on wetting (Fredlund and Rahardjo, 1993). In the United States alone, the damage caused by expansive soils in a typical year caused more financial loss than the combined damages from floods, hurricanes, earthquakes and tornadoes (Jones and Holtz, 1973, Geology.com, 2015).

Collapse settlement or sometimes referred to as hydro-consolidation (Rogers et al., 1994) is a phenomenon in unsaturated soils where wetting caused a decrease instead of an increase in volume (swelling). Collapse occurs due to the weakening of the bonding which maintained the macropore as the soil gets saturated and its magnitude depends on the origin, structural-textural features, lithology, age, water content, the clay mineral content and the particle sizes (Grabowska-Olszewska, 1988, Assallay et al., 1998). One example of collapsible soil is Aeolian sediment which is known as loess. The thickness of loess can vary from 10 m thick such as in western Europe up to 300 m thick such as in the Gansu province of China (Derbyshire and Mellors, 1988). Loess has caused lots of catastrophic damages in Poland (Grabowska-Olszewska, 1988), Russia and Ukraine (Rogers et al., 1994). Excessive collapse settlement due to loess that can be in the order of meters (Denisov, 1951, Feda et al., 1993, Rogers et al., 1994) commonly damaged infrastructure includes canals, roads, pipelines, and transmission towers (Knodel, 1981).

Wetting and drying-induced volume change affects matric suction which in turn affects the compressibility of soils. This phenomenon has been observed by a number of researchers through compression tests under either constant matric suction or constant water content condition. Thus, the sequence of change in matric suction and net normal stress is consequential in determining the volume change of unsaturated soils. Ignoring load path dependency in determining the volume change of unsaturated soils may lead to catastrophic failure due to the unexpected change in volume.

A number of methods has been proposed to estimate the volume change of unsaturated soils due to change in net normal stress and matric suction. However, most of these methods rely on tests which require matric suction to be controlled or to be measured which is expensive and could be time consuming. In addition, accurate matric suction measurement remains difficult especially in the field as it is usually limited either by the maximum matric suction that

can be measured or by the long duration that is required for the measurement. Thus, without reliable matric suction measurement, it is difficult to rely on a model which used matric suction as an input parameter. All of these issues caused a significant gap between research and practice (Kodikara, 2012). Therefore, it is acknowledged that current methods have to be made simpler in terms of experimental procedures to reduce expenses and test duration without reducing their accuracy.

1.2. Objective and scope

The objective of this research is to study the compression, shrinkage and wetting induced behaviour of unsaturated soils. The scope of this research includes:

1. To study the interaction between shrinkage, swelling, collapse and compression behaviour of unsaturated soils by using shrinkage test, saturated compression test, constant water content compression test, and inundation test under constant net normal stress and constant volume condition.
2. To study the effect of density on the soil-water characteristic curve by using axis-translation apparatus and chilled-mirror dew point device.
3. To study the effect of compression on the matric suction of unsaturated soils in constant water content compression test by incorporating high capacity tensiometer in the consolidometer.
4. To study the effect of strain rate on the compression of unsaturated soils by using constant rate of strain apparatus.
5. To propose a model to estimate the change in volume due to constant water content loading and due to change in water content under constant net normal stress condition.

The effects of temperature, osmotic suction and secondary compression are not considered in this study. The loading condition which is of interest is one-dimensional loading. Thus, total normal stress and net normal stress is used instead of total stress and net stress, respectively.

1.3. Organisation of reports

This dissertation is organised as follows:

- Chapter 1.** presents the background on the volume change of unsaturated soils due to the change in degree of saturation and net normal stress. The objective and scope of study are described.
- Chapter 2.** reviews the state variables for unsaturated soils, soil-water characteristic curve and shrinkage curve, compression and wetting-induced behaviour of unsaturated soils.
- Chapter 3.** presents the apparatuses, materials and testing procedures which were used in this study.
- Chapter 4.** presents the test results for soil-water characteristic and shrinkage tests. A general equation to curve-fit a multi-linear segments curve which can be applied to the soil-water characteristic and the shrinkage curves by using graphically obtainable parameters is proposed. The general equation for the SWCC was also extended to estimate the SWCC of soil with different densities.
- Chapter 5.** presents the test results on the constant water content compression tests under different degrees of saturation by using oedometer and constant rate of strain (CRS) apparatuses. It also presents the matric suction measurement by using high-capacity tensiometer during constant water content compression test under incremental loading and different strain rates on unsaturated soils.

Chapter 6. proposes a method to estimate volume change due to the change in net normal stress under constant water content condition and volume change due to the change in water content under constant net normal stress condition.

Chapter 7. presents the conclusion and the recommendations for further study.

Chapter 2. Literature review

2.1. Introduction

The presence of negative pore-water pressure in unsaturated soil affects its shear strength, compressibility and permeability. The volume change of unsaturated soils can be due to the change in either its total normal stress σ or in its water content w which is usually translated as a change in negative pore-water pressure u_w or matric suction s which is u_w as referenced to the pore-air pressure u_a . The effects of σ and u_w on the volume change of unsaturated soil are interdependent and thus the volume change of unsaturated soil is far more complicated than the case of saturated soil. For instance, besides compression, unsaturated soils can also experience swelling, shrinkage and collapse. Such phenomena may cause catastrophic damages if not properly accounted for. Thus, understanding the volume change behaviour of unsaturated soil is important.

In order to study the volume change behaviour of unsaturated soils, the understanding of state variables, the soil-water characteristic curve (SWCC), shrinkage curve, wetting-induced behaviour and compression behaviour of unsaturated soils are important and are reviewed in this chapter.

2.2. State variables of unsaturated soils

In order to represent the unsaturated condition of soils, different state variables such as negative pore-water pressure, matric suction, degree of saturation, gravimetric water content, net normal stress and effective stress have been used. The determination of the state variables plays a pivotal role as it affects the way the unsaturated soil is being tested and also the development of models to estimate its behaviour. However, determining which state variables to use is still contentious among researchers. Thus, identifying the most appropriate and practical approach in explaining the behaviour of unsaturated soil remains a highly active

research area (Lu and Likos, 2006). In most of the cases, stress-state variables such as matric suction and net normal stress are preferred as the state variables. The disagreements over which state variables to use are usually based on:

1. Appropriateness to use soil properties in the stress-state variables.
2. Choice of state variables which can ensure continuity between unsaturated and saturated states.
3. Difficulties in obtaining the state variables in the laboratory and on the field.

There are four approaches which are commonly used to represent the unsaturated condition in soil, which are

1. Effective stress approach,
2. Two independent stress-state variables approach,
3. Modified stress variables approach, and
4. Water content approach.

Each approach has its own advantages and disadvantages. In the following sections, each approach is reviewed.

2.2.1. Effective stress approach

The compressibility of saturated soils is controlled by a single stress-state variable, the effective stress σ' (Terzaghi, 1936). The efficacy of the effective stress concept in saturated soils has encouraged researchers to extend this concept to unsaturated soils (Croney et al., 1958, Bishop, 1959, Aitchison, 1961, Jennings, 1961, Aitchison, 1965, Richards, 1966) which has been highly criticized as inadequate by others (Jennings and Burland, 1962, Morgenstern, 1979).

The main advantage of using a single effective stress approach is the simplification in the expression of unsaturated soil behaviour as the continuity between unsaturated and saturated

state is ensured. Bishop (1959) effective stress equation is perhaps the most famous effective stress approach for unsaturated soil and is given as:

$$\sigma' = (\sigma - u_a) + \chi s \quad (2.1)$$

where χ is the effective stress parameter and $\sigma - u_a$ is commonly referred to as net normal stress. The χ parameter describes the effect of matric suction on the effective stress and is usually related to the degree of saturation S_r where $\chi = 1$ when $S_r = 100\%$ and $\chi = 0$ when $S_r = 0$. Karube et al. (1996) proposed a similar approach where the term χs is referred as suction stress with χ parameter is obtained from the effective degree of saturation S_e as a function of matric suction s .

The use of effective stress in unsaturated soils is considered inappropriate by a number of researchers as soil properties is used to estimate the value of χ . Morgenstern (1979) and Fung (1977) suggested that the stress-state variables should be related to equilibrium alone and to link it to deformations through constitutive behaviour instead of introducing constitutive behaviour into the stress variable. To refute this idea, Lu and Likos (2006) cited the work of Skempton (1960) who re-expressed the effective stress for saturated soil by incorporating soil properties as:

$$\sigma' = \sigma - \left(1 - \frac{c_g}{c_s} \right) u_w \quad (2.2)$$

where c_g is the compressibility of grains, c_s is the compressibility of the granular skeleton and c_g/c_s is the compressibility ratio which is coincidentally negligible and therefore can be ignored. Khalili et al. (2004) also argued that net normal stress is a macro-scale concept while matric suction is a micro-scale concept and thus required a scaling factor to take into account the difference in the scale which can be represented by χ .

The debate over whether soil properties can be incorporated into the stress-state variables is still unresolved. Thus, it is important to examine the appropriateness of using a single effective stress concept based on the experimental evidence. The experimental evidence showed that:

1. The value of χ when determined for volume change was found to be different when determined for shear strength (Morgenstern, 1979). Thus, the value of effective stress for unsaturated soil is different when it is used to determine shear strength and when it is used to determine volume change.
2. The collapse phenomenon which was observed in the double oedometer test by Jennings and Burland (1962) invalidates the concept of effective stress as the inundation which decreases the value of effective stress causes the soil to decrease in volume while it is expected to increase in volume.
3. The χ parameter is commonly related to degree of saturation which often leads to significant error and unrealistic value especially for clay (Alonso et al., 2010).

These experimental evidence discourage the use of effective stress in unsaturated soils and thus hinder its developments (Frydman and Baker, 2009). Several attempts have been made to re-interpret this experimental evidence. For example, Lu (2011) argued that the collapse phenomenon cannot be explained by the effective stress concept because Jennings and Burland (1962) restricted the interpretation of the state of stress and volumetric changes prior to the failure and therefore overlook the failure and post-failure behaviour. Lu (2011) interpreted that the decrease in effective stress due to inundation actually causes the soil's Mohr-Coulomb circle to exceed the limit of the Mohr-Coulomb envelope and therefore causes the soil to fail.

Despite these debates, the acceptance of effective stress concept in unsaturated soils may require additional extensive experimental evidence to prove that such a concept can be used

to explain the collapse phenomenon and there is a unique χ function which can be used to estimate the volume change and shear strength of unsaturated soils.

2.2.2. Independent stress-state variables approach

In this approach, different combinations of stress-state variables are used to describe the behaviour of unsaturated soils. Fredlund (1973), Fredlund and Morgenstern (1976) and Fredlund and Morgenstern (1977) proposed that any two out of the three possible combinations of stress variables can be used as the stress-state variables which are:

1. $\sigma - u_a$
2. s
3. $\sigma - u_w$

Out of these three possible combinations, it appears that net normal stress ($\sigma - u_a$) and matric suction (s) are favoured (Matyas and Radhakrishna, 1968, Barden et al., 1969) and seems to be the most satisfactory for engineering practice (Fredlund, 1979, Fredlund and Rahardjo, 1987).

In a constant water content test, u_a equals to 0 kPa and thus net normal stress is equal to σ . However, when the unsaturated soil approaches saturation, u_w equals to the u_a and thus s becomes zero while net normal stress becomes effective stress for saturated soils.

In this approach, the investigation on unsaturated soil behaviour is commonly done using the “divide and conquer” method where the effect of one stress-state variable on the unsaturated soil is investigated with the other stress-state variable is kept constant (Zhang, 2016a). A parameter relating the change in one stress-state variable to the unsaturated soil properties can thus be obtained with complexities ranging from as simple as a constant parameter to a very complex parameter which is related to the stress-state variables and soil properties. Thus, several constant variable planes such as constant water content plane, constant volume plane,

constant suction plane and constant net normal or mean stress planes are commonly used for the investigation as shown in Figure 2-1a.

Figure 2-1 Constant variable planes and stress paths which were used in this study

Different relationships have been used to describe the volume change of soil under different constant stress planes. For instance, shrinkage curve and soil-shrinkage characteristic curve (SSCC) can be used to describe the deformation of soils due to the change in w and s in the constant net normal stress plane, respectively (Salager et al., 2010, Fredlund et al., 2011) while compression curve can be used to describe the deformation of soils due to the change in net

normal stress under constant matric suction plane. Another important constant variable plane is the constant water content plane. Soil which is instantaneously compressed due to the change in net normal stress in the field is subjected to the constant water content condition leading to changes in both volume and matric suction of the soil. Figure 2-1b illustrates a soil which is dried following the constant net normal stress plane (path 3 - 4), and then compressed under constant water content plane (path 4 – 5) and then inundated under constant net normal stress plane (path 6 – 7).

Lu and Likos (2006) criticized that the parameters which are used in the independent stress-state variables approach such as the friction angle which represents the contribution of matric suction to shear strength have uncertainties in their uniqueness over a wide range of saturation and thus, limits its practical applicability. Other example is the compression indices of soils where some researchers believe that it is either decreasing with increasing suction (Alonso et al., 1990, Alonso et al., 1999, Pham, 2005, Alonso et al., 2013), increasing with increasing suction (Toll, 1990, Alonso, 1993, Wheeler and Sivakumar, 1995, Honda, 2000, Sivakumar and Wheeler, 2000, Sun et al., 2000, Futai et al., 2002, Silva et al., 2002) or is unaffected by matric suction (Larson et al., 1980, O'Sullivan, 1992, Smith et al., 1997, Arvidsson and Keller, 2004, Imhoff et al., 2004, Berli et al., 2015). These contradictory observations on the compression index lead to difficulties in interpreting the unsaturated soil behaviour.

2.2.3. Modified stress variables approach

Modified stress variables approach is a combination of effective stress and independent stress-state variables approaches. In this approach, soil properties are incorporated into the stress-state variables. However, modified stress variables approach does not try to find a single stress-state variable as it is possible to use two modified stress variables to describe the behaviour of unsaturated soil just like in the independent stress-state variables approach.

This approach was developed based on the hypothetical question on how matric suction can affect the soil skeleton. Thus, soil properties are incorporated in order to take into account the effect of both net normal stress and matric suction.

Houlsby (1997) derived a modified stress which represents the effect of normal stress on the unsaturated soil based on the work conjugate principle for unsaturated soils which is defined as average skeleton stress by Jommi (2000) and was adopted by other researchers (Wheeler et al., 2003, Tarantino and De Col, 2008). The average skeleton stress σ^* is defined as:

$$\sigma^* = \sigma - u_a + S_r s \quad (2.3)$$

Jommi (2000) argued that S_r should not be considered as a constitutive parameter as it represents the volume fraction. Another modified stress which represent the matric suction of the soil was also given by Houlsby (1997) and is referred to as modified suction s^* defined as:

$$s^* = n_i s \quad (2.4)$$

where n_i is the instantaneous porosity of the soil specimen. A similar approach was adopted by Alonso et al. (2013) where the behaviour of unsaturated soils was modelled by using constitutive stress $\sigma^{*'}$ and effective suction s' . Constitutive stress and effective suction are defined as:

$$\sigma^{*'} = \sigma - u_a + s' \quad (2.5a)$$

$$s' \equiv S_r' s \quad (2.5b)$$

$$S_r' = \begin{cases} \frac{S_r - \xi_m}{1 - \xi_m}; & S_r > \xi_m \\ 0 & S_r \leq \xi_m \end{cases} \quad (2.5c)$$

where ξ_m is e_m/e and e_m is the microstructural void ratio. Despite having slightly different forms, the concept behind the previously mentioned modified stresses remains the same. Gens et al. (2006) defined a general form for constitutive stress and modified suction, respectively, as follows:

$$\sigma - u_a + \mu_1(s, \dots) \quad (2.5d)$$

$$\mu_2(s, \dots) \quad (2.5e)$$

The forms of constitutive stress and modified suction depend on the forms of μ_1 and μ_2 which can be a function of s and other soil properties. For example, when $\mu_1 = 0$, Equation (2.5d) becomes net normal stress. When μ_1 is a function of s and S_r , i.e., $\mu_1(S_r, s)$, it can be the Bishop's effective stress or the so called average skeleton stress defined by Jommi (2000).

When μ_2 is only a function of s , then it becomes matric suction. When μ_2 is a function of s and n , i.e., $\mu_2(s, n)$, it can be the modified suction as defined by Houlsby (1997). When μ_2 is a function of s and S_r $\mu_2(s, S_r)$, it can be the effective suction as defined by Alonso et al. (2013)

2.2.4. Water content approach

Matric suction plays a central role as a stress-state variable and is commonly used to represent the unsaturated condition. Despite its wide acceptance, the use of matric suction does not run from criticisms. Several criticisms on the use of matric suction are given by Baker and Frydman (2009) such as:

1. At equilibrium, it is the water potential rather than the water pressure which is equal at all points in the system. Thus, every apparatus which are used to measure the pore-water pressure is actually measuring water potential. de Marsily (1986) described this water potential as an energy which is required to extract water from the soil and therefore is not the same as negative pore-water pressure which is exerted by the

water meniscus on the soil skeleton. The generated tension on the soil skeleton is unlikely to reach the water potential unless the soil remains saturated and thus remains a continuum medium (Lu and Likos, 2006).

2. The axis-translation technique which is widely used to investigate unsaturated soils hinder the cavitation in an unsaturated soil specimen which is unnatural as in the field condition, the air pressure is not artificially elevated (Burland and Ridley, 1966) and can only be justified if both the water and capillary are rigid (Marinho et al., 2005). The effect of cavitation on the unsaturated soil specimen is unclear but some experiments using osmotic suction and high-capacity tensiometer (HCT) gave different results compared to experiments using the axis-translation technique (Zur, 1966, Williams and Shaykewich, 1969, Ng et al., 2007).
3. At lower potential (0 to 100 kPa), the water is characterized as adsorbed which does not have stress characteristic (Lu and Likos, 2004). Thus, it is doubtful that the water potential in thin water films can be used to define the macroscopic variable.
4. Several authors (Koorevaar et al., 1983, Gray and Hassanizadeh, 1991, Iwata et al., 1995, Lu and Likos, 2004) suggested that water tension cannot be larger than 100 kPa in the soils.

Kodikara (2012) also mentioned that the ability to predict the behaviour of unsaturated soil is far from satisfactory due to the inability in obtaining accurate measurement of suction especially in the field. Moreover, the range of possible matric suction that can be measured is limited by the devices and requires an exhaustive time to wait for the equilibrium which then limits the ability of the devices to give rapid measurement of the change in pore-water pressure except by using HCT (Ridley and Burland, 1993, Guan and Fredlund, 1997, Ridley and Burland, 1999, Tarantino and Mongiovi, 2002, He et al., 2006). However, the use of HCT is limited to laboratory tests. Some examples of devices for suction measurement are given in Table 2-1.

Table 2-1 Devices for suction measurement (updated from Ridley and Burland, 1933)

Devices	Suction	Direct/indirect	Range (kPa)	Equilibrium time
Desiccator	Total	Indirect	10^3 - 10^6	Months
Psychrometer	Total	Indirect	300-7000	Months
Filter paper	Total	Indirect	1000-30000	Weeks
Chilled-mirror dew-point	Total	Indirect	$100 - 10^6$	minutes
Porous block	Matric	Indirect	30-3000	Weeks
Thermal block	Matric	Indirect	0-175	Days
Suction plate	Matric	Direct	0-90	Hours
Tensiometer	Matric	Direct	0-90	Hours
High-capacity tensiometer	Matric	Direct	0-1500	Minutes
Pressure plate	Matric	Direct	0-5000	Hours
Osmotic tensiometer	Matric	Direct	0-1500	Days

A number of researchers (Tuffour, 1984, Rahardjo et al., 2004, Lee et al., 2005, Lu and Likos, 2006, Gould et al., 2011, Kodikara, 2012) used hydraulic properties i.e. water content of the soil to replace the matric suction variable in describing the behaviour of unsaturated soils. Gravimetric water content is preferred so that the laboratory testing procedure can be simplified as the ability to measure the water content in the laboratory and in situ i.e. oven drying method and time-domain reflectometry, respectively, far exceeds the ability of measuring suction in the laboratory and in-situ i.e. using HCT and in-situ tensiometer (Lu and Likos, 2006). Moreover, Fleureau et al. (2002) showed that relationship between void ratio and water content (also degree of saturation) does not display hysteresis between wetting and drying path, in contrast with relationship between matric suction and void ratio (also water content and degree of saturation) which displays significant hysteresis between drying and wetting paths. Such behaviour opens an opportunity to simplify the volume change analysis of unsaturated soils due to the change in water content. Lu (2008) justified this approach by stating that the state variable is any variable that has a profound effect on a state of interest (Baker and Frydman, 2009).

In this approach, constant water content test is commonly used. Constant water content test can be done much faster as it does not require constant matric suction condition to be maintained. Thus, adopting this approach requires less resources, is faster and the test can be performed for any degree of saturation as no matric suction measurement is required.

2.3. Soil-water characteristic curve (SWCC)

2.3.1. Introduction

The analysis of unsaturated soils often requires the measurement of matric suction. However, the distribution of matric suction especially in the field is non-uniform due to the complex mechanism of evaporation and infiltration. On the other hand, measuring matric suction often requires long test duration which makes it impractical to be performed for a very wide variation of matric suction in the field.

To alleviate the problem of long unsaturated soil test duration in geotechnical engineering practice, the soil-water characteristic curve (SWCC) has been heavily utilized to determine unsaturated soil properties. The SWCC is the relationship between the soil-water content (by mass or volume) and the soil-water matric potential (Leong and Rahardjo, 1997a). One key parameter of the SWCC is the air-entry value (AEV) which demarcates the change from boundary effect zone to transition effect zone as shown in Figure 2-2. The AEV is defined as the matric suction where air starts to enter the largest pores in the soil (Fredlund and Xing, 1994). The AEV appears in many equations used to estimate unsaturated soil properties such as shear strength (Bao et al., 1998, Khalili and Khabbaz, 1998, Rassam and Williams, 1999, Rassam and Cook, 2002, Tekinsoy et al., 2004, Xu, 2004, Lee et al., 2005, Goh et al., 2010) and permeability function (Rijtema, 1965, Philip, 1986, Hunt, 2004, Mbonimpa et al., 2006, Watabe et al., 2006).

In Figure 2-2, the SWCC is divided into three linear segments which are initial drying line, virgin drying/desaturation line and residual line. In Figure 2-2, the ordinate axis labelled “water

content” can be either gravimetric water content w , volumetric water content θ or degree of saturation S . Fredlund et al. (2001) suggested that the same information in the SWCC is conveyed regardless of the term used to describe water content provided the structure of the soil is incompressible. If the soil undergoes volume change as a result of an increase in soil suction, the AEV should be obtained from the SWCC where the ordinate axis is degree of saturation i.e. SWCC-S (Fredlund et al., 2011).

Figure 2-2 SWCC and its properties (adopted from Kohgo, 2003)

In agriculture-related disciplines, volumetric water content is commonly used to plot SWCC. Volumetric water content is defined as the volume of water in the soil referenced to the instantaneous total volume of the soil. However, it is quite common to assume that volume change is negligible to determine volumetric water content using the original volume of the soil. If this is the case, both SWCCs, where the ordinate axis is gravimetric water content (SWCC- w) and where the ordinate axis is volumetric water content (SWCC- θ) or degree of saturation (SWCC- S), show similar information. The use of SWCC- θ is ambiguous as it may be plotted with volume of water in the soil referenced to initial total volume or instantaneous total volume of the soil. In geotechnical engineering, gravimetric water content is commonly used to describe the amount of water in the soil and it should be used to plot the soil-water characteristic curve when volume measurements have not been made (Fredlund et al. 2001).

It is recognized that soil-water characteristic curves (SWCC) can be bimodal due to the presence of bimodal pore-size distribution (Zhang and Chen, 2005, Li, 2009, Satyanaga et al., 2013, Li et al., 2014) as shown in Figure 2-3. Thus, it is important to recognize whether the soil has unimodal or bimodal SWCC.

a) Unimodal SWCC

b) Bimodal SWCC

Figure 2-3 Unimodal and bimodal SWCCs and their parameters

2.3.2. Unimodal SWCC equation

A number of single continuous equations were proposed for unimodal and bimodal SWCCs and its parameters were commonly determined by an optimization method where the SWCC data are curve fitted with a SWCC equation and the error is minimized by iterating the parameters. Leong and Rahardjo (1997b) have reviewed a number of unimodal SWCC equations and have found that van Genuchten (1980) and Fredlund and Xing (1994) equations give the best fit to unimodal SWCC. Fredlund and Xing (1994) equation in terms of gravimetric water content w is given as follows:

$$w = w_s \frac{C(s)}{\left\{ \ln \left[\exp(1) + \left(\frac{s}{a_f} \right) \right]^{n_f} \right\}^{m_f}} \quad (2.6a)$$

$$C(s) = 1 - \frac{\ln \left(1 + \frac{s}{\Psi_r} \right)}{\ln \left(1 + \frac{10^6}{\Psi_r} \right)} \quad (2.6b)$$

where w_s is the saturated gravimetric water content, s is the matric suction, $C(s)$ is a correction function, Ψ_r is the residual matric suction, a_f , n_f and m_f is the curve fitting parameter.

The parameters of SWCC equations are usually inter-dependent in controlling the shape of the curve (Gitirana and Fredlund, 2004). Thus, different combinations of parameters may produce the same SWCC. For example, Leong and Rahardjo (1997b) showed that the parameter Ψ_r in $C(s)$ is not the residual matric suction as three different values of Ψ_r (3000 kPa, 300 kPa and 30 kPa) may give the same residual matric suction while changing the initial part of SWCC by using the same values of $a_f = 300$ kPa, $n_f = 10$ and $m_f = 0.5$ as shown in Figure 2-4. Such interdependency is disadvantageous as correlation of the parameter with other soil properties becomes inconsistent. To overcome this problem, Zhai and Rahardjo (2012) showed that more consistent parameters in Fredlund and Xing (1994) equation can be obtained mathematically

while Chin et al. (2010) limited the non-uniqueness of the parameters in Fredlund and Xing (1994) equation for unimodal SWCCs by using a single bounded variable χ in estimating the parameters. Another solution is to directly incorporate a more consistent or graphically obtainable parameters into the equation such as the slope and intersection point between linear segments of the SWCC (Pham and Fredlund, 2008, Gould et al., 2012). Equations which used these parameters are presented in Table 2-2.

2.3.3. Bimodal SWCC equation

A number of researchers have proposed bimodal SWCC equations. These bimodal SWCC equations can be categorized into three approaches of development. The first approach in developing a bimodal SWCC equation is the piecewise approach (Smettem and Kirkby, 1990, Wilson et al., 1992, Burger and Shackelford, 2001) where the bimodal SWCC is separated into two unimodal SWCCs corresponding to the macropores and micropores in the soil where the merging point or “junction” is arbitrarily determined (Burger and Shackelford, 2001).

Each unimodal curve is then fitted separately. The determination of the merging point affects the value of the parameters as different merging points produce different SWCC curves and thus the parameters are highly non-unique. The advantage of this approach is that the parameters corresponding to the macropores and micropores can be optimized separately which makes parameter optimization easier and faster. This approach can be mathematically written as:

$$w(s) = \begin{cases} w_1(s) & s \leq s_m \\ w_2(s) & s > s_m \end{cases} \quad (2.7)$$

where s_m is the matric suction at the merging point, $w_1(s)$ is the SWCC equation corresponding to the macropores while $w_2(s)$ is the SWCC equation corresponding to the micropores.

Figure 2-4 Parametric studies on parameter Ψ_r by using $a = 300$ kPa, $n = 10$ kPa, $m = 0.5$ kPa from Leong and Rahardjo (1997b)

Table 2-2 Unimodal SWCC equations that mainly used linear parameters

Reference	Equation	Input parameter
Pham and Fredlund (2008)	$w = S_2 f_2 + S_3 + m_3 \log \left(\frac{10^6}{s} \right)$ $f_i(s, s_i, c_{pi}) = f_i = \frac{s_i^{c_{pi}}}{s_i^{c_{pi}} + s^{c_{pi}}}; g_i(s, s_i, c_{pi}) = g_i = 1 - f_i(s, s_i, c_{pi})$ $f'_i(s, s_i, c_{pi}, m_{i-1}, m_i) = \frac{df_i(s, s_i, c_{pi})}{\log s} \alpha = f'_i = f_i g_i (m_{i-1} - m_i) \frac{\ln(10)}{2c_{pi}}$ $S_i = f_i (m_i - m_{i-1}) \log \left(\frac{s}{s_2} \right) + f'_i$ <p>When it is desired force $w = 0$ at $s = 10^6$ kPa</p> $m_3 = \frac{w_{sat} + (m_2 - m_1) \log s_2 - m_2 \log s_3}{\log \frac{10^6}{s_3}}$	<p>General notation: s = matric suction s_1 = matric suction at w_{sat} For Unimodal SWCC: s_2 = matric suction at the intersection point of segment 1 and 2 s_3 = matric suction at the intersection point of segment 2 and 3 s_{min} = matric suction at zero water content w_{sat} = saturated gravimetric water content</p> <p>m_1 = slope of segment 1 m_2 = slope of segment 2 m_3 = slope of segment 3</p> <p>Additional parameter for Pham and Fredlund (2008): c_{p2} = parameter that controls the curvature at s_2 c_{p3} = parameter that controls the curvature at s_3</p>
Gould et al. (2012)	$w = w_{sat} + S_2 + S_3 + \frac{m_1 + m_3}{2} \log \frac{s}{s_{min}}$ $f(x, y, c_{Gi}) = c_{Gi} \log \frac{x}{y} \tan^{-1} \left(c_{Gi} \log \frac{x}{y} \right) - \frac{1}{2} \ln \left[1 + \left(c_{Gi} \log \frac{x}{y} \right)^2 \right]$ $S_i = -\frac{m_{i-1} - m_i}{c_{Gi} \pi} [f(s, s_i, c_{Gi}) - f(s_{min}, s_i, c_{Gi})]$	<p>Additional parameters for Gould et al. (2012): c_{G2} = parameter that control the curvature at s_2 c_{G3} = parameter that control the curvature at s_3</p>

The second approach in developing bimodal SWCC equation is the fraction of total volume approach (Othmer et al., 1991, Ross and Smettem, 1993, Durner, 1994, Mallants et al., 1997, Zhang and Chen, 2005). In the second approach, the total porosity which is represented by its saturated water content is separated into the fractions belonging to the macropores and micropores. Under this approach, a continuous form is possible but the optimization of all parameters have to be done at the same time which escalates the problem associated with non-uniqueness and optimization in determining the parameter. This approach can be mathematically written as:

$$w(s) = w_M f_1(s) + w_m f_2(s) \quad (2.8a)$$

$$W_{sat} = W_M + W_m \quad (2.8b)$$

where w_M is the saturated water content of the macropores, w_m is the saturated water content of the micropores, $f_1(s)$ and $f_2(s)$ are the dimensionless SWCC functions corresponding to the macropores and micropores, respectively.

The third approach in developing a bimodal SWCC equation is the unique parameter approach (Gitirana and Fredlund, 2004, Li, 2009, Satyanaga et al., 2013, Li et al., 2014). Bimodal SWCC equations have more parameters compared to unimodal SWCC equations which exacerbates the problems associated with the non-uniqueness and optimization of the parameters. In the third approach, the non-uniqueness of parameters are reduced by incorporating parameters which can be determined graphically. Using such an approach, optimization becomes easier as there are less parameters to determine. For example, Gitirana and Fredlund (2004) equation used matric suction and water content at the intersection points and therefore only one parameter needs to be determined. Li (2009) and Li et al. (2014) equations use the matric suction and water content at each intersection point and require eight additional parameters to be determined. Satyanaga et al. (2013) equation uses the matric suction and water content

at each intersection point, the matric suction at the inflection points and requires two additional parameters to be determined. The third approach is more attractive than the first two approaches as less parameters need to be determined (Gitirana and Fredlund, 2004, Pham and Fredlund, 2008, Gould et al., 2012). Examples of equation which used the unique parameters approach for bimodal SWCC are given in Table 2-3.

Table 2-3 Bimodal SWCC equations that used unique parameters approach

Reference	Equation	Input parameter
Gitirana and Fredlund (2004)	$w = f_{n+1} + \sum_{i=2}^n \frac{f_i - f_{i+1}}{1 + \left(\frac{s}{\sqrt{s_i s_{i+1}}} \right)^{d_i}}$ $f_i = \frac{\tan \theta_i (1 + r_i^2) \ln(s/s_i)}{(1 - r_i^2 \tan^2 \theta_i)} +$ $(-1)^{i-1} \times \frac{(1 + \tan^2 \theta_i)}{(1 - r_i^2 \tan^2 \theta_i)} \sqrt{r_i^2 \ln^2(s/s_i) + \frac{a^2 (1 - r_i^2 \tan^2 \theta_i)}{1 + \tan^2 \theta_i}} + w_i$ $d_i = 2 \exp \left[\frac{1}{\ln(s_{i+1}/s_i)} \right]; r_i = \tan \left(\frac{\lambda_{i-1} - \lambda_i}{2} \right); \theta_i = -\frac{\lambda_{i-1} + \lambda_i}{2}$ $\lambda_i = \arctan \left[\frac{(w_i - w_{i+1})}{\ln(s_{i+1}/s_i)} \right]$ <p>Unimodal equation one bending point; $i = 2$ For two bending point ($n = 2$) and bimodal SWCC ($n = 4$):</p>	<p>General notation: AEV = air entry value Ψ_r = curve fitting parameter which represent residual matric suction s_{m-i} = matric suction at inflection point of segment i s = matric suction</p> <p>s_1 = Matric suction at w_{sat} For Bimodal SWCC: s_2 = matric suction at the intersection point of segment 1 and 2 s_3 = matric suction at the intersection point of segment 2 and 3 s_4 = matric suction at the intersection point of segment 3 and 4 s_5 = matric suction at the intersection point of segment 4 and 5 $s_6 = s_{min} = s$ at zero water content</p> <p>$w_1 = w_{sat}$ $w_2 = w$ at s_2 $w_3 = w$ at s_3 $w_4 = w$ at s_4 $w_5 = w$ at s_5 $w_6 = 0$</p>
Satyanaga et al. (2013)	$w = \left\langle w_r + \sum_{i=2}^n (w_{i-1} - w_{i+1}) \left\{ \operatorname{erf} \left[\frac{\ln \left(\frac{s_i - s}{s_i - s_{m-i}} \right)}{S_{si}} \right] \right\} \right\rangle \left[1 - \frac{\ln \left(1 + \frac{s}{\Psi_r} \right)}{\ln \left(1 + \frac{10^6}{\Psi_r} \right)} \right]$ $\operatorname{erf} = 1 - \int_{-\infty}^x \frac{1}{\sqrt{2\pi}} \exp \left(-\frac{x^2}{2} \right) dx$ <p>For unimodal SWCC $n = 2$ while for bimodal SWCC $n = 3$</p>	<p>m_1 = slope of segment 1 m_2 = slope of segment 2 m_3 = slope of segment 3 m_4 = slope of segment 4 m_5 = slope of segment 5</p> <p>a = Additional curve fitting parameter of Gitirana and Fredlund (2004)</p> <p>S_{si} = Additional curve fitting parameter of Satyanaga et al. (2013)</p> <p>$n_i, l_i, m_{L_i}, \lambda_i$ = curve fitting parameters of Li (2009) equation</p>
Li (2009) or Li et al. (2014)	$w(s) = (w_{sat} - w_3) \frac{\sqrt{s_2 s_3}^{n_1 / \log(s_3/s_2)}}{s^{n_1 / \log(s_3/s_2)} + \sqrt{s_2 s_3}^{n_1 / \log(s_3/s_2)}} + (w_{sat} - w_3) / \lambda_1 \frac{(l_1 s_3)^{m_{L_1}}}{s^{m_{L_1}} + (l_1 s_3)^{m_{L_1}}}$ $+ (w_4 - w_5) \frac{\sqrt{s_4 s_5}^{n_2 / \log(s_5/s_4)}}{s^{n_2 / \log(s_5/s_4)} + \sqrt{s_4 s_5}^{n_2 / \log(s_5/s_4)}} + (w_4 - w_5) / \lambda_2 \frac{(l_2 s_5)^{m_{L_2}}}{s^{m_{L_2}} + (l_2 s_5)^{m_{L_2}}}$ <p>Additional constrain: $(w_{sat} - w_3)(1 + \lambda_1) + (w_4 - w_5)(1 + \lambda_2) = w_{sat}$</p>	

2.3.4. Effect of density on the SWCC

The soil density which can be represented by the saturated water content w_{sat} , void ratio e , or relative density D_r affects the SWCC. Therefore, it is not suitable to use a single SWCC for soils with various densities. The effect of soil density on the SWCC also depends on whether the SWCC is plotted based on gravimetric water content (Jotisankasa et al., 2007, Salager et al., 2010, Salager et al., 2011) or degree of saturation (Huang et al., 1998, Gallipoli et al., 2003, Jotisankasa et al., 2007, Nuth and Laloui, 2008, Tarantino, 2009, Sheng and Zhou, 2011, Zhou et al., 2014). Salager et al. (2010) found that when the SWCC is plotted with gravimetric water content (SWCC-w), a decrease in initial void ratio causes a decrease in saturated gravimetric water content and therefore affects the initial portion of the SWCC w. However, the SWCC-w of the same soil at various densities merges into the virgin drying line and residual line at high matric suction as shown in Figure 2-5a. Similar observation was also observed on the wetting SWCC where SWCCs with different water contents at zero matric suction merge into a unique curve at high matric suction (Romero et al., 2011) as shown in Figure 2-5b.

When SWCC is plotted with degree of saturation (SWCC-S), a decrease in the void ratio causes the AEV to increase and therefore shifts the SWCC-S to the right as it requires a higher matric suction to reach the same degree of saturation (Huang et al., 1998, Gallipoli et al., 2003, Tarantino and Tombolato, 2005, Jotisankasa et al., 2007, Nuth and Laloui, 2008, Tarantino, 2009, Sheng and Zhou, 2011, Zhou et al., 2014). However, the change in matric suction which is required to achieve the same degree of saturation is difficult to quantify as degree of saturation is affected by the water content and the void ratio where both water content and void ratio change with matric suction independently and is unique for different soils.

a) Drying SWCCs of clayey silty sand specimens which were compacted into different initial void ratios (from Salager et al., 2010)

b) Wetting SWCCs of three materials compacted on the dry side of optimum (from Romero et al., 2011)

Figure 2-5 Effects of different initial void ratios on drying and wetting SWCCs

As a result, it requires several additional tests and mathematical assumptions to model the effect of density on SWCC-S making it more difficult to model than the SWCC-w. Several

equations have been proposed to account for the effect of void ratio on the SWCC-w and SWCC-S as shown in Table 2-4. In Table 2-4, Zhou et al. (2014) equation use initial porosity n_0 to directly describe the effect of density on the SWCC-S and therefore has a significant advantage as it requires only one test to obtain all of the parameters. Tarantino (2009) proposed an equation to determine the SWCC-S where all of the parameters can be determined from one test as well but it requires the instantaneous void ratio which sometimes is not available in the SWCC-w measurement. Gallipoli et al. (2003), Zhou et al. (2012c) and Salager et al. (2010) equations can be used to take into account the effect of density on the SWCC but it requires additional test and thus can be considered as less advantageous compare to Zhou et al. (2014) and Tarantino (2009) equations.

2.3.5. Hysteresis on SWCC

Hysteresis is a condition where the drying path SWCC (SWCC that is constructed by increasing matric suction) is different from wetting path SWCC (SWCC that is constructed by decreasing matric suction). Wetting path SWCC has less water content compare to the drying path SWCC (Pham et al., 2005). There are several reasons for this phenomenon (Klausner, 1991):

1. Irregularities in the cross-sections of the void passages or the “ink-bottle” effect (Haines, 1930).
2. The contact angle being greater in an advancing meniscus than in a receding meniscus.
3. Entrapped air, which has a different volume when the soil suction is increasing or decreasing.
4. Thixotropic regain or aging due to the wetting and drying history of the soil.

Considering that the complete set of SWCCs with hysteresis is time consuming to obtain experimentally, SWCC is usually assumed to be non-hysteretic (Pham et al., 2005).

Table 2-4 SWCC equations which account for density

Reference	Equation	Parameters
Gallipoli et al. (2003) – modified van Genuchten (1980) Equation	$S_r = \left\{ \frac{1}{1 + [s\phi(e)]^{n_g}} \right\}^{m_g}$ <p>ϕ and Ψ requires additional SWCCs to be determined</p>	w = gravimetric water content w_{sat} = saturated gravimetric water content S_r = degree of saturation n_0 = initial porosity e = void ratio e_0 = initial void ratio
Tarantino (2009) - van Genuchten (1980) Equation	$S_r = \left\{ 1 + \left[\left(\frac{e}{A_t} \right)^{1/B_t} s \right]^{n_g} \right\}^{-B_t/n_g}$	s = matric suction s_r = residual matric suction s_p = matric suction of virgin drying line at 0 water content m_2 = slope of virgin drying line a_f, n_f, m_f = Fredlund and Xing (1994) curve fitting parameters n_g, m_g = van Genuchten (1980) curve fitting parameters A_s, B_s = Salager et al. (2010) curve fitting parameters ϕ, ψ = Gallipoli et al. (2003) curve fitting parameters A_t, B_t = Tarantino (2009) curve fitting parameters
Salager et al. (2010) – modified Fredlund and Xing (1994) equation	$w = \frac{w_{sat}}{\left[\ln[\exp(1) + (s/a_f)^{n_f}] \right]^{m_f}} \left\{ 1 - \frac{\ln[1 + (s/s_r)]}{\ln[1 + (10^6/s_r)]} \right\}$ $m_f = 3.67 \ln \left\langle \frac{w_{sat}}{m_2 \ln \left(\frac{s_p}{a_f} \right)} \left\{ 1 - \frac{\ln \left[1 + \left(\frac{a_f}{s_r} \right) \right]}{\ln \left[1 + \left(\frac{10^6}{s_r} \right) \right]} \right\} \right\rangle$ $n_f = 3.72 \frac{1.31^{m_f+1}}{m_f \left\{ 1 - \frac{\ln \left[1 + \left(\frac{a_f}{s_r} \right) \right]}{\ln \left[1 + \left(\frac{10^6}{s_r} \right) \right]} \right\}} \left\{ \frac{m_2}{w_{sat}} - \frac{a_f}{1.31^{m_f} (a_f + s_r) \ln \left[1 + \left(\frac{10^6}{s_r} \right) \right]} \right\}$ $a_f = A_s e_0^{B_s}$ <p>A_s and B_s require additional SWCCs to be determined</p>	s = matric suction s_r = residual matric suction s_p = matric suction of virgin drying line at 0 water content m_2 = slope of virgin drying line a_f, n_f, m_f = Fredlund and Xing (1994) curve fitting parameters n_g, m_g = van Genuchten (1980) curve fitting parameters A_s, B_s = Salager et al. (2010) curve fitting parameters ϕ, ψ = Gallipoli et al. (2003) curve fitting parameters A_t, B_t = Tarantino (2009) curve fitting parameters
Zhou et al. (2012c)	$S_e = - \int \frac{S_e}{e_i} (1 - S_e)^{\zeta} de_i$ <p>S_e is determined from other SWCC equation. Solution based on Simpson's rule is given as:</p> $\ln \frac{e_f}{e_i} \approx \frac{S_{e,f} - S_{e,0}}{6} [f(S_{e,0}) + 4f\left(\frac{S_{e,0} + S_{e,f}}{2}\right) + f(S_{e,f})]$ $f(x) = \frac{1}{x(1-x)^{\zeta}}$ <p>ζ requires additional SWCCs to be determined</p>	S_e is determined from other SWCC equation. Solution based on Simpson's rule is given as: ζ = Zhou et al. (2012c) curve fitting parameters
Zhou et al. (2014) – modified Fredlund and Xing (1994) equation	$S_r = \frac{1}{\left[\ln[\exp(1) + (s.n_0/a_f)^{n_0 n_f}] \right]^{n_0 m_f}} \left\{ 1 - \frac{\ln[1 + (s/s_r)]}{\ln[1 + (10^6/s_r)]} \right\}$ $w = \frac{e}{G_s \left[\ln[\exp(1) + (s.n_0/a_f)^{n_0 n_f}] \right]^{n_0 m_f}} \left\{ 1 - \frac{\ln[1 + (s/s_r)]}{\ln[1 + (10^6/s_r)]} \right\}$	ζ = Zhou et al. (2012c) curve fitting parameters

It is noted that during the compression under constant water content, the degree of saturation increases due to the decrease in void ratio. This type of wetting is considered as mechanical wetting. As water content remains constant, the hysteresis in the matric suction is due to the re-distribution of pore-water. An example of such a case is given by Tarantino and De Col (2008) and is shown in Figure 2-6. In Figure 2-6, a soil specimen was loaded up to point 8 with vertical stress around 900 kPa, preconsolidation void ratio e_c around 1.05 and matric suction around 730 kPa. The soil specimen was then unloaded to point 9 and then loaded back to point 8 with vertical stress around 900 kPa.

The value of e_c does not change but the matric suction changes to around 700 kPa. In constant water content compression, there should not be any ink bottle effect as there is no change in water content. However, when the soil was compressed, there is a possibility that the water moves between the pores and therefore may affect the amount of entrapped air and also the water-air formation inside the soil pores. This observation shows that while the value of water content and e_c is unaffected by the constant water content loading path, it slightly affects the matric suction of the soil. Such behaviour shows the ease in using water content instead of matric suction.

Figure 2-6 Void ratio and suction changes on an unloading-reloading cycle of specimen with $w = 25.4\%$ (from Tarantino and De Col, 2008)

2.4. Shrinkage curve

2.4.1. Introduction

Unsaturated soil may change in volume due to mechanical change (change in net normal stress) or due to change in water content. Shrinkage curve is commonly used to describe the volume change of soils due to drying and is usually plotted as void ratio versus water content.

The water content on the shrinkage curve can be either gravimetric water content (Fredlund et al., 2002) or moisture ratio (Cornelis et al., 2006a). The shape of shrinkage curve differs for different soil types and soil fabrics. Soil-shrinkage characteristic curve (SSCC) conveys similar information except that the x axis is matric suction instead of water content. Thus, SSCC is more difficult to construct than shrinkage curve as it requires matric suction measurement.

Peng and Horn (2013) proposed six types of shrinkage curves (A, B, C, D, E and F) based on the existence of the four shrinkage phases (i.e. zero shrinkage, residual shrinkage, proportional shrinkage and structural shrinkage) as shown in Figure 2-7. The difference in the types of the shrinkage curves relate to the existence of the four shrinkage phases as summarised in Table 2-5.

The shrinkage limit, macropores shrinkage limit, and saturated moisture ratio/water content separate each phase of the shrinkage curve. However, there are some differences regarding the definitions between disciplines. In geotechnical engineering, the point used to separate different phases is usually given by the intersection point between adjacent linear segments while in soil science, the point of convergence between the curvilinear part and the linear segment is used.

For example, Fredlund et al. (2002) and Holtz et al. (2011) defined the shrinkage limit as the intersection point between the loading line and the zero shrinkage line (shown as w_2 in Figure 2-7c).

a) Four-linear segments shrinkage curve (types A and B)

b) Three-linear segments shrinkage curve (types C and D)

c) Two-linear segments shrinkage curve (types E and F)

Figure 2-7 Shrinkage curve phases and boundaries

This definition may not give the true shrinkage limit (SL) as further decrease of water content will cause additional shrinkage. This point will be known as apparent shrinkage limit (SL') in this thesis.

On the other hand, Cornelis et al. (2006) defined shrinkage limit as the point of convergence between the curvilinear segment of the residual shrinkage and the zero shrinkage line where there is no further decrease in void ratio below this water content (shown as w_2 in Figure 2-7c). This point is the true shrinkage limit (SL) and is commonly referred to as the shrinkage limit in soil science. The true shrinkage limit is difficult to determine as it requires continuous measurement. In Figure 2-7, k_i is a parameter which represents the curvature between segments i and $i-1$.

Table 2-5 Classification of shrinkage curves based on the four shrinkage phases

Type	Structural shrinkage	Proportional shrinkage	Residual shrinkage	Zero shrinkage
A	Yes	Yes	Yes	Yes
B	Yes	Yes	Yes	No
C	Yes	Yes	No	No
D	No	Yes	No	No
E	No	Yes	Yes	No
F	No	Yes	Yes	Yes

2.4.2. Equation to represent shrinkage curve

The shrinkage curve can be modelled as a combination of linear and curvilinear segments (Braudeau et al., 1999). Using such an approach, a general shrinkage curve consists of n -linear segments where each segment i with slope m_i represents a different phase of the shrinkage curve (Figure 2-7) and the curvilinear segment between two linear segments being a curve with curvature k_i . Types A and B shrinkage curves can be considered as a 4-linear segments shrinkage curve (Figure 2-7a), types C and D can be considered as a 3-linear segments shrinkage

curve (Figure 2-7b), while types E and F can be considered as a 2-linear segments shrinkage curve (Figure 2-7c).

Parameters in the shrinkage curve equation can be categorised into graphically obtainable parameters and non-graphically obtainable parameters. Graphically obtainable parameter is a parameter which is obtained directly from the curve without using an optimization algorithm (e.g., the slope or intercept of the curve) whereas a non-graphically obtainable parameter is a parameter which has no graphical meaning and thus need to use an optimization algorithm. When parameters cannot be obtained graphically, it usually affects several properties of the curve (i.e. slopes, intersection points and the curvature) and thus, does not have a single unique value. On the other hand, graphically obtainable parameters have a single unique value as it can only be determined from the curve. Several advantages in using graphically obtainable parameters in a shrinkage curve equation are:

1. Graphically obtainable parameter can be directly obtained from the shrinkage curve data. Thus, it does not require any optimization algorithm.
2. The values of graphically obtainable parameters do not depend on the form of the equation and any error minimization criterion in determining their values.
3. Graphically obtainable parameter is unique, and therefore can be investigated separately, for example in correlating the parameter with other soil properties.

Although graphically obtainable parameters of the shrinkage curve can be easily obtained, it is not so easy to describe all types of shrinkage curve using a single continuous equation with graphically obtainable parameters. It is more common to use linear equation to describe the linear segments and another equation to describe the curvilinear segments of the shrinkage curve. Some of the extant shrinkage curve equations are summarised in Table 2-6.

Table 2-6 Equations for shrinkage curve

Reference	Equation	Parameters
Fredlund et al. (2002)	$e = e_{\min} \left[\frac{w^C}{SL'^C} + 1 \right]^{1/C}$ $\frac{e_{\min}}{SL'} = \frac{G_s}{S_0}$	$e_{\min}$ = minimum void ratio w = gravimetric water content SL' = apparent shrinkage limit G_s = specific gravity S_0 = initial degree of saturation c = the curvature of the shrinkage curve
Giraldez et al. (1983)	$e = 0.7429m_2v_{AE} + 0.23 \frac{m_2}{v_{AE}} v^2 + 0.0267 \frac{m_2}{v_{AE}^2} v^3$	v_{AE} = moisture ratio at air entry value v = moisture ratio m_2 = slope of the loading line
Kim et al. (1992)	$e = e_{\min} \exp(-\beta v) + w m_2$	β = slope parameter depending on AEV
Chertkov (2000, 2003)	$e = \begin{cases} e_{\min}; & v < v_{SL} \\ e_{\min} + \mu(v - v_{SL})^2 \frac{\rho_w^2}{\rho_s}; & v_{SL} \leq v \leq v_{AE} \\ v; & v > v_{AE} \end{cases}$	μ = model parameter v_{SL} = moisture ratio at true shrinkage limit
Cornelis et al. (2006a) or modified Chertkov (2000,2003) equation (ModC)	$e = \begin{cases} e_{\min}; & v < v_{SL} \\ e_{\min} + \mu(v - v_{SL})^2 \frac{\rho_w^2}{\rho_s}; & v_{SL} \leq v \leq v_{AE} \\ 2\mu(v - v_{SL})v + e_{\min} - \mu(v_{AE} - v_{SL})(v_{AE} + v_{SL}); & v_{AE} < v < v_{LL} \end{cases}$	v_{LL} = moisture ratio at liquid limit
Cornelis et al. (2006b) or modified Groenevelt and Grant (2001,2002) (ModGG)	$e = e_{\min} + \gamma \left[\exp\left(\frac{-\xi}{v^\zeta}\right) \right]$	γ, ζ, ξ = model parameter
McGarry and Malafant (1987)	$e = \begin{cases} e_{\min} + \frac{v}{v_{SL'}} (v_{SL'} - e_{\min} + e_n) & 0 < v < v_{SL'} \\ e_n + v & v_{SL'} \leq v \leq v_{SWL} \\ e_s + \frac{v}{v_{SWL}} (v_{SWL} - e_s + e_n) & v_{SWL} < v \leq v_S \end{cases}$	$v_{SL'}$ = moisture ratio at apparent shrinkage limit v_{inf} = moisture ratio at inflection point e_s = saturated void ratio β = model parameter
Gould et al. (2011)	$e = e_{\min} + \frac{e_s}{1 + \exp[-\beta(v - v_{inf})]}$ $e = f(w, a_G) - f(w, b_G) - f(0, a_G) + f(0, b_G) + e_r$ $f(x, y) = -\frac{m}{\phi\pi} \left\langle \phi(x - y) \tan^{-1}[\phi(x - y)] - 0.5 \ln \left\{ 1 + [\phi(x - y)]^2 \right\} \right\rangle$ <p>to account for net normal stress:</p> $a_G = \alpha_G \ln \left(1 + \frac{\sigma}{\sigma_0} \right) + a_0$ $e_r = \alpha_{er} \ln \left(1 + \frac{\sigma}{\sigma_0} \right) + e_{r0}$	$a_0, a_G, b_G, e_r, m, \phi, \alpha_G, \alpha_{er}, e_{r0}$ = model parameters

Chertkov (2000, 2003) used a combination of linear and polynomial equations to describe the shrinkage curve. Kim et al. (1992), Cornelis et al. (2006a) and Braudeau et al. (1999) used a combination of linear and exponential functions to describe the shrinkage curve. Giraldez et al. (1983) and Tariq and Durnford (1993) used a polynomial function to describe the shrinkage curve. Fredlund et al. (2002) and Olsen and Haugen (1998) used a hyperbola function, while McGarry and Malafant (1987) and Peng and Horn (2007) used a logistic function to describe the shrinkage curve. Cornelis et al. (2006a) compared eight shrinkage curve equations and concluded that the McGarry and Malafant (1987), Braudeau et al. (1999), modified Chertkov (2000, 2003) by Cornelis et al. (2006b), modified Groenevelt and Grant (2001, 2002) by Cornelis et al. (2006b) equations performed the best in terms of root mean square error (RMSE), coefficient of correlation (R^2) and the Akaike Information Criterion (AIC). The McGarry and Malafant (1987) equations do not describe the zero shrinkage phase. The original Chertkov (2000, 2003) equations are for clay pastes. Although the modified Groenevelt and Grant (2001, 2002) equation (Cornelis et al., 2006b) uses only one single equation to describe the complete shrinkage curve, it uses non-graphically obtainable parameters.

2.4.3. Hysteresis of the shrinkage curve

Figure 2-8 shows two soils (Jossigny loam and FoCa clay) which were dried from slurry and then wetted. Unlike SWCC and SSCC which have significant hysteresis path between drying and wetting curves (Vecchia and Romero, 2013), the hysteresis between shrinkage and swelling curve can be considered as negligible (Fleureau et al., 2002, Gould et al., 2011). Based on mercury intrusion porosimetry (MIP) carried out by (Romero, 2013), shrinkage and swelling behaviour is predominant in the micropores. Thus, shrinkage and swelling behaviour is a quasi-reversible process in terms of water content. This behaviour makes it possible to use a single shrinkage curve to estimate the actual amount of swelling due to the change in water content provided that the maximum amount of swelling can be determined.

Figure 2-8 Shrinkage and swelling curve (from Fleureau et al., 2002)

2.4.4. Effect of density on the shrinkage curve

Investigation on the effect of density on the shrinkage curve has been carried out by Salager et al. (2010). It was shown that even for soil below its minimum void ratio, the soil continues to shrink and thus changes the minimum void ratio of the unsaturated soil as shown in Figure 2-9. In this study, the upper bound of minimum void ratio is referred as the natural minimum void ratio e_{min-n} .

Figure 2-9 Shrinkage curve under different density (from Salager et al., 2010)

Salager et al. (2010) proposed that the relationship between initial void ratio e_0 and void ratio at different matric suctions $e(s)$ can be assumed to be linear under constant suction condition by using proportionality coefficient $\alpha_s(s)$ which is bound from 0.79 for dry condition to 1 for saturated condition. Thus, by knowing the value of e_0 , it is possible to determine the minimum void ratio $e_{\min}$ which corresponds to the value of $\alpha_s(s) = 0.79$. Gould et al. (2011) equation which is shown in Table 2-6 can also be used to estimate the effect of net normal stress on the shrinkage curve. Gould et al. (2011) equation can be used when the soil is loaded under saturated condition and then dried. However, for the case where the soil is dried first, then loaded and unloaded, the parameters in Gould et al. (2011) equation will change as there is hysteresis in the change in volume due to loading and unloading.

2.5. Compression behaviour of unsaturated soils

2.5.1. Experimental procedures in investigating compression behaviour of unsaturated soils

The compression behaviour of saturated soil is only affected by a single stress-state variable which is effective stress. However, the compression behaviour of unsaturated soils is affected by net normal stress, and also by changes in water content or degree of saturation which is usually represented by matric suction. Therefore, different combinations of both net normal stress and matric suction produce different compression curves.

Thus, in order to simplify the analysis on the compression behaviour of unsaturated soil, it is required to determine on which stress path the compression will take place. The compression behaviour of unsaturated soil is commonly investigated either under constant matric suction or constant water content conditions by using either oedometer or triaxial apparatus. The summary of conclusions regarding volume change of unsaturated soils based on constant water content and constant matric suction test is given in Table 2-7.

The biggest problem in testing unsaturated soil lies in the difficulties in controlling the properties of the specimen (Zhang et al., 2010, Zhang, 2016b). For example, carefully prepared identical specimens may have different preconsolidation pressures although the soil specimens were tested following exactly the same stress path (Wheeler and Sivakumar, 1995, Rampino et al., 2000, Blatz and Graham, 2003, Zhang, 2016b). Thus, it is important to make the experimental procedures to be as simple and as repeatable as possible to gain more confidence with the experimental results.

Table 2-7 Summary of unsaturated soil compression behaviour

Constant variable plane	Comment
Constant water content	<ol style="list-style-type: none"> 1. Difficult to separate the pure compression behaviour from the mechanical collapse behaviour (Alonso et al., 1990). 2. Drying the soils will increase preconsolidation pressure (Jotisankasa, 2005, Jotisankasa et al., 2007, Leong et al., 2013). 3. The unsaturated compression line (UCL) can either merge or not merge with the saturated normal compression line but the slope of the UCL seems to increase (less stiff) rather than to decrease (Jotisankasa, 2005, Burton et al., 2014). 4. Matric suctions at the top and bottom of the specimen may differ by around 5-15% for a thin specimen of about 20 mm (Jotisankasa, 2005). 5. The preconsolidation pressure is not only due to the past maximum compaction stress but also due to the inter-particle bonding which is caused by matric suction (Munoz et al., 2011). 6. The suction-degree of saturation relationship in the unloading-reloading path is reversible. However, the matric suction which is recorded prior to unloading is higher than the matric suction which is recorded at the end of the unloading-reloading cycle (Tarantino and De Col, 2008). 7. Change in matric suction due to the change in net normal stress under constant water content condition becomes less significant as the soil becomes drier (Rahardjo, 1990, Jotisankasa et al., 2007) 8. The slope of unloading-reloading line appears to be independent of matric suction (Munoz et al., 2011).
Constant matric suction	<ol style="list-style-type: none"> 1. An increase in constant suction will increase the preconsolidation pressure (Alonso et al., 1990, Lloret et al., 2003). 2. There are a number of uncertainties in the unsaturated compression line of the unsaturated soils and all of them are supported by experimental observations. The slope of the unsaturated compression line may either reduce with suction (Alonso et al., 1990, Pham, 2005), increase with suction and then merge with the saturated normal compression line (Toll, 1990, Alonso, 1993, Wheeler and Sivakumar, 1995, Honda, 2000, Sivakumar and Wheeler, 2000, Futai et al., 2002, Silva et al., 2002) or unaffected by matric suction (Larson et al., 1980, O'Sullivan, 1992, Smith et al., 1997, Arvidsson and Keller, 2004, Imhoff et al., 2004). 3. When degree of saturation equals to 1, the UCL merge with NCL regardless of the matric suction of the soils (Mun and McCartney, 2015) 4. The slope of unloading-reloading line appears to be independent of matric suction (Alonso et al., 1990, Wheeler and Sivakumar, 1995, Munoz et al., 2011, Hoyos et al., 2012). 5. Soils which are dried and then wetted to the desired matric suction are more susceptible to yield rather than soils which are only dried to the desired matric suction (Sivakumar et al., 2006)

2.5.1.1. Constant matric suction test

Constant matric suction test is frequently used to separate the effect of the change in matric suction from the effect of the change in net normal stress on the unsaturated compression curve. However, there are three issues related to the constant matric suction test:

1. It requires a complicated setup to control the matric suction of the soil.
2. It requires a long duration to complete the test.
3. The idealized stress path and the actual stress path during the test may not be the same.

Matric suction can be controlled by either using axis-translation technique or vapour equilibrium technique. The axis-translation technique is commonly used when the desired matric suction is below 1000 kPa while the vapour equilibrium technique is usually used when the desired matric suction is above 1000 kPa. Thus, it requires a complicated setup to control the matric suction of the soil (high-air entry disk, pressure transducer, air and water pressure controller, special cell) which is expensive compare to the conventional test. Moreover, constant matric suction test requires excessive durations to maintain the constant suction condition. Duration of each loading stage may vary from days to weeks and thus, it may require a few months to finish the test (Cunningham et al., 2003, Kayadelen, 2008, Cui et al., 2010).

Another issue with the constant suction test is in the difference between actual stress path and the idealized stress path as shown in Figure 2-10 (Zhang, 2016a). In order to achieve constant matric suction test, the soil was first loaded following stress path G-1, which then causes the matric suction of the soil specimen to reduce under constant water content condition. After that, the matric suction reverts back to the original matric suction following stress path 1-a. The effect of such testing stress path becomes apparent once the soil specimen has been loaded beyond its apparent preconsolidation pressure. Each time the soil specimen is loaded, a decrease in matric suction causes mechanical wetting which then leads to a collapse settlement. The higher the stress increment, the bigger the amount of collapse settlement. Increasing matric suction will not cause the soil to expand and therefore the unsaturated compression curve will not go back to the UCL under constant matric suction test. Thus, the expected idealized curve is located above the actual test result as collapse settlement

due to mechanical wetting affects the actual unsaturated compression curve. This phenomenon causes difficulties in interpreting the unsaturated compression curve as it actually follows a very complicated stress path instead of a simple constant matric suction stress path. This effect may vary depending on the equipment and procedures which were used to carry out the test and thus may cause inconsistent observation between researchers. Several examples of constant matric suction tests on unsaturated soil are given below.

Figure 2-10 Real stress path in constant suction test (Zhang, 2016a)

Matyas and Radhakrishna (1968) developed a modified triaxial apparatus as shown in Figure 2-11a. An inner perspex cylinder was added into the triaxial cell in order to minimize the error in measuring volume. The annular space between the inner cell and the specimen was filled with mercury to a level above the O-ring seals but below the top of the end platen. The changes in the level of the surface of the mercury were measured with a cathetometer. The suction measurement was done with electrical pressure transducer and axis-translation technique. An electrical lateral strain indicator was developed for the K_0 test. The measurements of the air and water volumes were done separately. Overall volume changes were obtained from independent measurement of lateral and axial deformations. Changes in the volume of pore-air were done only in constant water content test. The initial matric suction of the specimen was measured inside the triaxial test. The application of suction was done by adjusting the

pore-air pressure, pore-water pressure, and confining pressure σ_3 so that the desired matric suction can be achieved. The full saturation was done following two stages. First, small decrement in suction was done by allowing the imbibition of water which was supplied at a controlled pressure through the base of the specimen. Second, after the suction was reduced to zero, the degree of saturation was increased by applying a back pressure at both the top and the bottom of the specimen.

Blatz and Graham (2000) developed a triaxial test with a vapour equilibrium system to generate suction in the soil specimen. Psychrometers of capacity 8 MPa were used to measure the suction of the specimen. The schematic diagram of the new triaxial test with controlled suction is given in Figure 2-11b. The test was performed in the laboratory with temperature variation of $\pm 1^\circ\text{C}$.

The data was monitored using Campbell Scientific CR7 data acquisition and control system. The axial deformation was measured between the top and bottom platens. The suction of the specimen was controlled using a vapour equilibrium system where vapour at a known partial vapour pressure was circulated around the triaxial specimen using a combination of geosynthetic strips and porous stone inside the normal triaxial membrane made of silicon rubber. Psychrometers were installed at two locations. The first location was at the centre of the specimen that was installed during the initial compaction and the second location was at the top of the specimen during installation of the load cap.

Lloret et al. (2003) conducted suction-controlled oedometer tests in CIEMAT (Centro de Investigaciones Energeticas) and UPC (Universitat Politecnica de Catalunya) laboratories as shown in Figure 2-12. The CIEMAT suction-controlled oedometer test was used for the constant suction compression test and constant loading drying-wetting test while UPC suction controlled oedometer test was used for constant volume drying-wetting test.

For suction-controlled oedometer test, the application of suction to the specimen depends on the suction range. For suctions less than 14 MPa, the axis-translation technique was used while for suction higher than 14 MPa, vapour equilibrium technique was used. For the UPC suction-controlled oedometer test, the loading arm ratio is 1:20 in order to apply high stress. Vapour equilibrium technique was used to control the suction while the level arm is fitted with a load cell to measure the development of vertical stress due to the swelling pressure.

a) Schematic of control suction triaxial apparatus with inner mercury cell (Matyas and Radhakrishna, 1968)

b) Schematic of suction controlled suction triaxial apparatus by Blatz and Graham (2000)

Figure 2-11 Schematic of suction controlled triaxial apparatus

a) Schematic of suction controlled oedometer apparatus in the CIEMAT laboratory (Esteban, 1990)

b) Schematic of suction controlled oedometer apparatus in the UPC laboratory (Lloret et al., 2003)

Figure 2-12 Constant matric suction oedometer apparatus

2.5.1.2. Constant water content test

Constant water content test is easier to carry out as it only requires conventional apparatus. However, its parameter interpretation is more complicated as matric suction changes during compression. Thus, when the change in matric suction is required, it is necessary to do rapid measurement of the negative pore-water pressure. Another challenge is also to make sure that there is no evaporation of the soil specimen which may affect its behaviour. The rapid change of negative pore-water pressure below -100 kPa can be recorded with a HCT (Ridley and

Burland, 1993, Guan and Fredlund, 1997, Ridley and Burland, 1999, Tarantino and Mongiovi, 2002, He et al., 2006). Jotisankasa et al. (2007) and Tarantino and De Col (2008) have incorporated the HCT into the oedometer to investigate the compression behaviour of unsaturated soils as shown in Figure 2-13.

Jotisankasa et al. (2007) used a HCT developed by Ridley and Burland (1993) to measure the change in matric suction due to incremental oedometric loading. The load increment was maintained until the volumetric strain was less than 0.005% per hour. Jotisankasa et al. (2007) used a load cap perforated with holes in order to carry out wetting tests. However, Jotisankasa et al. (2007) noticed that evaporation affected the HCT reading as the HCT was placed on top of the soil specimen which was subjected to drying during the test. Nevertheless, HCT was able to measure reasonable pore-water pressure changes.

Tarantino and De Col (2008) used the Trento high-capacity tensiometer (Tarantino and Mongiovi, 2002) to measure the change in matric suction of the soil specimen which was loaded pneumatically in an oedometer at a constant rate of 5 kPa/min with each net normal stress increment of 5 kPa maintained for 30 minutes. Tarantino and De Col (2008) spent only 1600 minutes to finish the whole cycle of loading and unloading test which was significantly faster compared to osmotic suction controlled oedometer test which required more than two months or eight days with an air-circulation system to finish a test (Cunningham et al., 2003).

To avoid evaporation, Tarantino and De Col (2008) used a loading pad which seems to be impermeable and an O-ring was positioned in the tensiometer hole to prevent evaporation from the measurement area. A rolling membrane was used to seal the gap between the loading pad and the oedometer ring. Such design seems to be effective in avoiding evaporation. However, it does not allow the soil to be inundated and when the soil becomes saturated, undrained condition may develop and causes development of positive pore-water pressure.

Figure 2-13 Modified oedometers for constant water content test

Le et al. (2011) used a HCT developed at CERMES (Mantho, 2005, Cui et al., 2008) where the HCT was installed at the base of a modified oedometer cell as shown in Figure 2-14a. The HCT was pressurized by using a special saturation cell which is shown in Figure 2-14b. The installation of the HCT must be carried out within 30s in order to avoid any desaturation of the soil specimen. A pressure of 25 kPa was applied for 48 hours while waiting for the HCT reading to reach equilibrium. Subsequently, a loading and unloading stage was applied for 24 hours. Important observation in the Le et al. (2011) experimental programme was in the pore-water pressure response of the HCT. When additional loading was applied, the HCT always recorded positive pore-water pressure initially that subsequently dissipated and reached the equilibrium negative pore-water pressure. Le et al. (2011) believe that it was due to the dynamic effect corresponding to step loading that induces some instantaneous parasitic effects on the response of the HCT.

It will be shown in Chapter 5 that the HCT will only generate positive pore-water pressure when the soil has reached the transition state between the unsaturated state and the saturated state and when the soil becomes saturated. The positive pore-water pressure which was observed by Le et al. (2011) was postulated due to the contact condition between the HCT and the soil specimen as the set-up has to be completed within 30s. Thus, loading causes a very thin gap

between the soil specimen and the ceramic disk of the HCT which then attracts a thin layer of water which generates positive pore-water pressure (similar to the wetting cycle in the evaporation test). The positive pore-water pressure will then slowly dissipate when the HCT comes into full contact with the soil specimen. Thus, it appears that improvement is needed such that saturation can be carried out without detaching the HCT from the cell base.

Figure 2-14 Le et al. (2011) modified oedometer cell and saturation cell

2.5.1.3. Constant degree of saturation test

Burton et al. (2016) conducted compression test under constant degree of saturation. In constant degree of saturation test, the volume of water and the change in volume during the compression are measured. When the soil is loaded, additional suction is applied in order to decrease the amount of water such that the degree of saturation remains constant. Slow loading rate (12 kPa/h) is used to minimise the loading rate effect on the volume change response.

New insight on the compression behaviour of unsaturated soil may be provided by constant degree of saturation test. However, such test requires a very good feedback control for air pressure, water volume and void ratio. Inaccuracy and lag in one of the measurements may lead to error in determining the degree of saturation and the amount of suction that needs to be applied. Moreover, as compression of unsaturated soil is quite instantaneous, an increase

in net normal stress leads to an instantaneous increase in degree of saturation and leads to instantaneous small mechanical collapse.

Thus, the unsaturated soil specimen follows a complicated stress path where net normal stress is increased first (which causes settlement and mechanical collapse) and then follows with an increase in matric suction. Thus, more uncertainty is involved in the constant degree of saturation test.

2.5.2. Modelling the compression behaviour of unsaturated soils

A number of methods have been proposed to model the compression curve of unsaturated soils. The simplest method in modelling the unsaturated compression curve is perhaps by using surface fitting equations (Fredlund, 1979, Lloret and Alonso, 1985, Fredlund, 2000, Hung, 2003) as shown in Table 2-8. The parameters in the surface fitting equations can be obtained by optimizing the parameters in the surface fitting equation to fit a set of laboratory data. However, as the compression curve of the unsaturated soil is stress path dependent, different sets of parameters are required for different testing paths (Pham, 2005, Zhang and Lytton, 2009).

Another approach which is commonly adopted is to use an elasto-plastic model i.e. the Barcelona basic model (BBM) (Alonso et al., 1990). In this approach, parameters which are related to the change in the volume due to the change in matric suction (or state variables which are used to represent unsaturated condition) and net normal stress are used for any stress path.

Zhang and Lytton (2009) proposed a new method to calibrate the parameters in elasto-plastic model by using a modified state surface. The modified state surface approach is useful as it gives more flexibility in obtaining parameters of the constitutive model. For instance, parameters in the BBM which require constant matric suction triaxial test can be calibrated by

using either constant matric suction oedometer test (Zhang et al., 2016a) or constant water content oedometer test with pore-water pressure measurement instead (Zhang et al., 2016b). However, it is important to emphasize that by using oedometer test, the parameters which are related to the shear strength of unsaturated soil in the BBM are inaccurate unless additional effort is made to provide the information on these parameters (Zhang et al., 2016b).

The differences in any models usually lie on the different assumptions in these four main properties:

1. Initial void ratio
2. Apparent preconsolidation pressure
3. Unsaturated compression line
4. Unloading-reloading line

Different assumptions on these four main properties and also different stress-state variables which are used in each model have led to a number of variations in the elasto-plastic models. Examples of the differences are given in Figure 2-15.

2.5.2.1. Initial void ratio of unsaturated soils

The initial void ratio of unsaturated soils in this study is referred to as the void ratio under net normal stress of 1 kPa. The initial void ratio of unsaturated soil is determined by its history. For soil which is consolidated under saturated condition and then dried, the initial void ratio of the unsaturated soil can be determined from its shrinkage curve.

For compacted soils, the initial void ratio is determined based on its compaction curve. Thus, it is more difficult to estimate the initial void ratio of compacted soil as it depends on the compaction energy and also the water content of the soils.

For soils which is consolidated under dry condition such as loess, it is possible for the initial void ratio of such soil to be even higher than its void ratio at liquid limit. This behaviour has led

a number of researchers to evaluate the probability of collapse for such soils based on its density or void ratio (Denisov, 1951, Priklonski, 1952, Clevenger, 1956, Gibbs, 1961, Gibbs and Bara, 1962, Feda, 1964, Clemence and Finbarr, 1981). In this study, the main concern is on soil which is consolidated under saturated condition and then dried prior to constant water content loading. Thus, the initial void ratio is determined based on the saturated initial void ratio and the shrinkage curve.

Table 2-8 Surface fitting equation

Model	Equation	Parameters
Fredlund (1979)	$e = a + b \log(\sigma - u_a) + c \log(u_a - u_w)$	
Lloret and Alonso (1985)	$e = a + b \log(\sigma - u_a) + c \log(u_a - u_w) + d \log(\sigma - u_a) \log(u_a - u_w)$ $e = a + b \log(\sigma - u_a) + c \log(u_a - u_w) + d(\sigma - u_a) \log(u_a - u_w)$ $S = a - \left(1 - e^{-b(u_a - u_w)}\right) \left(c + d(\sigma - u_a)\right)$ $S = a - \left(\frac{e^{b(u_a - u_w)} - e^{-b(u_a - u_w)}}{e^{b(u_a - u_w)} + e^{-b(u_a - u_w)}}\right) \left(c + d(\sigma - u_a)\right)$	$a, b, c, d, f, g, a_1, a_2, b_1, b_2, c, d_1$ and d_2 , are curve fitting parameters
Fredlund (2000)	$e = (a_1 + a_2(u_a - u_w)) + (b_1 + b_2 \ln(u_a - u_w)) \ln \left[1 + \left(\frac{\sigma}{c}\right)^2\right]$ $+ (d_1 + d_2 \ln(u_a - u_w)) \ln \left[1 + \left(\frac{\sigma}{f}\right)^2\right]$	
Hung (2003)	$S = a + b \log[1 + (\sigma - u_a) + c(u_a - u_w)]$ or $e = a + b \log[1 + (\sigma - u_a) + c(u_a - u_w)]$ $S = a + b \log[1 + c(\sigma - u_a) + (u_a - u_w)]$ or $e = a + b \log[1 + c(\sigma - u_a) + (u_a - u_w)]$ $S = a + b \log[1 + c(\sigma - u_a) + d(u_a - u_w)]$ or $e = a + b \log[1 + c(\sigma - u_a) + d(u_a - u_w)]$ $S = a + b \log \left[\frac{1 + (\sigma - u_a) + c(u_a - u_w)}{1 + d(\sigma - u_a) + f(u_a - u_w)} \right]$ or $e = a + b \log \left[\frac{1 + (\sigma - u_a) + c(u_a - u_w)}{1 + d(\sigma - u_a) + f(u_a - u_w)} \right]$ $S = a + b \log \left[\frac{1 + c(\sigma - u_a) + (u_a - u_w)}{1 + d(\sigma - u_a) + f(u_a - u_w)} \right]$ or $e = a + b \log \left[\frac{1 + c(\sigma - u_a) + (u_a - u_w)}{1 + d(\sigma - u_a) + f(u_a - u_w)} \right]$ $S = a + b \log \left[\frac{1 + c(\sigma - u_a) + d(u_a - u_w)}{1 + f(\sigma - u_a) + g(u_a - u_w)} \right]$ or $e = a + b \log \left[\frac{1 + c(\sigma - u_a) + d(u_a - u_w)}{1 + f(\sigma - u_a) + g(u_a - u_w)} \right]$

Figure 2-15 Differences in assumptions on the unsaturated compression curve

2.5.2.2. Preconsolidation pressure of unsaturated soils

For saturated soils, the saturated preconsolidation pressure $P_{c,s}$ is the maximum effective stress which has been applied to the soil. Thus, beyond its $P_{c,s}$, the soil starts to have plastic deformation following its normal compression line (NCL). In unsaturated soil, the preconsolidation pressure is not only affected by its loading history but also by its matric

suction or inter-particle bonding (Munoz et al., 2011). Thus, when it is plotted with net normal stress, the unsaturated compression curve may yield above the NCL and the compression curve beyond this preconsolidation pressure is referred to as unsaturated compression line (UCL) in this study. The preconsolidation pressure for unsaturated compression curve which yield at the UCL is referred to as the apparent preconsolidation pressure $P_{c,d}$.

The value of $P_{c,d}$ is always increasing with respect to an increase in initial matric suction regardless of being tested under constant matric suction or constant water content condition (Alonso et al., 1990, Lloret et al., 2003, Jotisankasa, 2005, Jotisankasa et al., 2007, Leong et al., 2013). The reason for the increase in the apparent preconsolidation pressure with respect to an increase in matric suction (or degree of saturation) is due to the development of metastable pores / macropores which are held by the bonding generated under unsaturated condition and caused the soil to be less compressible. The bonds which support the metastable pores will disappear under inundation and therefore cause the soil to collapse under high net normal stress.

A number of equations have been proposed to represent the effect of matric suction on the apparent preconsolidation pressure $P_{c,d}(s)$ as shown in Table 2-9. The variation of the proposed equation depends on the assumption for the compression behaviour of unsaturated soils. For example, in Barcelona basic model (BBM) which was proposed by Alonso et al. (1990), there are two important assumptions which affect how the $P_{c,d}$ can be obtained which are:

1. κ is constant under any saturation condition
2. $\lambda(s) \leq \lambda(0)$

where κ is the slope of unloading line with x axis is in natural log scale which is equivalent with C_r when x axis is in log scale, $\lambda(0)$ is the slope of the NCL and $\lambda(s)$ is the slope of the UCL with x axis is in natural log scale which are equivalent with C_c and $C_c(s)$ when x axis is in log scale,

respectively. Wheeler and Sivakumar (1995) agreed that κ is constant under any saturation condition. However, they believed that $\lambda(s) \geq \lambda(0)$ and thus proposed a more general form as shown in Table 2-9. The change in apparent preconsolidation pressure can also be based on other state variables such as effective degree of saturation (Zhou et al., 2012a). All of these equations are referred to as the loading-collapse (LC) curve or the yield curve (Alonso et al., 1990). However, yield curve is a more appropriate term as it will be shown in the next chapter that inundation at net normal stress below $P_{c,d}(s)$ may cause the unsaturated soil to collapse.

Table 2-9 Equations for estimating apparent preconsolidation pressure

Reference	Equation	Parameters
Alonso et al. (1990)	$\left[\frac{P_{c,d}(s)}{P_{ref}} \right] = \left(\frac{P_{c,s}}{P_{ref}} \right)^{[\lambda_s - \kappa]/[\lambda(s) - \kappa]}$	$P_{c,d}(s)$ = apparent preconsolidation pressure $P_{c,s}$ = Preconsolidation pressure P_{ref} = reference pressure
Wheeler and Sivakumar (1995)	$\left[\frac{P_{c,d}(s)}{P_{atm}} \right] = \exp \left\{ \left[\frac{\lambda_s - \kappa}{\lambda(s) - \kappa} \right] \ln \left[\frac{P_{c,s}}{P_{atm}} \right] + \left[\frac{\kappa_s}{\lambda(s) - \kappa} \right] \ln \left(\frac{s + P_{atm}}{P_{atm}} \right) + \left[\frac{N(s) - N_s}{\lambda(s) - \kappa} \right] \right\}$	λ_s = slope of the NCL with x axis in natural log scale $\lambda(s)$ = slope of the UCL as a function of matric suction with x axis in natural log scale κ = slope of the unloading line s = matric suction κ_s = ratio of change in volume due to the change in matric suction P_{atm} = atmospheric pressure
Zhou et al. (2012a)	$P_{c,d}(S_e) = P_{c,s}^{(\lambda_s - \kappa)/(\lambda(S_e) - \kappa)}$	$N(s)$ = specific volume of the UCL at net normal stress equal to 1 kPa $N(0)$ = specific volume of the NCL at net normal stress equal to 1 kPa $\lambda(S_e)$ = slope of the UCL as a function of effective degree of saturation with x axis in natural log scale S_e = effective degree of saturation

2.5.2.3. Unsaturated compression line (UCL)

Once the unsaturated soil has been subjected to a net normal stress above its preconsolidation pressure, the compression will follow the UCL. However, the slope of the UCL is highly affected by the testing procedures and thus resulted in inconsistent results among researchers. Three commonly used assumptions regarding the UCL are:

1. The slope of the UCL decreases with increasing matric suction (Alonso et al., 1990, Alonso et al., 1999, Pham, 2005, Alonso et al., 2013).

2. The slope of the UCL increases with increasing matric suction and then the UCL merges with the NCL (Toll, 1990, Alonso, 1993, Wheeler and Sivakumar, 1995, Honda, 2000, Sivakumar and Wheeler, 2000, Sun et al., 2000, Futai et al., 2002, Silva et al., 2002)
3. The slope of the UCL is unaffected by matric suction (Larson et al., 1980, O'Sullivan, 1992, Smith et al., 1997, Arvidsson and Keller, 2004, Imhoff et al., 2004, Berli et al., 2015).

All of these different observations may be due to different ranges of net normal stress, degree of saturations, durations of the test and techniques which were used to control the matric suction of the unsaturated soil. It appears that the UCL becoming steeper with an increase in the net normal stress is more logical based on two plausible reasons:

1. Under constant matric suction condition, the degree of saturation of the unsaturated soil is increasing. Additional observation on the UCL provided by Mun and McCartney (2015) indicates that unsaturated compression curves merge with the saturated compression curve when their degree of saturation becomes 1 even under constant matric suction condition. The maximum constant matric suction used in their study was up to 250 kPa as shown in Figure 2-16. This observation then suggests that degree of saturation governs the position of the UCL rather than the matric suction and an increase in degree of saturation causes the unsaturated soil to slowly collapse as the degree of saturation increases until it merges with the NCL.
2. The difference between actual result and idealized path (Figure 2-10) due to small collapse settlement is caused by a sudden increase in pore-water pressure. The amount of collapse settlement is dependent on the test procedures, loading duration on each test and the load increment which is used.

Several examples of the equation to represent the slope of UCL are given in Table 2-10 while examples of equations to represent the void ratio of the UCL at net normal stress equal to 1 kPa is given in Table 2-11.

Figure 2-16 Compression behavior of clay specimens under constant matric suction condition with different initial degree of saturation

2.5.2.4. Unloading and reloading lines (URL)

When the soil is unloaded and reloaded, there is some degree of hysteresis between the unloading and reloading lines which is usually negligible for sandy soils, apparent for silty soils and significant for clayey soils (Cui et al., 2013). The slope of the unloading line is dependent on the net normal stress at which the unloading starts. The unloading line which is obtained prior to the preconsolidation pressure is smaller than unloading line which is obtained after the preconsolidation pressure (Schmertmann, 1955, Wang and Frost, 2004, Cui et al., 2013). The slope of unloading line which is taken at net normal stress higher than the preconsolidation pressure can be used to represent the slope of initial compression line and to estimate the preconsolidation pressure of the soil (Wang and Frost, 2004).

In order to take into account the reduction of the slope of the unloading line C_r , Wang and Frost (2004) deduced from Schmertmann (1955) observations and proposed that:

$$C_r(e_1) = C_r(e_2) \left(\frac{e_2 + 1}{e_1 + 1} \right)^{R_c} \quad (2.9)$$

Table 2-10 Equations for estimating UCL

Reference	Equation	Parameters
Alonso et al. (1990)	$\lambda(s) = \lambda_s [(1-r)\exp(-\beta s) + r]$	λ_s = slope of the NCL with x axis in natural log scale
Zhou et al. (2012b)	$\lambda(S_e) = \lambda_s - (1-S_e)^{a_1} (\lambda_s - \lambda_d)$ $\lambda(S_e) = \lambda_s - (1-S_e)^{a_1} (\lambda_s - \kappa)$	$\lambda(s)$ = slope of the UCL as a function of matric suction with x axis in natural log scale s = matric suction r, β = model parameters S_r = effective degree of saturation λ_d = slope of the UCL when effective degree of saturation equals to 0 κ = slope of the unloading line
Lloret and Alonso (1985)	$\lambda(s) = b + d \ln s$	a_1 = model parameter b, d = model parameters
Sheng et al. (2008)	$\lambda'(s) = \lambda'_s$	λ'_s = slope of the NCL in double natural log scale
Alonso et al. (2013)	$\lambda(s') = \lambda_s \left\{ (1-r') \left[1 - \left(\frac{s'}{s_{\lambda'}} \right)^{1/(1-\beta')} \right]^{-\beta'} + r' \right\}$ $s' = n.s$	$\lambda'(s)$ = slope of the UCL as a function of matric suction in double natural log scale n = porosity s' = modified suction
Kohgo et al. (1993)	$\lambda(s) = \frac{\lambda_s}{1 + \left(\frac{s - AEV}{a} \right)^b}$	$r', \beta', s_{\lambda'}$ = model parameters AEV = air entry value a, b = model parameters
Sheng (2011)	$\lambda(s) = \lambda_s \frac{\ln(\sigma - u_a)}{\ln[\sigma - u_a + f(s)]}$ $f(s)$ can be in several form such as: $f(s) = s$ $f(s) = S_r s$	σ = total stress u_a = pore-air pressure α, k = model parameters
Wong et al. (2010)	$\lambda(s) = \lambda_s - k \left(s \left(\frac{s}{AEV} \right)^{1/\alpha} - AEV \right)$	

Table 2-11 Equations to estimate the value of $e_N(s)$ or $N(s)$

Reference	Equation	Parameters
Toll (1990)	$N(S_r) = 1 + \frac{N_s - 1}{S_r}$	$N(S_r)$ = specific volume at the UCL as a function of S_r when net normal stress equals to 1 kPa S_r = degree of saturation N_s = Specific volume at the NCL when net normal stress equals to 1 kPa
Zhou et al. (2012b)	$e_N(S_r) = e_{N,s}$	$e_N(S_r)$ = void ratio at the UCL as a function of S_r when net normal stress equals to 1 kPa $e_{N,s}$ = void ratio at the NCL when net normal stress equals to 1 kPa
Berli et al. (2015)	$N(S_r) = N_s + b(C_c - C_r)(1 - S_r)$	C_c = slope of the normal compression line C_r = slope of the unloading-reloading line

where e_1 and e_2 are the void ratios when the soil starts to unload with slopes of the unloading line $C_r(e_1)$ and $C_r(e_2)$, respectively. R_c is an empirical parameter with average value for clay structure recommended to be 2.5. However, this value differs significantly for different clays (Wang and Frost, 2004).

The unloading line of unsaturated soil is usually assumed to be constant regardless of its degree of saturation or matric suction (Alonso et al., 1990, Alonso et al., 1999, Lloret et al., 2003, Jotisankasa et al., 2007, Sheng et al., 2008, Zhou et al., 2012b, Zhou et al., 2012a, Alonso et al., 2013) while the experimental evidence from the Futai and Almeida (2005) indicate that the slope of unloading line is smaller under unsaturated condition. Alonso et al. (2010) assumed that $C_{r,d}$ varies under different net normal stresses, degrees of saturation and matric suction but it is constant when effective stress for unsaturated soils is used. Examples of equations which represent the slope of the unloading line are given in Table 2-12.

Table 2-12 Equations to estimate the value of $C_{r,d}$

Reference	Equation	Parameters
Alonso et al. (1990), Berli et al. (2015), Pham (2005)	$C_{r,d} = C_{r,s}$	$C_{r,d}$ = slope of unloading line under unsaturated condition $C_{r,s}$ = slope of unloading line under saturated condition
Alonso et al. (2010)	$C_{r,d} = C_{r,s} \left(\frac{\sigma - u_a}{\sigma - u_a + S_r^e s} \right)$ $S_r^e = \frac{S_r - S_r^m}{1 - S_r^m}; S_r = S_r^m + S_r^M$	$\sigma - u_a$ = net normal stress S_r^e = effective degree of saturation considering the micro and macropore S_r^m = microscopic degree of saturation S_r^M = macroscopic degree of saturation

2.6. Wetting-induced behaviour of unsaturated soils

2.6.1. Introduction

The amount of settlement due to the change in total stress of unsaturated soils in general is less than those of saturated soils. However, when the unsaturated soil is wetted at different net normal stresses, different wetting-induced behaviour such as swelling and collapse may take place. Double oedometer test proposed by Jennings and Knight (1957b) has been used to

investigate the wetting-induced volume change of unsaturated soils (Jennings and Knight, 1957a, ASTM-D4546, 2008). Using double oedometer test, Jennings and Burland (1962) showed that under high net normal stress, decreasing matric suction of the soil through inundation with water causes the soil to decrease in volume. This phenomenon is referred to as collapse (ASTM-D4546, 2008) and has been observed by a number of researchers (Alonso et al., 1990, Jotisankasa, 2005, Jotisankasa et al., 2007). Swelling may cause excessive swelling pressure while collapse may cause excessive settlement.

Swelling and shrinkage is commonly associated with the change in the volume of the micropores (Alonso et al., 1999). Thus, drying and wetting a specimen will always generate shrinkage and swelling provided that the micropores are able to release and absorb the water as the matric suction acts like the effective stress on that particular pores (Alonso et al., 1999). On the other hand, collapse phenomenon of an unsaturated soil which is compressed to sufficiently high net normal stress is commonly associated with the meta-stable structure of the macropores where wetting will cause the bond which holds the macropores to break with time (Fredlund and Gan, 1995). Thus, re-drying a soil which just collapse will not cause the soil to expand as collapse settlement is irreversible.

There are two important questions in investigating the wetting-induced behaviour of unsaturated soils:

1. What is the maximum wetting-induced behaviour?
2. What is the actual wetting-induced behaviour?

While the maximum value can be investigated by inundating the unsaturated soil specimen with water, the actual value requires a relationship between the changes in the state variables which represent the unsaturated condition (i.e. matric suction, water content, degree of

saturation) and the desired wetting-induced behaviour such as swelling, swelling pressure or collapse.

Swelling or collapse settlement is commonly investigated by reducing matric suction of the specimen by either spraying or inundating the soil specimen with water under constant net normal stress (Jennings and Knight, 1957b, ASTM-D3877, 2008, ASTM-D4546, 2008, ASTM-D4829, 2011) or constant volume condition (Sullivan and McClelland, 1969, Fredlund et al., 1980, ASTM-D4546, 2008). ASTM-D4546 (2008) specifies three methods to investigate swelling which are Method A: Wetting-after-loading tests on multiple specimens, Method B: Single point wetting-after-loading test on a single specimen, and Method C: Loading-after-wetting test. Method A tries to measure the wetting-induced volume change of the entire range of net normal stresses while method B tries to measure the wetting-induced volume change at a particular net normal stress. Method C investigates the post-wetting compression behaviour and thus can be conducted together with either Method A or B.

In ASTM-D4546 (2008), swell pressure is defined as the minimum vertical stress required to prevent swelling. The swell pressure can be determined by using either constant volume test (Fredlund et al., 1980) or Method C (ASTM-D4546, 2008). The swell pressure which is obtained from constant volume test is known as constant volume swell pressure while the net normal stress which is required to revert the void ratio after swelling to the void ratio before swelling is referred to as the load-back swelling pressure in this study. Load-back swelling pressure is higher than constant volume swell pressure. The difference between swelling pressure determined by the constant net normal stress and the constant volume tests is attributed to crystalline and osmotic swell (Nelson et al., 2015). Fredlund et al. (1980) recommended the use of constant volume test with corrected swelling pressure to account for the sampling disturbance as shown in Figure 2-17a.

- a) Determination of corrected swelling pressure in the constant volume test (Fredlund et al., 1980)

- b) Determination of swelling pressure in free swell test modified from Shuai and Fredlund (1998)

Figure 2-17 Determination of swelling pressure

The illustration of constant volume swelling pressure is shown in Figure 2-17b. In constant volume test, matric suction of the soil is reduced through inundation with water and the volume is maintained constant by increasing the net normal stress. Both free swell and constant volume tests are commonly performed using the oedometer.

2.6.2. Swelling and swelling pressure

The maximum amount of swelling is commonly investigated based on the difference in the saturated and unsaturated compression curves. For example, Yoshida et al. (1983) proposed an equation which is given as:

$$\Delta e = C_{r,s} \log \left(\frac{P_s}{P_i} \right) \quad (2.9)$$

where Δe is the change in void ratio due to wetting, $C_{r,s}$ is the slope of the unloading line under saturated condition, P_s is the swelling pressure and P_i is the inundation pressure or the net normal stress which is applied during inundation. Equation (2.9) assumes that the swelling line (Figure 2-17b) has a slope which is equal to the slope of the unloading line $C_{r,s}$ and a change in net normal stress below P_s does not cause any change in the void ratio under dry condition and therefore, only $C_{r,s}$, P_s and P_i determine the amount of swelling.

However, the maximum amount of swelling may give overly conservative value especially when the depth of the active zone is very deep. Thus, in order to overcome this problem, matric suction or water content is commonly used to estimate the amount of swelling due to the change in these state variables. A ratio of change in volume to the change in the state variable is commonly used for the estimation. This ratio is commonly assumed to be constant (Alonso et al., 1990) but it is also possible to change with matric suction (Sheng et al., 2008). Some of these equations are given in Table 2-13.

Table 2-13 Equations to estimate the amount of swelling

Reference	Equation	Parameters
Yoshida et al. (1983)	$\Delta e = C_{r,s} \log \left(\frac{P_s}{P_i} \right)$	Δe = the maximum change in void ratio due to wetting $C_{r,s}$ = slope of the unloading line under saturated condition
Alonso et al. (1990)	$\Delta e(s) = \kappa_s \ln \left(\frac{s + P_{at}}{P_{at}} \right)$	$\Delta e(s)$ = the change in void ratio due to wetting as a function of matric suction κ_s = ratio of change in volume due to the change in natural log of change in matric suction
McKeen (1992)	$\Delta H = C_h \Delta s \cdot H \cdot c_1 \cdot c_2$ $C_h = -10 \left(\frac{\Delta s}{\Delta w} \right)^{-2}$ $c_1 = \frac{1 + 2K_0}{3}$ $c_2 = 1 - \frac{P_i}{P_{s,cv}}$	ΔH = amount of heave C_h = suction compression index Δs = change in matric suction in pF H = thickness of the layer c_1 = later restraint factor c_2 = coefficient for load effect P_i = inundation pressure $P_{s,cv}$ = constant volume swelling pressure
Department of the Army (1983)	$\Delta H = \frac{C_m}{1 + e_0} H \log \left(\frac{s_0}{s_f} \right)$ $C_m = \frac{\alpha G_s}{100B}$	C_m = suction index α = slope of shrinkage curve with specific volume as y axis B = slope of SWCC G_s = specific gravity of the soil e_0 = initial void ratio s_0 = initial matric suction s_f = final matric suction
Hamberg and Nelson (1984)	$\Delta H = \frac{C_w}{1 + e_0} \Delta w$ $C_w = \frac{\Delta e}{\Delta w}$	C_w = slope of the shrinkage or swelling curve Δw = change in water content
Lytton (1994)	$\frac{\Delta H}{H} = f \frac{\Delta V}{V}$ f varied from 0.5 for dry conditions up to 0.8 for wetting conditions $\frac{\Delta V}{V} = -C_{sv} \log \frac{s_f}{s_0} - C_{pv} \log \frac{P_f}{P_i} - C_{\psi v} \log \frac{\Psi_f}{\Psi_0}$	P_f final mean principal stress C_{sv} = matric suction volumetric compression index C_{pv} = mean principal stress volumetric compression index $C_{\psi v}$ = osmotic suction volumetric compression index Ψ_f = final osmotic suction Ψ_0 = initial osmotic suction

2.6.3. Collapse settlement

There are two prerequisites for collapsible soil (Feda, 1988, Feda et al., 1993):

1. The existence of a metastable structure which is sometimes referred to as macropores, and
2. The metastable structure of the soil has to be supported by additional bonding or cementation which disappears upon wetting.

There are two types of macropores based on its origin which are physically-based macropores and mechanically-based macropores. Physically-based macropores originate from the physical interaction between soil and external factors such as worm or root channels which cause the soil to display structural shrinkage behaviour (Cornelis et al., 2006a). This type of macropores does not directly disappear due to the saturation but become highly compressible under saturated condition. The amount of collapse of this type of macropores depends on the origin, structural-textural features, lithology, age, water content, the clay mineral content and the particle sizes (Grabowska-Olszewska, 1988, Assallay et al., 1998).

The second type of macropores originates from the mechanical interaction between unsaturated soils and net normal stress. Under unsaturated condition, the soil specimen becomes highly incompressible due to some of the pores are supported by the matric suction of the soils. Pores which are less affected by net normal stress due to the support from the matric suction are referred as macropores. Macropores are metastable because once the bond disappear i.e. due to wetting, the pores will collapse.

Considering the differences between physically-based macropores and mechanically-based macropores, it is possible for soils which has physically-based macropores to also develop mechanical-based macropores without significantly losing its physical-based macropores provided that the soil is compressed under highly unsaturated condition. This study focused on the development of the mechanically based macropores.

The amount of collapse usually increases with a decrease in initial water content or degree of saturation of the soil as shown in Figure 2-18a. In Figure 2-18a, the collapsing coefficient i_{mp} is defined as (Grabowska-Olszewska, 1988):

$$i_{mp} = h_i' - h_i'' \quad (2.10)$$

where h_i' is the height of the specimen at inundation pressure prior to inundation while h_i'' is the height of the specimen at inundation pressure after inundation. The amount of collapse is also dependent on the clay mineral content (Assallay et al., 1998) as shown in Figure 2-18b.

A number of criteria has been proposed to identify the possibility of collapsible soils. A summary of these criteria are shown in Table 2-14. The criteria in Table 2-14 can be differentiated into those which are related to the natural density and density at liquid limit which are also interchangeable with water content and void ratio (Denisov, 1951, Priklonski, 1952, Clevenger, 1956, Gibbs, 1961, Gibbs and Bara, 1962, Feda, 1964, Clemence and Finbarr, 1981), based on the percentage of certain soil particles (Handy, 1973) and on oedometer testing (Abelev, 1948, ASTM-D5333, 2003). For collapsibility based on density, collapse is more intense in a young loess and is exclusively restricted to loess of fine particles with sizes below 0.002 mm which commonly have loose structure (Grabowska-Olszewska, 1988, Yang, 1989). Thus, it seems that soils which have low preconsolidation pressure are more prone to collapse behaviour. A review of the criteria which are based on the index properties is given in Ayadat and Hanna (2012). Based on their evaluation, it is quite difficult to predict the collapsibility of soils based on their index properties.

For loess distributed by the wind under dry condition, the mineral bond allow the soil to have natural void ratio higher than its void ratio at liquid limit. Under saturated condition, it is not possible for a soil to have void ratio higher than its liquid limit as the soil particles are not in contact. Thus, soil will collapse when the soil is wetted to water content higher than its liquid limit. The weakness of this criterion is that it neglects the loading history of soils which may have an effect to the collapsibility of soils and the amount of collapse is not clear.

The other criterion is based on the amount of specific soil particles i.e. Handy (1973) who determined the collapsibility of soils based on the Iowa loess with clay contents. This criterion is based heavily on empirical prejudice and therefore ignores the effect of the state variable

that highly affects the possibility of collapse. Such method may only work for soil which has been subjected to a similar loading history.

a) Interrelation between the collapsing coefficient (i_{mp}) of loess deposits and their initial water content (Grabowska-Olszewska, 1988)

b) Effect of clay mineral to the amount of collapse (Assallay et al., 1998)

Figure 2-18 Relationship between initial water content and clay mineral content

Table 2-14 Criterion for soils collapsibility (from Rogers et al., 1994 and Howayek et al., 2011)

Author	Equation	Parameters
Abelev (1948)	$i = \frac{\Delta e_a}{1 + e_a}$ $i > 0.02 \rightarrow$ Highly collapsible	Δe_a = decrease in void ratio caused by wetting e_a = void ratio before wetting
Denisov (1951)	$K = \frac{e_{LL}}{e_0}$ $K = 0.5-0.75 \rightarrow$ highly collapsible $K = 1 \rightarrow$ non-collapsible loam $K = 1.5-2 \rightarrow$ non-collapsible soil	K = Coefficient of subsidence e_{LL} = void ratio at liquid limit e_0 = initial void ratio
Prikonski (1952)	$K_D = \frac{w_n - PL}{PI}$ $K_D < 0 \rightarrow$ highly collapsible soils $K_D > 0.5 \rightarrow$ non-collapsible soils $K_D > 0 \rightarrow$ swelling soils	K_D = Prikonski coefficient w_n = natural water content PL = plastic limit PI = plasticity index
Clevenger (1956)	$\gamma_{dry} < 12.6 \text{ kN/m}^3 \rightarrow$ large settlement $\gamma_{dry} > 14 \text{ kN/m}^3 \rightarrow$ small settlement	γ_{dry} = dry unit weight
Gibbs (1961)	$R = \frac{w_{sat}}{LL}$	R = Collapsible ratio w_{sat} = saturated water content LL = liquid limit
Gibbs and Bara (1962)	$\frac{LL}{w_{sat}} = \frac{LL}{100(1/\gamma_d - 1/2.7)}$ $LL/w_{sat} < 1 \rightarrow$ collapsible soils $LL/w_{sat} > 1 \rightarrow$ safe soils	
Feda (1964)	$K_L = \frac{(w_n / S_r) - PL}{PI}$ For $S_r < 100\%$, if $K_L > 0.85$, the soil is considered collapsible	K_L = Feda coefficient S_r = degree of saturation
Benites (1968)	2 g of soil are dropped into 12 ml of distilled water and specimen is timed until dispersed. Dispersion times for 20 to 30s were obtained for collapsing Arizona soils	
Handy (1973)	Iowa loess with clay (<0.002mm) contents: <16% : high probability of collapse 16-24% : probability of collapse 24-32% : < 50% probability of collapse >32% : usually safe from collapse	
Clemence and Finbarr (1981)	Sausage test: Sample soil is broken into two pieces into approximately equal volume. One is wetted and moulded in the hands to form a damp ball. The volume of the two specimens are compared again. Collapse is expected if the size of the damp ball is much smaller.	
ASTM-D5333 (2003)	$i = \frac{\Delta e_a}{1 + e_a}$ $i=0 \rightarrow$ none $0.1 < i < 2 \rightarrow$ slight $2.1 < i < 6 \rightarrow$ moderate $6.1 < i < 10 \rightarrow$ moderately severe $i > 10 \rightarrow$ severe	

Criterion which is based on the oedometer test is the most reliable and gives accurate prediction in terms of collapsibility and its amount. However, it is the most expensive especially when the initial degree of saturation varies widely and needs to be investigated. Many double oedometer tests have shown that the collapse settlement of the soil is dependent on the applied net normal stress during inundation or inundation pressure (Lin and Wang, 1988, Lin, 1995, Howayek et al., 2011, Al-Obaidi et al., 2013). Thus, the estimation of the amount of collapse must take into account the effect of the inundation pressure on the soil specimen. Feda et al. (1993) showed that the slope of the normal compression line (C_c) of reconstituted loess is the same as the natural loess and therefore it is possible to use reconstituted loess to investigate the compression behaviour of natural loess.

Leong et al. (2013) proposed an equation to estimate the settlement of unsaturated soils under different initial degrees of saturation. Leong et al. (2013) equation is:

$$e = e_0 \left[1 + \alpha S_0^\eta \left(\frac{\sigma}{P_a} \right) \right]^\beta \quad (2.11)$$

where e is the void ratio at net normal stress σ , P_a is the atmospheric pressure ($\approx 100\text{kPa}$) used to make σ dimensionless, α and β are empirical parameters which are obtained from the saturated compression curve, S_0 is the initial degree of saturation and η is an empirical parameter which is related to the unsaturated compression curve. Using Equation (2.11), it is possible to model any unsaturated compression curve at different initial degrees of saturation. The difference between the saturated and the unsaturated compression curves will be the amount of change in volume due to wetting which can be either swelling or collapse. However, as parameters α , β and η are commonly obtained by curve-fitting the oedometer test results, consistent values of α , β and η may not be obtained. The compression of unsaturated soils under constant water content condition increases the degree of saturation of the soils as the net normal stress increases and thus causes the unsaturated compression curve to slowly

merge into the saturated compression curve (Toll, 1990, Alonso, 1993, Wheeler and Sivakumar, 1995, Honda, 2000, Sivakumar and Wheeler, 2000, Futai et al., 2002, Silva et al., 2002). Equation (11) is useful when the change in degree of saturation is negligible. Leong et al. (2013) equation do not consider the change in degree of saturation upon compression which caused mechanical wetting. Thus, estimation of collapse settlement will be conservative.

In elasto-plastic model, the amount of collapse can be taken as the difference between UCL and NCL where UCL is commonly a function of matric suction or degree of saturation. Therefore, the amount of collapse is highly dependent on the assumption of the UCL. For instance, in the BBM model, UCL becomes gentler as matric suction increases. Thus, the amount of collapse is higher with an increase in the net normal stress. On the other hand, in the elasto-plastic model proposed by Wheeler and Sivakumar (1995), the UCL becomes steeper as matric suction increases until it merged with the NCL. Thus, the amount of collapse decreases as the net normal stress increases.

2.7. Effect of rate of strain on unsaturated soils

The investigation on the effect of strain rate on the compression behaviour of saturated soils has led to two hypotheses which are hypotheses A and B (Ladd et al., 1977, Jamiolkowski et al., 1985). In hypothesis A, changes in soil layer thickness will not affect the end of primary consolidation (EOP) and preconsolidation pressure. Therefore, EOP laboratory compression curve is the same as that in the field. Mesri and Choi (1985) and Sheahan and Watters (1997) presented data which support hypothesis A for non-cemented clay.

In hypothesis B, higher EOP vertical strains occur in laboratory test due to the viscous behaviour that occurs during pore-water pressure dissipation. The viscous behaviour varies depending on the thickness of the soil. Data which support hypothesis B produced lower preconsolidation pressure and a downward shift of the compression curve for cemented soils at low strain rates (Taylor, 1942, Suklje, 1957, Crawford, 1964, Bjerrum, 1967) and higher

preconsolidation pressure at high strain rate (Nash et al., 1992) are available in the literature. There are also cases where typical rate-dependent consolidation characteristics only appears in the recompression range (Vaid et al., 1979, Leroueil et al., 1983a, Leroueil et al., 1983b, Leroueil et al., 1985). Leroueil et al. (1983a) and Kabbaj et al. (1988) also showed laboratory and field data on cemented clays that supported hypothesis B. Under hypothesis B, laboratory compression curve requires correction in order to match those in the field.

In contrast to the numerous investigations of the strain rate effect on the compression behaviour of saturated soils, investigation of the strain rate effect on the compression behaviour of unsaturated soils remains scarce. As the unsaturated soil behaviour is dependent on its testing method, the effect of strain rate is investigated under constant matric suction or constant water content condition.

It is difficult to investigate the effect of strain rate on compression behaviour of unsaturated soils under constant matric suction condition due to the long equilibrium time required to maintain the matric suction in constant suction tests which limits the strain rates (Ho and Fredlund, 1982). Mun and McCartney (2015) used strain rate of 1%/h for soils which is nearly saturated ($S_r = 0.8$ to 1). The results indicate that when the degree of saturation reached 1, all of the unsaturated compression curve merge with saturated compression curve. Unfortunately, as there is no additional strain rate were used, there is no additional conclusion can be obtained.

It is possible to investigate the effect of strain rate on compression behaviour of unsaturated soils under constant water content condition due to the simple testing procedure. However, when matric suction measurement is desired, it requires rapid measurement of matric suction during the test. The development of HCT (Ridley and Burland, 1993, Guan and Fredlund, 1997, Ridley and Burland, 1999, Tarantino and Mongiovi, 2002, He et al., 2006) makes it possible to

investigate the change in pore-water pressure under constant water content condition (Jotisankasa et al., 2007, Tarantino and De Col, 2008, Munoz et al., 2011).

2.8. Summary

In this chapter, different approaches in utilising state variables to describe the behaviour of unsaturated soils have been discussed. Each approach has its advantages and disadvantages. Matric suction plays an important role in the behaviour of unsaturated soils and is most commonly used as the state variable for unsaturated soil. There are a number of theories to estimate the shear strength and permeability of unsaturated soils based on its matric suction. Soil-water characteristic curve is commonly used to relate the change in matric suction with the shear strength and hydraulic properties of soil. However, it is noted that SWCC is not unique under different density and under drying and wetting conditions. This hysteresis causes difficulties in using matric suction as a state variable.

However, hysteresis between drying and wetting in the relationship between water content and void ratio seems to be negligible. Therefore, it opens a possibility that water content can be a better state variable especially in considering the volume change of unsaturated soils. The constant water content approach is not expensive and requires short test duration as there is no need to control the matric suction of the specimen. Therefore, it is promising to use water content as a state variable to replace matric suction.

Investigations on the volume change behaviour of unsaturated soils (i.e. compression, shrinkage, swelling and collapse) showed numerous contradictory experimental evidence regarding the compression behaviour of unsaturated soils such as how the matric suction affects the slopes of the UCL and the URL. All of this inconsistent observations may not only due to the complicated behaviour of the unsaturated soil itself but also due to the different testing procedures which are used to interpret the soil behaviour.

The most popular methods in analysing unsaturated soil behaviour are by using constant matric suction and constant water content test. Constant matric suction test requires a very long test duration and expensive equipment which seems to be difficult to implement in practice which then exacerbates the gap between research and practice in implementing unsaturated soil mechanics. On the other hand, constant water content test seems to be more practical in terms of equipment and the duration. Thus, developing a model based on constant water content test seems to be more beneficial for geotechnical engineers especially when time and resources are limited.

Chapter 3. Materials and Methods

3.1. Introduction

In this chapter, the soils, apparatuses and experimental procedures to investigate the compression behaviour of unsaturated soils are described. There are five apparatuses used in this study which are:

1. Oedometer apparatus
2. Constant rate of strain (CRS) apparatus
3. Axis-translation apparatus
4. High-capacity tensiometer (HCT)
5. Chilled-mirror dew-point device

Oedometer and CRS apparatus were used to study the compression behaviour of soil specimens while axis-translation apparatus, HCT and chilled-mirror dew-point device were used to measure the matric suction of the soils. The axis-translation apparatus was mainly used to obtain the soil-water characteristic curve (SWCC) under zero net normal stress condition, the HCT was mainly used to measure the change in matric suction under constant water content condition, while the chilled-mirror dew point device was used to obtain the total suction of soils with extremely low water content.

The experimental procedures consist of:

1. Basic soil properties test and soil specimen preparation
2. Saturated compression test (SCT)
3. Constant water content compression test without pore-water pressure measurement (CWC)
4. Constant water content compression test with pore-water pressure measurement (CWC-P)

Table 3-1 Basic soil properties of the specimens

Basic soil properties	Kaolin	Residual soil	Sand	How to obtain:
S (%)	98.3	97.6	-	$w.G_s/e$
e	1.656	0.925	-	$(V_0 G_s \gamma_w / W_s) - 1$
n (%)	62.35	48.05	-	$e/(1+e)$
w_s (%)	61.2	34.2	-	ASTM D2216
θ (m ³ /m ³)	61.3	47.1	-	S.n
ρ_{dry} (kg/m ³)	1001	1377	-	$G_s \cdot \gamma_w / (1+e)$
ρ_t (kg/m ³)	1614	1848	-	$G_s(1+w)/(1+e)$
γ_t (kN/m ³)	16.14	18.48	—	$\rho_t \cdot g$
G_s	2.66	2.65	2.64	ASTM D854
PL	32	34	-	ASTM D4318-10
LL	60	56	-	BS1377 part 2
PI	28	22	-	LL-PL
Casagrande SL(%)	22	24	-	Casagrande method
%clay	100	0	-	ASTM-D422-63
%silt	0	17.5	-	ASTM-D422-63
%sand	0	82.5	99.3	ASTM D6913
$\rho_{dry-minimum}$ (t/m ³)	-	-	1.352	ASTM-D4253-14
$\rho_{dry-maximum}$ (t/m ³)	-	-	1.736	
e_{min}^*	-	-	0.518	
e_{max}	-	-	0.949	

3.2.2. Soil specimen preparation

Disturbed residual soil sample from Bukit Timah Granite was air dried for a month under the sun. The dried sample was then broken down and sieved through sieve #8 (2.36 mm). The sieving is needed to satisfy the maximum grain size criterion for oedometer test as listed in ASTM-D2435/D2435M (2011). The same procedure was adopted for preparation of both kaolin and residual soil from Bukit Timah Granite specimens. The powdered soil was mixed with water to about 1.5 times of their liquid limit (LL) to form a slurry. In order to ensure that the soil was mixed uniformly, a mixer as shown in Figure 3-2a was used. The soil slurry was then poured into a 30 cm diameter consolidation tank as shown in Figure 3-2b and a 150 kPa consolidation pressure was applied to consolidate the soil slurry.

a) Mixer

b) Consolidation tank

Figure 3-2 Mixer and consolidation tank

The change in height of the soil sample was then recorded until there was no change in height (approximately three to four days). When there was no more change in height (less than 0.5% strain), the consolidation of the soil sample was deemed to be completed. The soil sample was then extruded and then cut into smaller pieces of arbitrary dimensions (larger than the test specimen), wrapped with cling film, aluminium foil and then waxed. Excess samples from the different layers were then used for the measurement of the water content as shown in Tables 3-2 and 3-3. The soil samples were then stored in the moist room. As the water in the soil drained out of the consolidation tank during the consolidation, it was not possible for the water to flow back into the soil during unloading. Thus, the soil may become slightly unsaturated due to an increase in volume under constant water content condition. Unlike the other soil specimens, the sand specimen was prepared differently. The sand was sieved through sieve #10 (2.38 mm) so that only sand of particle sizes less than 2.38 mm was used.

Table 3-2 Water content distribution of kaolin sample in the consolidation tank

Water content calculation (by using leftover samples)				
Can. No	$W_{can}(\text{gram})$	$W_{can+wet\ soil}(\text{gram})$	$W_{can+dry\ soil}(\text{gram})$	$w(\%)$
1st layer	204.1	861.9	613.7	60.6
2nd layer	207.7	935.3	661.9	60.2
3rd layer	247.0	875.7	636.9	61.3
4th layer	258.8	920.6	669.4	61.2
5th layer	207.7	871.2	620.2	60.9
6th layer	215.2	858.3	613.6	61.4
7th layer	213.8	816.6	589.2	60.6

Table 3-3 Water content distribution of Bukit Timah Granite residual soil sample in the consolidation tank

Water content calculation (by using leftover samples)				
Can. No	$W_{can}(\text{gram})$	$W_{can+wet\ soil}(\text{gram})$	$W_{can+dry\ soil}(\text{gram})$	$w(\%)$
Layer 1	235.0	1215.0	960.0	35.2
Layer 2	233.0	1006.0	806.9	34.7
Layer 3	304.0	1036.0	849.8	34.1

3.3. Apparatuses

3.3.1. Oedometer apparatus

The oedometer apparatus used in this study is shown in Figure 3-3a. The loading arm ratio (LAR) of the oedometer is 10. There are two oedometer specimen ring sizes used in this study which are 6.3 cm and 5 cm. The 6.3 cm oedometer specimen ring was used to cut specimens for the oedometer test on saturated specimens. The 6.3 cm oedometer ring was also used to cut the specimens which were then dried for oedometer test on unsaturated specimens. The oedometer apparatus for tests on unsaturated specimens uses a 5 cm oedometer ring.

Unsaturated soil specimen may become brittle and hard which makes it difficult to cut from a large sample. It was found that pre-cutting the saturated specimen using a 6.3 cm diameter ring, letting it desaturate and then final cutting using a 5 cm diameter ring reduces the possibility of damaging the specimen greatly. A linear variable differential transformer (LVDT) was used to record the deformation of the specimen during the compression test.

During the compression test, the oedometer apparatus was subjected to deformation as well. In order to take into account the compliance of the oedometer apparatus, the oedometer apparatus was calibrated by replacing the specimen with a hard steel disk and then loaded and unloaded with the stress paths which were used in this study. The calibration curve was then constructed and used to correct the compression curve of the specimen. Details of the compliance correction can be seen in Appendix A.

3.3.2. Constant rate of strain apparatus

The constant rate of strain apparatus used in this study is shown in Figure 3-3b. The strain rate of the apparatus can be controlled in terms of mm/min. Load cell with capacity 5000 N was used to record the net normal stress which is applied to the specimen while a LVDT was used to record the deformation of the 5 cm diameter specimen.

Similar to the oedometer apparatus, compliance test was carried out by replacing the soil specimen with a hard steel disk. The deformation and the net normal stress along the test stress path were then recorded. The calibration curve was then constructed and the deformation of the specimen was then corrected with the calibration curve.

3.3.3. Axis-translation apparatus

The axis-translation apparatus uses the axis-translation principle (Hilf, 1956) and described in methods B and C in ASTM-D6836-02 (2008). In order to ensure that the air does not go into the water reservoir at the bottom of the specimen (which is used to measure the pore-water pressure), a high-air entry (HAE) disk is used to separate the air phase from the water phase. The high-air entry disk is permeable to water but impermeable to air as long as the applied air pressure is less than the air-entry value (AEV) of the HAE disk.

The soil specimen is placed at the top of the HAE disk while the water reservoir at the bottom of the HAE disk is connected to the pore-water pressure transducer in order to measure the water pressure. The value of the water pressure can be read from a pressure readout unit. Over time, air diffuses through the HAE disk and re-appears as air bubbles in the water reservoir. A flushing line connects the water reservoir to a flushing pot and enables the air bubbles to be flushed out of the water reservoir.

a) Oedometer apparatus for saturated compression test

b) CRS apparatus for saturated compression test
Figure 3-3 Apparatus for Saturated compression test

During the test, the flushing line be can either closed or opened. There are two tests which are commonly performed using the axis-translation apparatus (Murray and Sivakumar, 2010):

1. Null-type test for matric suction measurement
2. Water extraction test for controlling matric suction

In the null-type test, the flushing line is closed so that the water pressure in the water reservoir is equal to the pore-water pressure of the specimen u_w . Air pressure u_a is then adjusted such that the water pressure reading is around 0 kPa. Zero water pressure is chosen to avoid the possibility of net water into or out of the soil specimen thus affecting its volume. Once there is no more change in the pore-water pressure, the matric suction is calculated as the difference between the applied u_a and the measured u_w . Using the axis-translation technique, the matric suction of the soil can be measured without triggering cavitation.

A null-type test can be used to measure the matric suction of soil but requires the pore-air pressure to be adjusted manually. Thus, Leong et al. (2009) modified the axis-translation apparatus with an active control system. The schematic drawing of both conventional and modified axis-translation apparatus are shown in Figure 3-4.

The difference between the conventional and modified axis-translation apparatus is that the conventional axis-translation uses an air regulator with a pressure gauge to adjust u_a manually. On the other hand, in the modified axis-translation apparatus, the air pressure is controlled by using an electro-pneumatic pressure regulator. The electro-pneumatic pressure regulator applies air pressure according to the feedback from the feedback control board. The feedback control board read the water pressure from the pressure readout unit and thus gives the signal to increase the pore-air pressure to the electro-pneumatic pressure regulator when the pressure is lower than -10 kPa and to decrease the pore-air pressure when the pressure is higher than 10 kPa.

a) Conventional Axis translation apparatus

b) Modified axis translation apparatus with active control system

Figure 3-4 Axis translation apparatus which were used in this study (from leong et al., 2009)

Thus, by using the modified axis-translation apparatus with active control system, the matric suction measurement can be made more convenient. In this study, both of the apparatuses were used to measure the initial matric suction of the specimens and to construct the SWCC of the specimens.

In the water extraction test, the flushing line is opened so that the water migrates from the soil specimen to the flushing line until the matric suction of the soil specimen is equal to the difference between applied air and water pressures. Thus, when the water pressure is equal to 0 kPa, the matric suction of the soil specimen is equal to the applied air pressure. This technique is used when it is desired to control the matric suction of the specimen. Tempe cell

which is specially designed only for water extraction test for up to 100 kPa matric suction is also used in this study.

As mentioned in the previous section, the axis-translation technique is criticized for not duplicating the field condition as it does not allow the cavitation phenomenon. However, the significance of cavitation in unsaturated soil remains debatable and the axis-translation technique remains popular in laboratory unsaturated soil tests.

3.3.4. High-Capacity tensiometer

High-capacity tensiometer (Ridley and Burland, 1993, Guan and Fredlund, 1997, Ridley and Burland, 1999, Tarantino and Mongiovi, 2002, He et al., 2006) was developed to extend the measurement range of the normal tensiometer which is up to -90 kPa (Stannard, 1992). By using this device, it is now possible to measure the rapid change in negative pore-water pressure up to a very high value, typically -1500 kPa. The schematic drawing of a HCT is shown in Figure 3-5.

Figure 3-5 Schematic drawing of high-capacity tensiometer which is used in this study

The HCT is an indirect proof that it is possible for matric suction higher than 100 kPa to develop although it does not prove that alleviating the pore-air pressure does not affect the behaviour of unsaturated soils. The principle of HCT is to use a very small water reservoir between the transducer and the high air-entry (HAE) disk so that it is difficult for the air bubble to appear.

By pressurizing the water in the reservoir with high water pressure, all the remaining air can be dissolved.

The matric suction measurement can be simply performed by placing the soil specimen in contact with the HAE disk. Once the soil specimen is in contact with the HAE disk, water pressure in the HCT starts to equilibrate with the pore-water pressure in the soil specimen. The equilibrium time between HCT and the soil specimen is affected by several factors such as the permeability of soil and the high-air entry disk which are dependent on their suction level and the contact condition between the soil and the HCT (Oliveira and Marinho, 2008). Oliveira and Marinho (2008) suggested to use a soil paste such as kaolin with water content between its plastic and liquid limits to improve the contact condition between HCT and the soil specimen.

However, the use of HCT has not been standardized as there are several issues which remain under investigation such as:

1. How small the water reservoir in the HCT should be?
2. What is the best and most convenient method to saturate the high-air entry disk?
3. How to improve the contact between the specimen and the HCT?
4. What can be done to prevent evaporation from the soil specimen during the measurement?

It is uncertain on how small the water reservoir in the HCT should be. At first, it is believed that the water reservoir should be extremely small (i.e. 200 μm gap which gives 3 mm^3 water volume) to inhibit the formation of air bubbles (Ridley and Burland, 1993). However, further investigation which has been carried by Mendes and Buzzi (2013) regarding the size of the water reservoir indicated that it is possible to increase its size up to approximately 1 cm gap which gives 1100 mm^3 of water volume.

Saturating high-air entry disk can be quite challenging especially when the air-entry value of the disk is very high (i.e. around 1500 kPa). The most common method to saturate the HAE disk is by applying high water pressure and sometimes followed by applying vacuum (He et al., 2006). The water pressure enables the water to flow through the HAE disk into the bottom water reservoir and then dissolved the air in the water reservoir. Another option is to use CO₂ as a replacement of air before saturating as CO₂ is hydrophilic (Acikel and Mancuso, 2010). Unfortunately, Acikel and Mancuso (2010) has not tested whether it is possible to measure matric suction more than 1500 kPa, which is the normal range of matric suction that can be measured by using water pressure to saturate the HCT.

Oliveira and Marinho (2008) showed that the contact condition between HCT and the soil specimen can be improved by using a soil paste (i.e. kaolin or residual soil) with water content between its plastic and liquid limits. Several advantages in using the soil paste are:

1. It helps to join the water phase in the specimen and the water phase in the HCT.
2. It reduces the evaporation rate prior to placing the specimen at the top of the HCT.
Thus, allowing a more flexible setup.
3. It improves the contact between the HCT and the soil specimen especially when the surface of the soil is “rough”.

Thus, in order to improve the contact condition, kaolin paste was used for residual soil specimens from Bukit Timah Granite which have rough surfaces.

The versatility of using HCT is its small size which allows it to be installed into any devices by using minimum modification. Thus, HCT is commonly installed in oedometer and triaxial apparatuses to measure the matric suction under constant water content condition. For example, Tarantino and De Col (2008), Jotisankasa et al. (2007) and Le et al. (2011) have incorporated the HCT into the oedometer to investigate the compression behaviour of unsaturated soils as shown in Figure 2-13 and 2-14. However, it is quite common that the soil

specimen is connected to the atmosphere in order to let the pore-air pressure u_a remain at atmospheric condition. Thus, it is possible for the soil specimen to dry due to evaporation during the test (Jotisankasa et al., 2007). Such a situation is not desirable as it becomes difficult to interpret the relationship between measured matric suction and the water content of the specimen. Latex or aluminium membrane is commonly used to reduce the evaporation of the soil specimen (Ridley and Burland, 1999, Jotisankasa et al., 2007, Tarantino and De Col, 2008, Munoz et al., 2011).

Lourenço et al. (2009) observed three variations of HCT response during matric suction measurement before it reaches equilibrium as shown in Figure 3-6. Type A equilibrium is the normal equilibrium where pore-water pressure decreases from zero until it reached the matric suction of the soil specimen.

In type B equilibrium, negative pore-water pressure decreases and then suddenly increases until it reached equilibrium. Lourenço et al. (2009) suggested that this phenomenon is due to the pore-water pressure being not fully equalized throughout the specimen before the measurement was taken. However, it is doubtful that pore-water pressure in the soil specimen requires less than one hour to reach equilibrium. From the evaporation test, it is known that when the HCT is exposed to dry air, the pore-water pressure in the HCT starts to go negative. Thus, type B equilibrium seems to be due to the inadequate contact between HCT and the soil specimen. An inadequate contact may cause the HCT to be subjected to evaporation which causes the HCT to read a higher negative pore-water pressure. However, the amount of water which is connected between soil specimen and the HCT increases with time. Once there is sufficient water which connects the HCT with the soil specimen, the HCT starts to read the negative pore-water pressure of the soil specimen and causes the reversal in the HCT response until it reads the pore-water pressure of the soil specimen.

a) Type A equilibrium

b) Type B equilibrium

c) Type C equilibrium

Figure 3-6 Equilibrium of the high-capacity tensiometer (Lourenço et al., 2009)

In type C equilibrium, the negative pore-water pressure keeps decreasing. The reason for such phenomenon appears to be due to the evaporation which may take place during the test (Lourenço et al., 2009).

3.3.5. Chilled-mirror dew-point device

Chilled-mirror dew-point device (Figure 3-7) measures the total suction of the soil specimen by measuring relative humidity of the air above the specimen. It is considered to be the most precise method for dew-point measurement (Leong et al., 2003). The relative humidity measurement is performed by placing the soil specimen in a sealed chamber containing a mirror and a detector of the condensation on the mirror. Inside the close chamber, the relative humidity of the air is changed until it is equal to the relative humidity of the soil specimen. At the dew-point (temperature at which the air can no longer hold all of the water vapour and thus caused water vapour to condense), the water vapour is sufficient to saturate the air. The relative humidity is then taken as the ratio of the saturated vapour pressure of water at the dew point to the saturated vapour pressure of water at the air temperature.

Figure 3-7 Schematic of chilled-mirror dew-point device (Leong et al., 2003)

The advantages of using chilled-mirror dew-point device is the ability to measure large suction within a short time (15-30 minutes). However, as the size of the container is small, the soil specimen may have to be broken up in order to be measured. Thus, the SWCC which is produced using this device is from a highly disturbed sample and therefore may not be able to

consider the loading history of the soil. This device is thus suitable for high total suction measurement where the loading history no longer affects the SWCC of the soil (Jotisankasa et al., 2007, Salager et al., 2010, Salager et al., 2013).

3.4. Saturated compression test (SCT)

The saturated compression test (SCT) was carried out in order to obtain the normal compression line and it also served as the reference curve for the unsaturated compression curves. Based on the SCT, the slope of the normal compression line C_c , the void ratio at net normal stress equals to 1 kPa of the saturated normal compression line $e_{N,s}$, the slope of the unloading-reloading line $C_{r,s}$ and the saturated preconsolidation pressure $P_{c,s}$ were obtained.

The SCT was conducted by using either the oedometer apparatus (ASTM-D2435/D2435M, 2011) or the constant rate of strain (CRS) apparatus (ASTM-D4186, 2006) as shown in Figure 3-3. Several strain rates were used to verify that the strain rate is appropriate.

3.5. Constant water content test with and without pore-water pressure measurement

The constant water content test (CWC) and constant water content test with pore-water pressure measurement (CWC-P) were conducted in order to study the compression behaviour of unsaturated soils under constant water content condition. In constant water content test, the soil specimen is assumed to have a constant water content while the matric suction changes with degree of saturation of the soil specimen.

In general, the tests were carried out to investigate several compression parameters such as slope of the unsaturated compression line (UCL), slope of the unloading line $C_{r,d}$, apparent preconsolidation pressure $P_{c,d}$, and the void ratio at net normal stress equals to 1 kPa of the UCL $e_{N,d}$. Additional investigation on the change in matric suction due to the change in net normal stress was carried out using CWC-P.

In CWC-P, a high-capacity tensiometer (Figure 3-8a) was used to measure the change in matric suction during compression. HCT was installed at the centre of the modified base plate (Figure 3-8b).

A mini saturation chamber as shown in Figure 3-9 was designed such that it can be used to apply water pressure to the HCT while the HCT is attached to the modified base plate. By using the mini saturation chamber, there is no need to detach the HCT from the modified base plate like in Le et al. (2011). Thus, when the specimen was placed on the modified base plate, it was directly placed on top of the HCT. It is shown in Chapter 5 that the negative pore-water pressure changes smoothly as the pressure was applied since the HCT was already properly secured to the modified base plate. The modified base plate was made such that it can be assembled into the oedometer and CRS apparatus. The schematic drawing of the modified base plate is given in Figure 3-10 while the schematic drawing of the oedometer and CRS apparatuses are given in Figure 3-11.

HCT was calibrated using positive pore-water pressure as shown in Figure 3-12a by assuming that the flexing of the diaphragm (inward or outward) follows Hooke's law (Ridley and Burland, 1993, He et al., 2006) and the calibration curve does not change whether the HCT measures positive or negative pore-water pressure. Thus, measurement of negative pore-water pressure was done by extrapolating the calibration curve. The calibration was carried out each time before the HCT was used.

The response of the HCT was measured through evaporation test. In evaporation test, the HCT was dried in air to reduce its matric suction as shown in Figure 3-12b. Several wetting and drying cycles were used in the evaporation test to evaluate the responsiveness of HCT. HCT was wetted by applying a water droplet on the ceramic disk to increase the pore-water pressure while tissue paper was used to wipe the water droplet to allow the HCT to undergo drying.

Figure 3-12b shows that the HCT can undergo several drying and wetting cycles without any problem. The capacity of the HCT was between 1200 kPa to 1500 kPa, depending on the time duration used to pre-pressurize the HCT. Therefore, CWC-P is limited only to soil specimens with matric suction less than 1500 kPa. For matric suction higher than 1500 kPa or when the HCT cavitates during the test, CWC was then used to investigate the compression behaviour of unsaturated soils.

a) High-capacity tensiometer

b) Modified base plate

c) Modified base plate mounted at oedometer apparatus

d) Modified base plate mounted at CRS apparatus

Figure 3-8 Apparatus for constant water content compression test with pore-water pressure measurement (CWC-P)

a) Schematic top view

b) Top view

c) Schematic bottom view

d) Bottom view

e) Schematic side view

Figure 3-9 Mini saturation chamber for saturating high-capacity tensiometer

Figure 3-10 Schematic diagram of the modified base plate

Preparation of the kaolin specimen for the CWC and CWC-P test are as follows:

1. Cut the specimen by using a 6.3 cm internal diameter ring with a cutting edge.
2. Dry the soil specimen to the desired matric suction or water content by using either pressure plate, desiccator, or by air drying.
3. The specimen was then cut with a 5 cm internal diameter oedometer ring.
4. Dry filter paper was then placed at the top and bottom of the specimen unless the bottom of the specimen was in contact with the HCT (CWC-P).
5. For the CWC-P, the specimen was then placed on the modified base plate and the initial matric suction of the soil specimen was then measured by the HCT.
6. In order to improve the contact between high-capacity tensiometer and the soil specimen, kaolin paste was used to cover the surface of the HCT.
7. A dry load cap was then placed on top of the dry filter paper and soil specimen.
8. The soil was then loaded/unloaded by using either the oedometer or the CRS apparatus.

To evaluate strain rate effects in the CRS tests on the compression behaviour of the unsaturated soils, several strain rates were used.

a) Modified oedometer apparatus

b) Modified CRS apparatus

Figure 3-11 Schematic diagrams of the modified oedometer and the CRS apparatus

a) Calibration of the high-capacity tensiometer

b) Evaporation test

Figure 3-12 Calibration of the high-capacity tensiometer and evaporation test

3.6. Inundation test under constant net normal stress and constant volume condition

Inundation test was carried out in the middle of the CWC or CWC-P test. The purpose of the inundation test is to investigate the wetting induced behaviour and the compression behaviour of the unsaturated soil post inundation.

When oedometer apparatus was used, the inundation is under a constant net normal stress which may lead to either swelling or collapse phenomenon. When CRS apparatus was used, the inundation is under constant volume condition which leads to the development of swelling pressure. For constant volume test, the load cell was used to record the change in net normal stress due to wetting.

3.7. Investigation on SWCC of soil specimen at different initial densities

The SWCC of the soil specimen was obtained by using either the Tempe cell (Figure 3-13a), the pressure plate (Figure 3-13b), the HCT or the desiccator. The SWCC from the Tempe cell or the pressure plate was obtained following ASTM-D6836-02 (2008). The calculation of the water content of the soil specimen at different matric suctions was obtained as follows:

$$w_i = \left(\frac{W_i}{W_{\text{reff}}} \right) (w_{\text{reff}} + 1) - 1 \quad (3.1)$$

where w_i is the water content at specimen weight W_i , w_{reff} is the reference water content at specimen weight W_{reff} . The value of w_{reff} can be either the initial estimated or oven dried water content of the specimen.

Figure 3-13 Tempe cell

The investigation on SWCC of soil specimen under different initial densities was carried out differently for sand specimens and fine-grained soil specimens. For the sand specimens, the sand specimens were compacted at different relative densities by tamping it inside the 1 bar

Tempe cell. The weight of the Tempe cell (W_{Tempe}) and the weight of the dry sand specimen (W_{sand}) were recorded.

In order to saturate the sand specimen, the sand specimen was inundated for one day by allowing water to flow from bottom to top. An air pressure of 0.1 kPa was then applied on the sand specimen until no excess water flows out of the Tempe cell. The weight of the Tempe cell with the wet sand specimen was then measured ($W_{\text{Tempe}} + W_0$). Air pressure was then applied and the weight of the Tempe cell was measured daily. Once there was no more change in the weight of the Tempe cell, it is assumed that the sand specimen has reached equilibrium and it has matric suction equal to the applied air-pressure. The weight of the Tempe cell with sand ($W_{\text{Tempe}} + W_i$) was used to calculate the water content of the sand specimen at the applied matric suction. Once the maximum air pressure has been reached, the weight of the Tempe cell (W_i) was recorded. Then, the reference water content w_{reff} of the sand specimen was determined by oven drying the sand specimen. The water content of the sand specimen w_i was then obtained by using Equation (3.1). The test was carried out for several different relative densities in order to observe the effect of density on the SWCC.

For the fine-grained soils, the pressure plate apparatus was used to obtain the SWCC. The soil specimen was cut by using a 6.3 cm internal diameter ring. The soil trimmings were used to obtain the initial water content w_0 which also served as the reference water content w_{reff} . The initial weight of the specimen W_0 was determined. Then, the soil specimen was placed in the axis-translation apparatus and air pressure was applied. The weight of the specimen was monitored until it reaches equilibrium. Equation (3.1) was then used to calculate water content at each matric suction.

The shrinkage curve of the soil specimen was obtained by measuring the height and diameter of the soil specimen as the soil specimen dries. The height and diameter were taken twice at

four different locations of the specimen in order to take into account the non-uniformity of the soil specimen. The void ratio of the specimen was then calculated as follows:

$$e = 0.25\pi D_{ave}^2 H_{ave} \left(\frac{1 + w_{reff}}{M_{reff}} G_s \right) - 1 \quad (3.2)$$

where D_{ave} is average diameter of the specimen and H_{ave} is average height of the specimen.

3.8. Summary of tests

Every soil specimen in this study may be subjected to different sets of test. The summary of the tests for all of the soil specimens are given in Table 3-4 to Table 3-7. In these tables, notation A stands for available, SCT stands for saturated compression test, CWC stands for constant water content test without pore-water pressure measurement, CWC-P stands for constant water content test with pore-water pressure measurement, IT stands for inundation test under constant net normal stress, SWCC(e) stands for soil-water characteristic curve test after compression test, CV stands for constant volume test, SC(e) stands for shrinkage curve test after compression test.

Table 3-4 Summary of tests on kaolin specimens which were tested using the oedometer apparatus

Specimen	SCT	CWC	CWC-P	IT	SWCC(e)
KOdSat1	A	-	-	-	-
KOdSat2	A	-	-	-	-
KOdSat3	A	-	-	-	-
KOdSat4	A	-	-	-	A
Kw1	-	A	-	A	-
Kw5	-	A	-	A	-
Kw6	-	A	-	A	-
Kw9	-	A	-	A	-
Kw60s40	-	-	A	-	-
Kw57s110	-	-	A	-	-
Kw57s81	-	-	A	-	-
Kw48s287	-	-	A	-	-
Kw47s252	-	-	A	-	-
Kw42s347	-	-	A	-	-
Kw37s399	-	-	A	-	-
Kw30s468	-	-	A	-	-
Kw30s556	-	-	A	A	-
Kw23s895	-	-	A	A	-
Kw15s1044	-	-	A	A	-

*A stands for available

Table 3-5 Summary of tests on kaolin specimens which were tested using the CRS apparatus

Specimen	SCT	CWC	CWC-P	CV
KCRS-0.002	A	-	-	-
KCRS-0.005	A	-	-	-
Kw1.5-0.1	-	A	-	A
Kw1.2-0.01	-	A	-	A
Kw1.9-0.001	-	A	-	A
Kw47.5-362-0.01	-	-	A	-
Kw37.5-465-0.1	-	-	A	-
Kw37.5-490-0.001	-	-	A	-
Kw47.5-310-0.001	-	-	A	-
Kw48.37-345-0.001	-	-	A	-
Kw48.87-387-0.001	-	-	A	-
Kw47.87-335-0.01	-	-	A	-
Kw48-322-0.01	-	-	A	-
Kw48-363-0.01	-	-	A	-
Kw49-225-0.01	-	-	A	-
Kw49-300-0.01	-	-	A	-
Kw37.5-450-0.1	-	-	A	-
Kw47.3-363-0.1	-	-	A	-
Kw47.6-363-0.1	-	-	A	-
Kw48-0.1-400	-	-	A	-
Kw48-271-0.1	-	-	A	-
Kw47-0.003	-	-	A	-
Kw51-500-0.003	-	-	A	-
Kw47.72-384-0.0005	-	-	A	-
Kw47.35-370-0.005	-	-	A	-
Kw48.54-450-0.005	-	-	A	-
Kw49.2-275-0.005	-	-	A	-
Kw44-393-0.001	-	-	A	-
Kw53-155-0.005	-	-	A	-

*A stands for available

Table 3-6 Summary of tests on residual soil specimens from Bukit Timah Granite which were tested using the oedometer apparatus

Specimen	SCT	CWC	CWC-P	IT	SC(e)
BTodSat1	A	-	-	-	-
BTodSat2	A	-	-	-	A
BTw0.1	-	A	-	A	-
BTw1.3	-	A	-	A	A
BTw20s902	-	-	A	A	A
BTw26s332	-	-	A	A	A

*A stands for available

Table 3-7 Summary of tests on residual soil using the specimens from Bukit Timah Granite which were tested using the CRS apparatus

Specimen	CWC	CWC-P	CV	SC(e)
BTw15-0.001	A	-	A	-
BTw18-0.001	A	-	A	A
BTw18-975-0.01	-	A	A	A
BTw18-1039-0.1	-	A	A	-
BTw26-321-0.001	-	A	A	-
BTw26-251-0.01	-	A	A	-
BTw26-300-0.1	-	A	A	A

*A stands for available

Chapter 4. General equation for SWCC and Shrinkage curve

4.1. Introduction

In this study SWCC-w is the main focus. By using SWCC-w, it is easier to construct the other SWCCs (i.e. SWCC-S and SWCC- θ) and the effect of density on the SWCC-w is only on the initial part of the curve. A numbers of equations have been proposed to curve fit the SWCC and shrinkage curve. Several problems which have been discussed regarding SWCC and shrinkage curve equations are:

1. Most of the equations which are used to represent SWCC or shrinkage curve do not use graphically obtainable parameters and therefore their parameters may not be unique and require optimization to be obtained.
2. Both the shrinkage curve and the SWCC are affected by the density of the soil and therefore, parameters which do not use graphically obtainable parameters are different for different densities.
3. Linear segments of the shrinkage curve may vary from 2 to 4 while linear segments of the SWCC may vary from 3 to 5. Some equations were made for specific type of shrinkage curve and thus cannot be applied generally.
4. Equations which used graphically obtainable parameters are usually difficult to derive and in some cases difficult to extend to bimodal SWCC or structural shrinkage curve.

In this chapter, a general equation to represent SWCC-w and shrinkage curve is proposed. The proposed equation uses graphically obtainable parameters and can be extended to any number of linear segments. An effort is also made to extend the proposed equation to take into account the effect of densities. Data from experiments and literature are used to evaluate the performance of the proposed equation.

Shrinkage curves were obtained for kaolin while SWCC was obtained for kaolin and sand to observe the difference in the SWCCs between coarse-grained and fine-grained soils with different initial densities.

4.2. General equation to curve fit multi-linear segments curve

The shrinkage curve and SWCC are multi-linear segments curve. However, a number of empirical equation which have been proposed for shrinkage curve and SWCC curve use parameters which are obtained by curve fitting and requires an optimization method. On the other hand, using graphically obtainable parameters such as the slope and the intersection point between linear segments have some advantages such as:

- a) The values of the slopes of virgin drying line and residual line of SWCC-w remain constant regardless of densities.
- b) The intersection point and slope of each linear segment can be obtained graphically.
- c) The values of the parameters are unique which is useful especially in correlating the value of the parameter with other parameters.

In this section, a new equation to represent multi-linear segments curve is derived. The application of the proposed equation for shrinkage curve and SWCC is illustrated. The proposed equation is then extended to model unimodal SWCC-w of soils with different initial densities.

Figure 4-1 shows a multi-linear segments curve with function $f(x)$. Each segment on this curve can be represented with three parameters which are m_i , x_i and k_i representing the slope of segment i , intersection point between segments i and $i-1$, and the curvature parameter between segments i and $i-1$, respectively. Heaviside unit step function $H(x, x_i)$ can be used to describe the abrupt transition at $x = x_i$ between linear segments. The value of $H(x, x_i)$ is equal

to 0 when $x < x_i$ and $H(x, x_i)$ is equal to 1 when $x > x_i$. When a smooth transition is desired, the approximate value of $H(x, x_i, k_i)$ is given as:

Figure 4-1 Multi-linear segments curve

$$H(x, x_i, k_i) = \frac{1}{2} + \frac{1}{2} \tanh k_i (x - x_i) \quad (4.1)$$

when the value of k_i reaches infinity, the approximate Heaviside function in Equation (4.1) tends to the Heaviside function. Thus, the change in slope between each segment can be represented as:

$$\frac{df(x)}{dx} = m = m_1 + \sum_{i=2}^n H(x, x_i, k_i) (m_i - m_{i-1}) \quad (4.2)$$

where n is the number of linear segments. In order to obtain the value of $f(x)$, Equation (4.2) is integrated with respect to $x = x_1$ up to $x = x$ such that:

$$f(x) = f(x_1) + \int_{x=x_1}^{x=x} m dx = f(x_1) + m_1 (x - x_1) + \sum_{i=2}^n \int_{x=x_1}^{x=x} H(x, x_i, k_i) (m_i - m_{i-1}) dx \quad (4.3)$$

Since $(m_i - m_{i-1})$ is constant, Equation (4.3) can be solved as:

$$f(x) = f(x_1) + \int_{x=x_1}^{x=x} m dx = f(x_1) + m_1(x - x_1) + \sum_{i=2}^n R_i(x, x_i, k_i)(m_i - m_{i-1}) \quad (4.4)$$

where $R_i(x, x_i, k_i)$ is the ramp function and is given as:

$$R_i(x, x_i, k_i) = \int_{x=x_1}^{x=x} H(x, x_i, k_i) dx = \frac{1}{2} \left\langle (x - x_1) + \frac{1}{k_i} \ln \left\{ \frac{\cosh[k_i(x - x_i)]}{\cosh[k_i(x_i - x_1)]} \right\} \right\rangle \quad (4.5)$$

where x_1 is the minimum axis value in $f(x_1)$. The term $R_i(x, x_i, k_i)(m_i - m_{i-1})$ represents each linear segment from $i = 2$ to $i = n$. Thus, it can be defined as a transition function $T_i(x, x_i, m_i, m_{i-1}, k_i)$ such that:

$$T_i(x, x_i, m_i, m_{i-1}, k_i) = R_i(x, x_i, k_i)(m_i - m_{i-1}) \quad (4.6)$$

Substituting Equation (4.6) into Equation (4.4) gives the general equation for a curve with multi-linear segments:

$$f(x) = f(x_1) + m_1(x - x_1) + \sum_{i=2}^n T_i(x, x_i, m_i, m_{i-1}, k_i) \quad (4.7)$$

The curvature parameter k_i can be estimated by using the properties of the derivative of the Heaviside function which is the Dirac delta function. The Dirac delta function is given as:

$$\delta_i(x, x_i, k_i) = \frac{k_i}{2 \cosh^2[k_i(x - x_i)]} \quad (4.8)$$

The maximum value of the δ_i is given when $x = x_i$. Thus, substituting the value of $x = x_i$ into Equation (8) gives:

$$\delta_i(x = x_i, x_i, k_i) = \delta_{\max} = \frac{k_i}{2} \quad (4.9)$$

In Figure 4-2, it is shown that the value of $\delta_{\max}$ is equal to 0.5 and the Heaviside function and Dirac delta function are plotted with x_i equals to 10 and k_i equals to 1. When $x < x_i$, the value of H_i tends to 0 and thus, m tends to m_{i-1} . When $x > x_i$, the value of H_i tends to 1 and thus, m tends to m_i .

Figure 4-2 Relationship between x_{i-} , x_{i+} , x_i and k_i

Therefore, when the $\delta_{\max}$ line which has slope equals to $\delta_{\max}$ gives H_i equals to 0, the curve starts to depart from segment $i - 1$. On the other hand, when the $\delta_{\max}$ line gives H_i equals to 1, the curve starts to converge into segment i . Thus, the curvature parameter k_i can be estimated as:

$$k_i = \frac{2}{x_{i+} - x_{i-}} \quad (4.10a)$$

$$k_i = \frac{1}{x_i - x_{i-}} \quad (4.10b)$$

$$k_i = \frac{1}{x_{i+} - x_i} \quad (4.10c)$$

where x_{i-} is the point of departure where the curve starts to separate from segment $i-1$ while x_{i+} is the point of convergence where the curve starts to converge into segment i . Any of the three equations, i.e. Equations (4.10a), (4.10b) and (4.10c) can be used, depending on the

critical information which is available. For example, in the case of the shrinkage curve with two linear segments, x_{i-} is the true shrinkage limit SL' , x_{i+} is the air entry point (AE) while x_i is the apparent shrinkage limit SL . For unimodal SWCC, x_i can be either the air entry value (AEV) or residual matric suction (s_r). To summarize, the parameters in the proposed equation can be obtained graphically as follows:

1. Draw a smooth continuous curve through the data points.
2. Construct linear lines to represent all linear segments of the curve.
3. Determine the slope of each linear line (m_1 to m_n).
4. Determine the matric suctions at the intersection points between two linear lines to obtain x_2 to x_n .
5. Determine the departure and convergence points (x_{i-} and x_{i+}) of the curve between two linear segments, and then use Equation (4.10) to represent the curvature parameters (k_2 to k_n).
6. Substitute the parameters m_i , x_i and k_i into either Equation (4.7) to represent the multi-linear segments curve.

4.3. General equation for shrinkage curve and experimental results

4.3.1. General equation for shrinkage curve

Shrinkage curve equation can be obtained from Equation (4.7) by letting $x = w$, $f(x) = e(w)$, $x_1 = 0$, $f(x_1) = e_{\min}$, and $x_i = w_i$. The shrinkage curve equation for any number of linear segments is given as:

$$e(w) = e_{\min} + m_1 w + \sum_{i=2}^n T_i(w, w_i, m_i, m_{i-1}, k_i) \quad (4.11a)$$

Some literature used moisture ratio u was used instead of w where $u = w.G_s$. Thus, Equation (4.11a) becomes:

$$e(v) = e_{\min} + m_1 v + \sum_{i=2}^n T_i(v, v_i, m_i, m_{i-1}, k_i) \quad (4.11b)$$

where v_i is the moisture ratio at the intersection point between segment i and $i-1$.

Peng and Horn (2013) classified shrinkage curve into six types (A to F) which have been outlined in the literature review. The four shrinkage curve types can be differentiated into four segments shrinkage curve ($n = 4$), three segments shrinkage curve ($n = 3$) and two segments shrinkage curve ($n = 2$) as shown in Figure 2-7.

In the next section, the proposed equation is applied to six types of shrinkage curve based on Peng and Horn (2013) classification which can be regrouped into three types of shrinkage curve according to the number of linear segments. The results are shown by plotting the respective shrinkage curve data along with the shrinkage curve equation, parameters and the value of the coefficient of determination R^2 . The curvature parameter k_i for each segment was determined by using Equation (4.10) where the values of v_{i-} and v_{i+} are substituted to x_{i-} and x_{i+} , respectively. The values of each departure and convergence points are given and plotted as well in the figures with square symbols.

4.3.2. Shrinkage curve types A and B

Shrinkage curve types A and B can be considered as four linear segments curve and therefore requires three T_i in order to represent linear segment $i = 2$ to linear segment $i = 4$. The first segment is the zero shrinkage line with slope $m_1 = 0$, the second segment is the basic shrinkage of the microstructure with slope equals to m_2 , the third segment is the structural shrinkage line with slope equals to m_3 while the fourth segment is the saturation or loading line with slope $m_4 = G_s/S_0$ and S_0 is usually 100%. Four soils which are lixisol from horizons A, B and C from Cornelis et al. (2006a) and dystric gleysol from Peng and Horn (2013) were used to illustrate shrinkage curve types A and B and the results are shown in Figure 4-3. All of the

results indicate a very high R^2 value with the lowest R^2 value equals to 0.995 and the highest R^2 value equals to 0.999.

Figure 4-3 Types A and B shrinkage curves: 4-linear segments

4.3.3. Shrinkage curve types C and D

Shrinkage curve types C and D can be considered as three linear segments curve and therefore requires two T_i in order to represent linear segment $i = 2$ to linear segment $i = 3$. This type of shrinkage curve does not have the zero shrinkage line. Thus, the first segment is the basic shrinkage line of the microstructure, the second segment is the structural shrinkage line and

the third segment is the saturation or the loading line with m_3 equals to G_s/S_0 . Data of shrinkage curve types C and D were obtained from Peng et al. (2007) and Peng and Horn (2013) and are shown in Figure 4-4 with the shrinkage curve which was constructed by using Equation (11b).

Both specimens showed R^2 value of 0.998 for the proposed equation.

Figure 4-4 Types C and D shrinkage curves: 3-linear segments (Peng et al., 2007, Peng and Horn, 2013)

4.3.4. Shrinkage curve types E and F

Shrinkage curve types E and F can be considered as two linear segments curve and therefore only requires one T_i in order to represent linear segment $i = 2$. This type of shrinkage curve is the most common shrinkage curve. For this type, m_1 is equal to 0 as it corresponds to the zero shrinkage line, m_2 is equal to G_s/S_0 as it corresponds to the saturation or loading line, w_2 is the apparent shrinkage limit, w_{2-} is the true shrinkage limit and w_{2+} is the air-entry point which is a water content at air-entry value. For two linear segments shrinkage curve, Equation (4.11) can be simplified by substituting all the parameters such that:

$$e(w) = e_{min} + \frac{G_s}{2S} \left(w + \frac{1}{k_2} \ln \left[\frac{\cosh(k_2(w - SL'))}{\cosh(k_2 SL')} \right] \right) \quad (4.12a)$$

$$e(v) = e_{\min} + \frac{1}{2} S_0 \left(v + \frac{1}{k_2} \ln \left[\frac{\cosh(k_2(v - v_{SL}'))}{\cosh(k_2 v_{SL}')} \right] \right) \quad (4.12b)$$

Figure 4-5 shows four shrinkage curve data from vertisol horizons B1 and B3 (Cornelis et al., 2006a), marine clay from 0-22 cm and 78-120 cm depth (Bronswijk, 1991). The lowest R^2 value is given by marine clay from 78-120 cm depth which is 0.992 while the rest of the data shows a very high R^2 value which is around 0.999 to 1.000.

Figure 4-5 Types E and F shrinkage curves: 2 linear segments

In general, the proposed equation can fit any type of shrinkage curve without using a curve fitting algorithm. The greatest advantage of the proposed equation compare to the other equation is the control over the curvature by using the points of departure and convergence.

4.3.5. Experimental results of shrinkage curve

There are five kaolin specimens which were investigated (Specimen 1 to 5). Specimens 1 to 4 were obtained from kaolin specimens which were consolidated up to 150 kPa. For Specimen 5, the kaolin specimen was first loaded up to net normal stress of 2054 kPa and then unloaded as shown in Figure 4-6a. The shrinkage curves of the five specimens are given in Figure 4-6b.

a) Compression test on Specimen 5

b) Shrinkage curve of Specimen 1 to 5

Figure 4-6 Shrinkage curve of kaolin specimen

Kaolin specimens were subjected to shrinkage test by either drying in air, in a constant relative humidity desiccator or in a pressure plate. Each method has its own advantages. Air drying is the fastest method as it only requires three days to reach almost zero water content and therefore can be used to quickly determine the minimum void ratio e_{min} . Drying in a constant relative humidity desiccator takes more than one month to finish the test but more data can be obtained on a daily basis. The pressure plate gives the matric suction reading and therefore can be used to obtain SWCC as well but is the most expensive in terms of test duration. Figure 4-6b indicates that the three methods produced similar shrinkage curve.

The minimum void ratio, e_{min} , is commonly determined by using shrinkage test. The water content corresponding to e_{min} on the saturation line is referred to as the apparent shrinkage limit SL' . Thus, e_{min} which is obtained from slurry soil is referred to as the natural minimum void ratio e_{min-n} and is unique for different types of soil. Therefore e_{min-n} is usually estimated by using shrinkage limit based on Casagrande method. However, for the case of initial void ratio e_0 at saturated condition smaller than e_{min-n} , it is not possible for e_{min} to remain as e_{min-n} as it is not possible for a soil to swell during drying. Thus, e_{min} must be either equals to e_0 or less than e_0 . The first shrinkage curve was obtained for Specimens 1 to 4. The second shrinkage curve was obtained for Specimen 5 which has $e_0 < e_{min-n}$. Figure 4-6b shows that all of the specimens experienced high volume change due to shrinkage, even for Specimen 5. This observation agrees with the observation of Salager et al. (2010). Thus, this observation shows that e_{min} is affected by the loading history of the soil as well and it is possible for soil with $e_0 < e_{min-n}$ to shrink and to cause $e_{min} < e_0 < e_{min-n}$.

Figure 4-7 shows the shrinkage curve of 12 residual soil specimens from Bukit Timah Granite and all of them are represented with black circle symbol with title "Prior to compression". Residual soil specimens from Bukit Timah Granite is more inhomogeneous than the kaolin specimen. Thus, the shrinkage curve data is very scattered and some of them seems to possess

structural pores. This cause difficulties in determining the real shrinkage curve prior to compression. Some of the specimens which were compressed by using either oedometer or CRS apparatus were then dried as well to obtain the shrinkage curve after compression. A more consistent results are obtained for after compression shrinkage curve which appears to be due to the soil specimen which were subjected to compression under high net normal stress will lost its structural pores. However, a consistent observation can be seen where the value of e_0 is affecting the value of e_{min} .

Figure 4-7 Shrinkage curve of residual soil from Bukit Timah Granite

The reason for such behaviour is a decrease in water content causes an increase in matric suction which is akin to an increase in net normal stress provided that the soil remains fully saturated. Thus, an increase in matric suction causes a change in volume which is commonly referred to as shrinkage. It can be postulated that the value of e_{min} depends on the value of e_0 prior to shrinkage, the change in AEV due to a change in density, shrinkage limit and also the compressibility of the soil.

Salager et al. (2010) proposed that the relationship between e_0 and e_{min} can be assumed to be linear under constant suction condition by using proportionality coefficient $\alpha_s(s)$ which ranges

from 0.79 for dry condition to 1 for saturated condition. The $\alpha_s(s)$ for Specimens 1 to 4 ranges from 0.77 to 0.78 while $\alpha_s(s)$ for Specimen 5 is 0.85. Thus, the lower bound $\alpha_s(s) = 0.79$ proposed by Salager et al. (2010) seems to agree with the $\alpha(s)$ of Specimens 1 to 4 where $e_{\min} = e_{\min-n}$ but not for Specimen 5 which has $e_0 < e_{\min-n}$.

Based on this observation, assuming $\alpha_s(s) = 0.79$ to obtain $e_{\min}$ may lead to oversimplification but using a varying value of $\alpha_s(s)$ may complicate the process in obtaining $\alpha_s(s)$. Therefore, a better method to represent the effect of initial density on the shrinkage curve is needed. A method to describe the shrinkage curve for soil with different initial densities and net normal stresses will be given in the next chapter by considering the stress history of the soil.

4.4. Experimental results and modelling SWCC-w

4.4.1. General SWCC equation

The SWCC of soils may be unimodal or bimodal in nature. Unimodal and bimodal SWCCs can also be considered as multi-linear segments curve where unimodal SWCC consists of three linear segments while bimodal SWCC consists of five linear segments as shown in Figure 4-8. In Figure 4-8, $w(s)$ can be replaced with $\theta(s)$ and $S_r(s)$. However, as SWCC-w is preferred in this study, the proposed equation will use $w(s)$ instead of $\theta(s)$ and $S_r(s)$.

Equation (4.7) is used to represent the SWCC-w by letting $x = \log s$, $x_1 = \log s_1$, $x_i = \log s_i$, $f(x_1) = w_s$ where s is the matric suction of the soil, s_1 is the matric suction which corresponds to w_s , s_i is the matric suction at the intersection point between segment i and segment $i-1$, and w_s is the saturated gravimetric water content. In order to avoid confusion with the curvature parameter for the shrinkage curve, k_i is replaced with c_i for SWCC equation. Substituting all of these parameters into Equation (4.7) gives:

$$w(s) = w_{\text{sat}} + m_1 \log\left(\frac{s}{s_1}\right) + \sum_{i=2}^n T_i (\log s, \log s_i, m_i, m_{i-1}, c_i) \quad (4.13)$$

Similar to the proposed shrinkage curve equation, the curvature parameter c_i can be determined by using Equation (4.10) with $x_{i+} = s_{i+}$ and $x_{i-} = s_{i-}$ where s_{i-} is the matric suction at the point of departure while s_{i+} is the matric suction at the point of convergence. Thus, c_i is given as:

$$c_i = \frac{2}{\log\left(\frac{s_{i+}}{s_{i-}}\right)} \quad (4.14)$$

a) Unimodal SWCC

b) Bimodal SWCC

Figure 4-8 Unimodal and bimodal SWCCs and their parameter

In order to demonstrate the ability of the proposed equation to fit SWCC data, four sets of unimodal SWCC data (D1 to D4) from Dane et al. (1983) which are available in the SoilVision

database and four sets of bimodal SWCC data (S1 to S4) from Satyanaga et al. (2013) were used. All of the parameters were determined graphically, the data points and the curve are given in Figures 4-9 and 4-11. Figures 4-9 and 4-11 show that the proposed equation fits the data very well with R^2 ranging from 0.995 to 1.000.

a) D1 (Soil counter: 10718) and D2 (Soil counter : 10720)

b) D3 (Soil counter : 10721) and D4 (Soil counter : 10722)

Figure 4-9 Evaluation of the proposed equation with data from Dane et al. (1983)

4.4.2. Modelling SWCC-w under different density

For unimodal SWCC-w, the effect of density on the SWCC-w has been proven to affect only the initial drying line. The density of soils can be represented as void ratio e , saturated gravimetric water content w_s or relative density D_r . The density of sand is commonly reference to its D_r while for fine-grained soil it is reference to its e .

However, as SWCC-w used w_s , it is more convenient to convert e and D_r into w_s . The relationship between w_s and e is given as:

$$w_s = \left(\frac{S_r e}{G_s} \right) \quad (4.15)$$

a) S1 and S2

b) S3 and S4

Figure 4-10 Evaluation of the proposed equation with data from Satyanaga et al. (2013)

while relationship between e and D_r is given as:

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \quad (4.16)$$

where $e_{\max}$ is the void ratio which corresponds to the minimum dry density of the coarse-grained soils while $e_{\min}^*$ is the void ratio which corresponds to the maximum dry density of the coarse-grained soils. Rearranging Equation (4.16) gives:

$$e = e_{\max} (1 - D_r) + e_{\min}^* D_r \quad (4.17)$$

By using Equation (4.17) to represent void ratio, Equation (4.15) becomes:

$$w_s = \left(\frac{S_r}{G_s} \right) [e_{\max} (1 - D_r) + e_{\min}^* D_r] \quad (4.18)$$

As the density increases, the value s_2 (the intersection point of the initial drying line and the virgin drying line) also increases. By assuming that the value of m_1 remains the same regardless of the density of soil, the increase in s_2 due to an increase in density is given as:

$$s_{2,f} = s_{2,0} \cdot 10^{\frac{w_{s,0} - w_{s,f}}{m_2 - m_1}} \quad (4.19)$$

where $s_{2,0}$ is the value at the intersection point of initial and virgin drying lines of soil with saturated water content at known density $w_{s,0}$, $s_{2,f}$ is the value of the intersection point of initial and virgin drying lines of soil with saturated water content at the desired density $w_{s,f}$. Figure 4-11 illustrates the effect of Equations (4.18) and (4.19) to the SWCC equation.

Figure 4-11 Illustration of the proposed equation

Thus, modification of Equation (4.13) can be easily done by estimating $w_{s,f}$ using Equations (4.15) and (4.18) and by substituting $s_{2,f}$ in Equation (4.19) into s_2 in Equation (4.13). The equation is then given as:

$$\begin{aligned}
 w(s) = & w_{s,f} - m_1 \log \frac{s}{s_1} \\
 & - T_2(\log s, \log s_1, \log s_2 = \log \left\{ s_{2,0} \cdot 10^{\frac{w_{s,0} - w_{s,f}}{m_2 - m_1}} \right\}, m_2, m_1, k_2) \\
 & - T_3(\log s, \log s_1, \log s_3, m_3, m_2, k_3)
 \end{aligned} \tag{4.20}$$

As shown in Equation (4.20), the modification can be easily done by replacing the parameters in the transition function without changing any of the other parameters.

4.4.3. Modelling SWCC-S

The relationship between degree of saturation as a function of matric suction $S_r(s, e)$ in SWCC-S with gravimetric water content as a function of matric suction and initial void ratio $w(s, e_0)$ in SWCC-w is given as:

$$S_r(s, e) = \frac{w(s, e_0) G_s}{e(w)} \tag{4.21}$$

where $e(w)$ is the void ratio as a function of gravimetric water content w which can be obtained from the shrinkage curve. As shown in Equation (4.21), SWCC-w is a function of both matric suction and initial void ratio while e is a function of w . Thus, by substituting $w(s, e_0)$ from Equation (4.20) and $e(w)$ from Equation (4.12), SWCC-S in Equation (4.21) can be estimated.

4.4.4. Experimental results of SWCC-w

In Figure 4-12, SWCC-w of kaolin specimens with different initial densities are given. The first SWCC-w was obtained by using pressure plate test for matric suctions below 500 kPa and

chilled-mirror dew-point method for matric suction above 500 kPa. The second SWCC-w is SWCC-w of kaolin Specimen 5 which was obtained by using the pressure plate test.

Figure 4-12 Estimation of kaolin SWCC-w under different initial void ratio

In Figure 4-13a, SWCC-w of sand specimens which were compacted to several different initial relative densities were obtained by using Tempe cell. The properties of each specimen are shown in Table 4-1. The results indicate that the SWCC-w always merged into the virgin drying line regardless of its initial density. Therefore, the initial density only affects the initial part of the SWCC-w prior to reaching the virgin drying line. This observation agrees with the observations from Salager et al. (2010).

Such behaviour is advantageous because by knowing the location of the virgin drying line of the SWCC-w, it is possible to estimate the SWCC-w for any initial density by modifying the initial part of the curve. The SWCC-S is controlled by the SWCC-w and the shrinkage curve. Therefore, it is difficult to directly observe the effect of density on SWCC-S as the effect of density appears on both the SWCC-w and the shrinkage curve. In order to avoid the difficulties in obtaining SWCC-S, it is recommended to estimate SWCC-S by using SWCC-w and shrinkage curve.

Table 4-1 Properties of each sand specimens

Name	w_f (%)	w_0 (%)	e_0	D_r (%)
S1	1.74	20.40	0.538	95.50
S2	1.82	26.13	0.689	60.43
S3	1.68	24.69	0.651	69.25
S4	1.79	25.97	0.685	61.37
S5	1.95	24.13	0.636	72.67

a) Estimation of the SWCC- w of the sand specimens under different densities

b) 3D plot of the SWCC- w of the sand specimen

Figure 4-13 Results of proposed equation in estimating SWCC- w for Specimen S2, S3, S4 and S5 by using data from Specimen S1

4.4.5. Validation of the proposed equation to estimate SWCC-w for different densities

In order to show the application of Equation (4.20), sand (Specimens S1 to S5) and kaolin specimens were used. In Figure 4-13, SWCC of Specimen S1, was used to obtain the necessary parameters where the parameters are given in the table in Figure 4-13a while SWCCs of Specimens S2 to S5 were estimated. The curvature parameters (k_2 and k_3) were assumed to be constant regardless of the density of the specimen. It is shown that the proposed equation manages to fit all the data very well and convey all the important properties such as all the SWCCs merged into the virgin drying line. Moreover, the proposed equation only requires one SWCC-w in order to estimate the other SWCC-w. In Figure 4-13b, the 3D plot is given to show a continuous evolution of SWCC-w due to the change in relative density. In Figure 4-12, the estimated SWCC-w for the kaolin specimen is given. It is shown that the proposed equation can also estimate the SWCC-w of fine-grained soils.

Zhou et al. (2014) and Tarantino (2009) equations were compared with the proposed equation as both of them required only one SWCC-w to calibrate their parameters. As both of the equations used degree of saturation, the gravimetric water content for both equation was obtained following the basic soil properties relationship between w , e and specific gravity G_s . For Tarantino (2009) equations, the instantaneous void ratio is replaced with initial void ratio. The comparison of the three equations in estimating the SWCC-w of the sand specimens is given in Figure 4-14. The three equations were calibrated with Specimen S1. The parameters which are obtained from Specimen S1 were then reused to estimate the SWCCs of the sand specimen under different void ratio.

Four sets of SWCC-w data from Salager et al. (2010), Nishimura et al. (2011), Romero et al. (1999) and Gallage et al. (2013) were used to further compare the three equations. Salager et al. (2010) tested five different initial densities (void ratios of 1.01, 0.86, 0.68, 0.55 and 0.44) of compacted clayey silty sand which consists of 72% sand, 18% silts and 10% clay and $G_s = 2.65$.

Nishimura et al. (2011) investigated the SWCC of five different densities Toyoura sand (relative densities were 0.39, 11.6, 54.9, 63 and 73.3) with $G_s = 2.64$.

Figure 4-14 Comparisons of the three equations in estimating SWCC-w of the sand specimens

Table 4-2 Toyoura sand properties

D_r (%)	$\theta_{s\text{-measured}}$	e_0	w_0	$\theta_{s\text{-calculated}}$
0.39	40.6	0.976	36.97	49.39
11.6	41.6	0.934	35.39	48.30
54.9	37.6	0.773	29.28	43.60
63	39.5	0.743	28.13	42.62
73.3	40	0.704	26.68	41.33

Table 4-2 shows that the initial volumetric water content θ_s which were measured by Nishimura et al. (2011) do not seem to be consistent as an increase in relative density should cause a decrease in θ_s . Therefore, θ_s was recalculated by using the relative density of the specimens and the subsequent volumetric water content at different matric suction θ_w were

calculated based on the change in volumetric water content. From the θ_w and e_0 , the gravimetric water content can be calculated and then plotted.

Romero et al. (1999) investigated the SWCC of statically compacted natural Boom clay which has liquid limit of 56% and plastic limit of 29% with 50% of the particles less than $2\mu\text{m}$. The compacted clay were of two different initial dry densities which are 13.7 kN/m^3 and 16.7 kN/m^3 . Thus, the two specimens were named after their initial dry densities which are S13.7 and S16.7, respectively.

Gallage et al. (2013) investigated the SWCCs of Edosaki sand (E12.2 and E13.5) with $G_s = 2.75$ and Chiba soil (C12.2 and C13.5) with $G_s = 2.72$. Each soil was compacted at two different densities which are 12.2 kN/m^3 (E12.2 and C12.2) and 13.5 kN/m^3 (E13.5 and C13.5). As the original data were given in terms of θ , w is obtained by assuming that there was no volume change when the soil specimens were dried with e_0 determined from the dry density of the soil specimen. The comparisons of the SWCC- w produced by the three equations along with their parameters are given in Figures 4-15 to 4-19.

The comparison of all estimated SWCC- w data points with the measured SWCC- w data points is given in Figure 4-20. Figure 4-20 shows that the proposed equation in most of the cases is closer to the 1 by 1 line compared to the other equations which indicates a better accuracy. Table 4-3 shows the RMSE value from each equations. The RMSE is defined as:

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^n (w_{\text{measured-}i} - w_{\text{estimated-}i})^2}{n}} \quad (4.22)$$

where i is the i^{th} data point, n is the number of data points for all the SWCC (calibrated and estimated), $w_{\text{measured-}i}$ is the measured water content at the i^{th} data point, $w_{\text{estimated-}i}$ is the estimated water content at the i^{th} data point. The smaller the value of RMSE, the better the agreement between estimated and measured data.

a) Proposed equation

b) Zhou et al. (2014) equation

c) Tarantino (2009) equation

Figure 4-15 Estimation of SWCC for Clayey silty sand from Salager et al. (2011)

a) Proposed equation

b) Zhou et al. (2014) equation

c) Tarantino (2009) equation

Figure 4-16 Estimation of SWCC-w of Toyoura sand Toyoura sand from Nishimura et al. (2011)

a) Proposed equation

b) Zhou et al. (2014) equation

c) Tarantino (2009) equation

Figure 4-17 Estimation of SWCC-w of Boom clay from Romero et al. (1999)

a) Proposed equation

b) Zhou et al. (2014) equation

c) Tarantino (2009) equation

Figure 4-18 Estimation of SWCC- w of Edosaki sand from Gallage et al. (2013)

a) Proposed equation

b) Zhou et al. (2014) equation

c) Tarantino (2009) equation

Figure 4-19 Estimation of SWCC-w of Chiba soil from Gallage et al. (2013)

a) Sand specimen

b) Clayey silty sand (Salager et al., 2010)

c) Toyoura sand (Nishimura et al., 2011)

d) Boom clay (Romero et al., 1999)

e) Edosaki sand (Gallage et al., 2013)

f) Chiba soil (Gallage et al., 2013)

Figure 4-20 Comparison between proposed equation and the other equations

Table 4-3 RMSE comparison between proposed equation and the other equations

Specimen	RMSE		
	Zhou et al. (2014)	Tarantino (2009)	proposed equation
Sand specimen	3.175	0.957	0.500
Clayey silty sand (Salager et al., 2010)	1.893	1.118	1.515
Toyoura sand (Nishimura et al., 2011)	4.047	2.393	1.797
Boom clay (Romero et al., 1999)	1.896	1.288	0.944
Edosaki sand (Gallage et al., 2013)	2.012	1.908	1.302
Chiba soil (Gallage et al., 2013)	2.991	1.645	0.965

It is shown that the proposed equation always merge with the virgin drying line. On the other hand, Zhou et al. (2014) equation does not guarantee this uniqueness and gives the worst performance compare to the other equations. Tarantino (2009) equation has a good feature as the SWCCs under different densities always merge into the virgin drying line. However, Tarantino (2009) equation sometimes has difficulties in modelling the residual line of SWCCs such as for the sand specimens (Figure 4-14d), Edosaki sand (Figure 4-18c), and Chiba soil (Figure 4-19c). Based on this evaluation, it appears that Tarantino (2009) equation is good in estimating SWCC-w of fine grained soils where there is a smooth transition between each segments. However, for soil with a drastic change in slopes between each segments, Tarantino (2009) equation tend to give incorrect residual line which is the reason why Tarantino (2009) equation seems unable to give a good prediction for small range of water content. The proposed equation on the other hand gives the best performances as it has full control over the SWCCs on every segment. Quantitative evaluation based on the RMSE values also indicate that the proposed equation in most of the cases gave the best performances compare to the other equations. Thus, the proposed equation is suitable for any type of soils.

4.4.6. Modelling SWCC-S for coarse grained soils under different initial densities

For coarse grained soils, $e(w)$ is constant and is equal to e_0 as coarse-grained soils does not normally shrink due to drying. Therefore, SWCC-S is only dependent on the SWCC-w and e_0 . In

this section, the example of how to model SWCC-S for coarse grained soils is given by using the sand specimen.

Figure 4-21 SWCC-S of the sand specimen

By substituting Equation (4.20) into Equation (4.21) and letting $e(w) = e_0$, the SWCC-S of the sand specimen is determined and is given in Figure 4-21. Figure 4-21 shows that the proposed equation can fit the SWCC-S data very well. Figure 4-21 shows that the SWCC-S of soil with lower density is located lower than SWCC-s of soil with higher density. Thus, soil with higher density requires higher matric suction to have the same degree of saturation with soil with lower density. Thus, SWCC-S of soil with lower density have lower AEV than SWCC-S of soil with higher density.

4.5. Summary

In this chapter, the behaviour of SWCC-w and shrinkage curve under different initial densities have been observed for coarse-grained and fine-grained soils. The SWCC-w has a consistent behaviour for any type of soils where SWCC-w for different initial densities will eventually merge into the virgin drying line and residual line. Such unique behaviour is advantageous in

modelling SWCC-w. However, shrinkage curve behaviour under different initial densities is more complicated as it shows that for soils with initial void ratio lower than its natural minimum void ratio, the soil shrinks as the matric suction acts as a net normal stress. Salager et al. (2010) proposed to use proportionality coefficient to estimate the ratio between initial void ratio and minimum void ratio which is equal to 0.79. Based on the tests which have been carried out in this study, the proportionality constant is not constant for different initial void ratios especially when the initial void ratio is below the natural minimum void ratio. A method to estimate the e_{min} is proposed in the next chapter by considering the loading history of the soil specimen.

In this chapter, a new equation to represent multi-linear segments curve has been proposed. The proposed equation can be used to curve fit both SWCC-w and shrinkage curve and has been extended to estimate SWCCs-w for soils with different initial densities. Unfortunately, as the effect of density on the bimodal SWCC-w remains unclear, the proposed method has not been extended to estimate the effect of density on the bimodal SWCC-w.

By using the proposed equation, it is also possible to estimate the SWCC-S for soils with different initial densities for coarse and fine-grained soils provided that the relationship between void ratio and gravimetric water content can be obtained. The estimation of SWCC-S for coarse-grained soils has been described in this chapter as the coarse-grained soil does not shrink. However, for fine-grained soils, the shrinkage curve under different initial densities is required. In the next chapter, it will be demonstrated that by estimating the value of e_{min} , it is possible to construct a shrinkage curve under different net normal stresses.

Visual basic codes to use the proposed equation to fit shrinkage curve and SWCC for any number of linear segments are provided in the APPENDIX B.

Chapter 5. Constant water content compression tests

5.1. Introduction

In this chapter, the constant water content compression test results which were obtained are presented. There are three behaviour which were investigated:

1. Constant water content compression behaviour of unsaturated soil under different initial water contents,
2. Wetting-induced volume change of unsaturated soil under different net normal stresses, and
3. The effect of strain rate on the compressibility and pore-water pressure response of unsaturated soil under constant water content condition.

In order to investigate the compression and wetting-induced volume change of unsaturated soils, saturated compression test (SCT), constant water content compression test with and without pore-water pressure measurement (CWC-P and CWC, respectively) and inundation test were conducted. The SCT was used to obtain the normal compression line (NCL) which is defined as the compression line under the fully saturated condition for normally consolidated soils with slope equals to C_c and other compression properties such as the slope of unloading line at the saturated condition $C_{r,s}$ and the saturated preconsolidation pressure $P_{c,s}$. The NCL is useful as a reference for the unsaturated compression line (UCL). In this study, UCL is defined as the compression line of the unsaturated soils after its apparent preconsolidation pressure.

Based on the observations, there are three different unsaturated compression behaviour:

1. The compression behaviour of unsaturated soil which has reached the saturated condition prior to the NCL.

2. The compression behaviour of unsaturated soil which remains unsaturated beyond the NCL but then slowly merges into the NCL due to significant change in degree of saturation.
3. The compression behaviour of unsaturated soil which has very low degree of saturation and do not show significant change in its degree of saturation under compression.

When the unsaturated soil becomes saturated prior to reaching NCL, it is considered as case I. For the cases where the soil specimen remains unsaturated after it reached the NCL is referred to as cases IIa and IIb when there is a significant and insignificant change in degree of saturation beyond the NCL, respectively. For the sake of clarity, all of the unsaturated compression test data were grouped and presented according to the three observed behaviour.

In order to investigate the effect of loading history on the wetting-induced behaviour of unsaturated soils, most of the specimens has been loaded to a specific net normal stress and void ratio prior to inundation. Thus letters A, B, C etc. are used to indicate the test paths of the specimens.

Investigation on the effects of different rates of strain and incremental loading on the unsaturated soil specimen's compressibility and pore-water pressure response are presented as well. The investigation on strain rate is helpful not only to understand the unsaturated soil compression behaviour in terms of its compressibility and pore-water pressure response but also to determine the duration of the experiment. For example, based on the tests which have been carried out in this study, the duration which is required to increase the net normal stress from 1 kPa to 2000 kPa is different for different strain rates. A strain rate of 0.1 mm/min requires 20 minutes, a strain rate of 0.01 mm/min requires one day, while a strain rate of 0.001 mm/min requires a few days. Thus, it is advantageous to use higher strain rates but it is desired to use a strain rate which does not significantly affect the interpretation of the soil behaviour.

The measurement of matric suction under constant water content loading requires a high-capacity tensiometer (HCT) especially when the matric suction which is measured is higher than 100 kPa. In this chapter, the interaction between HCT and the unsaturated soil specimen is also investigated. There are four main aspects of the interaction which are investigated related to the CWC-P in this study:

1. Equilibrium time of HCT
2. Pore-water pressure response due to incremental loading
3. Pore-water pressure response due to different constant rate of strain loading
4. Pore-water pressure response due to unloading

The effect of strain rates on the compressibility of unsaturated soils are investigated using results from oedometer and CRS apparatuses. Based on the pore-water pressure response and the compression behaviour of the unsaturated soil specimens, an appropriate strain rate is recommended.

5.2. Constant water content compression behaviour of unsaturated soils under different initial water content

5.2.1. Results from the saturated compression test (SCT)

The saturated compression curves for the kaolin specimens (Figure 5-1a) were obtained from four oedometer tests (Specimens KODSat1 to KODSat4) and two CRS tests (Specimens Kw62-0.002 and Kw62-0.005). For the residual soil specimen from Bukit Timah Granite (Figure 5-1b), two oedometer tests (BTODSat1 and BTODSat2) were used. The parameters of the compression curve for both kaolin and residual soil specimens are summarized in Table 5-1.

a) Kaolin

b) Residual soil from Bukit Timah Granite

Figure 5-1 Results of SCT

The saturated compression curves for the six kaolin specimens and the two residual soil specimens from oedometer and CRS tests agree very well. The normal compression line (NCL) is idealized as follows:

$$e_{NCL} = e_{N,s} - C_c \log P_{c,s} \quad (5.1)$$

where e_{NCL} is the void ratio along the NCL and $e_{N,s}$ is the void ratio of the NCL when $P_{c,s}$ is equal to 1.

Table 5-1 Summary of saturated compression test

Specimen	$w_0(\%)$	C_c	$C_{r,s-1}$	$C_{r,s-2}$	$P_{c,s}$ (kPa)
KOdSat1	58.32	0.582	0.064	0.120	157
KOdSat2	61.18	0.592	0.076	0.131	165
KOdSat3	61.22	0.546	0.093	-	146
KOdSat4	58.55	0.586	0.086	0.132	158
KCRS-0.002	61.96	0.604	0.102	0.141	139
KCRS-0.005	62.83	0.615	0.139		155
BTOdSat1	34.21	0.264	0.045	-	92
BTOdSat2	34.13	0.279	0.045	0.049	90

5.2.2. Compression curve of unsaturated soils which reached saturation prior to NCL (case I)

Table 5-2 to 5-5 summarize the test specimens which reached full saturation prior to NCL and thus yielded at the NCL (case I). Table 5-2 to 5-4 summarize the test result on kaolin specimens while Table 5-5 summarizes the test results on residual soil specimens. In Table 5-2 to 5-5, w_0 is the initial water content of the specimen, C_c is the slope of the NCL, C_r is the slope of unloading line and P_c is the preconsolidation pressure.

For the sake of clarity, kaolin specimens with water content between 53 to 60% are plotted in Figure 5-2a, kaolin specimens with water content between 47 to 51% are plotted in Figure 5-2b while kaolin specimens with water content between 37 to 42% are plotted in Figure 5-2c. The results from CRS tests are represented with dashed lines while the results from the oedometer tests are represented with lines and symbols. Figure 5-2 shows a consistent behaviour where all of the specimens yielded around the NCL and the unsaturated compression curve merged into the NCL.

Table 5-2 Summary of test on kaolin specimens with w ranging from 53 to 60% for case I

Specimen	w₀(%)	C_c	C_r	P_c (kPa)
Kw60s40	60.0	0.618	0.143	131
Kw57s110	57.6	0.627	0.071	162
Kw57s81	56.6	0.616	0.083	174
Kw57-60-0.003	57.7	0.570	-	137
Kw53-155-0.005	53.6	0.590	0.161	273

Table 5-3 Summary of test on kaolin specimens with w ranging from 47-51% for case I

Specimen	w₀(%)	C_c	C_r	P_c (kPa)
Kw48s287	48.6	0.607	0.075	319
Kw47s252	47.5	0.609	0.086	348
Kw47.5-310-0.001	47.5	0.590	0.061	409
Kw48.37-345-0.001	48.2	0.600	-	387
Kw48.87-387-0.001	48.0	0.595	-	430
Kw47.87-335-0.01	49.1	0.580	-	326
Kw48-322-0.01	48.4	0.590	0.110	337
Kw48-363-0.01	48.8	0.610	0.111	281
Kw49-225-0.01	47.8	0.590	0.123	238
Kw49-300-0.01	49.3	0.580	0.116	308
Kw47.3-363-0.1	47.3	0.590	-	411
Kw47-450-0.1	47.5	0.580	-	348
Kw47.6-363-0.1	47.6	0.580	-	366
Kw48-271-0.1	48.2	0.600	0.102	349
Kw48-400-0.1	48.2	0.590	0.178	227
Kw47-0.003	47.5	0.580	-	402
Kw51-500-0.003	51.0	0.590	-	316
Kw48.54-450-0.005	48.5	0.590	-	384
Kw49.2-275-0.005	49.2	0.610	-	221

Table 5-4 Summary of test on kaolin specimens with w ranging from 37-42% for case I

Specimen	w₀(%)	C_c	C_r	P_c (kPa)
Kw42s347	42.4	0.624	0.109	593
Kw37s399	36.7	0.740	0.058	819
Kw37-362-0.01	37.8	0.590	0.078	555
Kw37-465-0.1	37.6	0.600	0.075	416
Kw37-429-0.001	37.8	0.590	0.081	436

a) w from 53-60%

b) w from 47-51%

c) w from 37-42%

Figure 5-2 Kaolin specimens fall under case I

Table 5-5 Summary of test on residual soil specimens for case I

Specimen	w	C_c	C_{r-1}	C_{r-2}	P_c
BTw26s332	26.4	0.340	0.068	-	391.653
BTw26-321-0.001	26.5	0.337	0.065	0.069	494.2078
BTw26-251-0.01	27.5	0.494	0.057	0.135	482.9604
BTw26-300-0.1	25.8	0.405	0.052	0.052	463.1421
BTw20s902	20.3	0.322	0.034	-	692.7363
BTw18-975-0.01	18.9	0.481	0.020	0.036	591.6074
BTw18-1039-0.1	19.3	0.368	0.029	0.039	545.477
BTw18-0.001	19.9	0.416	0.044	0.059	538.1596

Similar to kaolin specimens, residual soil specimens with w around 26% are plotted in Figure 5-3a while residual soil specimens with w around 18% are plotted in Figure 5-3b. Residual soil specimens show more consistent results where all of the unsaturated compression curves merge into the NCL. It is clear from the data that a decrease in the initial void ratio e_0 causes an increase in the $P_{c,s}$. An increase in the $P_{c,s}$ due to a decrease in the e_0 is illustrated in Figure 5-4. In Figure 5-4, an increase in $P_{c,s}$ is analogous to a saturated specimen which has been subjected to applied stress $P_{c,s}$ and then unloaded to $e_{0,d}$. Thus, as saturated soils must yield at the NCL, unsaturated soils which have become saturated prior to the NCL must also yield at the NCL with net normal stress equals to $P_{c,s}$.

5.2.3. Compression curve of unsaturated soil which remains unsaturated beyond the NCL and merges with the NCL at high net normal stress (case IIa)

Some observations showed that the slope of the compression curve increases with respect to net normal stress and the unsaturated compression curve eventually merges with the NCL at high net normal stress (Toll, 1990, Alonso, 1993, Wheeler and Sivakumar, 1995, Honda, 2000, Sivakumar and Wheeler, 2000, Futai et al., 2002, Silva et al., 2002).

a) w around 26%

b) w around 18%

Figure 5-3 Residual soil specimens fall under case I

Figure 5-4 Illustration on effect of initial void ratio on the preconsolidation pressure

Three kaolin specimens (Kw30s556, Kw30s468, Kw23s895) were tested by using the oedometer apparatus and the test results are shown in Figure 5-5. Specimens Kw30s556 and Kw23s895 were inundated at 1630 kPa net normal stress in order to observe the position of their respective UCLs. The test results of the three kaolin specimens are given in Table 5-6.

Specimen Kw30s556 was not loaded under load increment ratio (LIR) of 1 but subjected to equal load increments (50 kPa for each increment) until the soil was inundated. The purpose of using equal load increments is to have smaller changes in volume and pore-water pressure during loading and unloading and to avoid the possibility of the pore-water pressure becoming positive. Compression curve of soil is affected by the LIR used in the oedometer test (ASTM-D2435/D2435M, 2011). For saturated soil, LIR of 1 is commonly used. Therefore, The effect of LIR on the unsaturated compression curve can also be obtained by comparing Specimens Kw30s556 and Kw30s468 as both of them have similar water content and void ratio. The difference between the unsaturated compression curves of Specimen Kw30s556 which used equal load increment appears to be slightly above the Kw30s468. However, the difference seems to be very small and therefore can be considered negligible.

Table 5-6 Summary of test results on kaolin specimens which remains unsaturated on reaching the NCL (case IIa)

Specimen	$w_0(\%)$	C_c	C_{r-1}	C_{r-2}	P_c
Kw30s556	30.8	0.790	0.015	0.030	629
Kw30s468	29.6	0.603	0.079	-	627
Kw23s895	22.9	0.568	0.007	0.035	651

The three specimens appear to have a very similar unsaturated compression curve (Figure 5-5a). The three specimens show that the unsaturated compression curve slowly merges into the NCL at high net normal stress. Specimen Kw30s468 merges into the NCL at net normal stress equals to 3260 kPa with degree of saturation $S_r = 100\%$. Thus, it is difficult to determine

the slope of the unsaturated compression curve after its preconsolidation pressure as it slowly merges into the NCL.

Figure 5-5 Compression behaviour of unsaturated kaolin specimens case IIa

Under load, degree of saturation increased and the matric suction reduces. Such a phenomenon is referred as mechanical wetting as it is similar to the wetting process by inundation except that this is due to a mechanical process (Tarantino, 2009) and causes additional settlement similar to collapse due to inundation. The additional settlement due to mechanical wetting causes the slope of the compression curve to increase until the unsaturated compression curve merges with the NCL.

5.2.4. Compression of unsaturated soils with insignificant change in degree of saturation (case IIb)

The UCL of unsaturated soils with a very low initial water content showed insignificant change in degree of saturation during loading. This condition is very close to the constant degree of saturation condition without any control of the matric suction and thus has less uncertainty compared to the constant degree of saturation test proposed by Burton et al. (2016). Table 5-7 summarizes the soil specimens which have very low initial water content. As the soil specimens were subjected to several unloading stages before and after inundation with water, the slope of the unloading line is given in brackets in Table 5-7 with the first value corresponding to the first unloading line and the second value corresponding to the second unloading line. The slope of the unloading line prior to inundation is $C_{r,d}$ and the slope of the unloading line after inundation is $C_{r,s}$. The C_c of each specimen is not given in Table 5-7 as each specimen was subjected to a different load path in order to observe its wetting-induced behaviour. Specimens Kw1 and Kw6 were loaded under constant water content condition as shown in Figure 5-6. Two loading-unloading paths (A-B-C, C-D-E) were carried out in order to obtain $C_{r,d}$ of the specimens. Both specimens were loaded to F in order to obtain the slope of the UCL. Specimen Kw6 was then inundated to observe whether the void ratio after inundation falls onto the NCL.

In Figure 5-6a, the UCL was constructed by using the test results of Specimen Kw1. The C_c of specimen Kw1 is 0.583 which is within the range of the C_c of saturated specimens that are shown in Table 5-1. Specimen Kw1 was chosen as it has the lowest initial water content and therefore represent the upper boundary of the UCL for the kaolin specimens tested.

The UCL appears to be parallel with the NCL provided that the C_c was obtained from the compression curve at higher net normal stresses beyond its apparent preconsolidation pressure. If the C_c was obtained from the compression curve at net normal stresses around or

less than its apparent pre-consolidation pressure, the C_c would be significantly smaller than the C_c of the saturated soils as observed by Alonso et al. (1990).

In Figure 5-6b, the change in degree of saturation of Specimen Kw1 throughout the entire loading phase did not exceed 2%. For Specimen Kw6, the unsaturated compression curve appears to be identical to that of specimen Kw1. However, the change in degree of saturation is higher (around 8 %) and it is observed that the void ratio corresponding to the net normal stress of 3260 kPa has the highest change in degree of saturation (about 5%) which could explain why the void ratio deviates slightly from the UCL at net normal stress of 3260 kPa (Figure 5-6a). Specimen Kw6 was then inundated and the void ratio after inundation is located on the NCL at G in Figure 5-6a.

Thus, from this observation it can be postulated that under relatively constant degree of saturation, the UCL is parallel to the NCL or in other words, the C_c remains constant. This hypothesis is supported by the observation from Mun and McCartney (2015) which showed that regardless of the matric suction of the specimens, when the degree of saturation of the specimen equals to 1, the UCL merges with the NCL.

5.2.5. Slope of the unloading line at unsaturated condition

The slope of the unloading line under constant water content condition $C_{r,d}$ was observed for all of the unsaturated specimens. It was found that the value of $C_{r,d}$ is significantly smaller than $C_{r,s}$ provided that the soil was unloaded under constant water content condition. This observation is different from what has been commonly assumed in most of the previously proposed models (Alonso et al., 1990, Wheeler and Sivakumar, 1995, Pham, 2005). In most of the previously proposed models, constant matric suction test is commonly used to obtain the $C_{r,d}$. When the soil is unloaded under constant water content condition, the matric suction of the soil increases due to dilation.

Table 5-7 Summary of specimens with very low initial water content (case IIb)

Name	w_0 (%)	$P_{c,d}$ (kPa)	e_0	C_{r-s}	C_{r-d}
Kw1	1.5	787	1.306	-	(0.019, 0.047)
Kw5	5.2	596	1.295	0.118	(0.012, 0.041)
Kw6	6.4	844	1.281	-	(0.011, 0.044)
Kw10	9.7	585	1.264	-	(0.015, 0.049)
Kw1.5-0.1	1.5	939	1.353	0.167	(0.069, 0.088)
Kw1.2-0.01	1.2	840	1.343	0.181	(0.068, 0.094)
Kw1.9-0.001	1.3	899	1.308	0.167	(0.067, 0.084)

a) Unsaturated compression curves

b) Change in degree of saturation due to the loading and unloading

Figure 5-6 Compression behaviour of Specimens Kw1 and Kw6 (case IIb)

Thus, in order to maintain the constant matric suction condition, water enters the soil as the soil dilates which then increase the value of $C_{r,d}$ to be similar to $C_{r,s}$. Therefore for unsaturated soil, the slope of the unloading line is $C_{r,d}$ when there is no increase in water content of the soil while the slope of the unloading line can change from $C_{r,d}$ to $C_{r,s}$ as the water content of the soil increases.

It is also shown that the slope of the unloading line which was obtained at net normal stress lower than the apparent preconsolidation pressure is much smaller than the slope of the unloading at net normal stress higher than the apparent preconsolidation pressure similar to the behaviour of saturated soil which is loaded prior to its preconsolidation pressure (Becker et al., 1987, Wang and Frost, 2004).

5.3. Wetting induced volume change of unsaturated soils under different net normal stresses

5.3.1. Wetting-induced behaviour of unsaturated soil at low net normal stress

In order to study the wetting-induced behaviour of unsaturated soils, specimens Kw5, Kw6, Kw9, Kw1.5-0.1, Kw1.2-0.01 and Kw1.9-0.001 were used. Specimen Kw5 (Figure 5-7) was loaded and unloaded following path A-B-C to obtain the first $C_{r,d}$. At C, Specimen Kw5 was then loaded to D such that the net normal stress at D became the apparent preconsolidation pressure $P_{c,d}$ and the corresponding void ratio was the preconsolidation void ratio e_c . Thus, the loading history of Specimen Kw5 is precisely determined.

Specimen Kw5 was then unloaded to E and then inundated with water. The inundation caused the soil to swell up to F. Specimen Kw5 was then loaded back to G and then unloaded to H to obtain $C_{r,s}$. Specimen Kw5 was then loaded to J to check whether it is located on the NCL. It was observed when the swelling line with slope equals to $C_{r,s}$ was constructed at point F for Specimen Kw5 that it passes through the NCL at void ratio equals to e_c .

Figure 5-7 Constant water content compression and free swell tests of Specimen Kw5

Based on this observation, it appears that the preconsolidation void ratios e_c before and after inundation are equal for swelling. Such a relationship can be mathematically written as:

$$e_{c,d}(P_{c,d}) = e_{c,s}(P_{c,s}) = e_c \quad (5.2)$$

where $e_{c,d}$ is the preconsolidation void ratio before inundation, $e_{c,s}$ is the void ratio after inundation, and $P_{c,d}$ is the apparent preconsolidation pressure or net normal stress which is located on the UCL. In order to further verify the uniqueness of e_c , a constant volume test at high net normal stress was conducted.

Constant volume test was conducted with Specimen Kw1.2-0.01 as shown in Figure 5-8. Specimen Kw1.2-0.01 was loaded and unloaded twice (A-B-C, C-D-E) to obtain two values of $C_{r,d}$. It was then loaded to D and then unloaded to E to obtain the reloading line. Specimen Kw1.2-0.01 was then inundated at E. This caused the net normal stress to increase from E to F and the net normal stress corresponding to F is referred to as the constant volume swelling pressure P_s . After inundation, Specimen Kw1.2-0.01 was then unloaded to G and then loaded

to H and then unloaded to J in order to obtain $C_{r,s}$. It is noted that the reloading line merges with the NCL at e_c . Thus, swelling due to inundation under constant net normal stress or constant volume condition does not change the value of e_c which agrees with the observation for Specimen Kw5.

The void ratio prior to inundation is $e_{d,i}$ while the void ratio on the unloading line after inundation is $e_{s,i}$. A swelling line with slope $C_{r,s}$ is constructed to pass through e_c on the saturated NCL ($P_{c,s}, e_c$). This line appears to follow the saturated unloading line. The observations on Specimen Kw5 and Kw1.2-0.01 suggest that the swelling line which pass through e_c at NCL determines the amount of swelling and the swelling pressure. The loading path G-H also indicates that after inundation, the compression curve merges with NCL at e_c .

5.3.2. Wetting-induced behaviour of unsaturated soil at high net normal stress

Figure 5-9 shows the compression of Specimen Kw1.5-0.1. Specimen Kw1.5-0.1 was loaded and unloaded twice (A-B-C, C-D-E) to obtain $C_{r,d}$. Figure 5-9 shows that B and D are located on the UCL of Specimen Kw1 which are expected as both points have similar water content and

thus indicates that the net normal stress at D is the current $P_{c,d}$ of Specimen Kw1.5-0.1. Inundation was carried out at D and the load cell recorded a reduction in the net normal stress until it reached the NCL. These observations further reinforce the validity of Equation (5.2) where the values of e_c before and after inundation are the same. Further compression from F to G caused Specimen Kw1.5-0.1 to follow the NCL. Specimen Kw9 (Figure 5-10) was loaded under a similar path as Specimens Kw1 and Kw6 except that Specimen Kw9 was inundated at D and then loaded to G.

As Specimen Kw9 has a higher initial degree of saturation than Specimen Kw1, its UCL is slightly below the UCL of Specimen Kw1. A consistent behaviour was also observed between Specimens Kw6 and Kw9 where inundation at sufficiently high net normal stress caused collapse settlement and the void ratio decreases from the UCL until it reached the NCL due to plastic deformation.

Figure 5-9 Specimen Kw1.5-0.1

Since the soil was inundated at the UCL, the inundation pressure P_i is equal to $P_{c,d}$, as it is the highest net normal stress which has been applied on the specimen prior to inundation. Thus,

the inundation pressure at the UCL becomes the saturated preconsolidation pressure $P_{c,s}$. Therefore for the collapse phenomenon, the relationship of the net normal stresses can be mathematically written as:

$$P_{c,d}(e_{c,d}) = P_{c,s}(e_{c,s}) = P_i \quad (5.3)$$

The evolution of $e_{c,s}$ to $e_{c,d}$ in Equation (5.3) is consistent with Equation (5.2) where inundation caused a reduction in net normal stress from $P_{c,d}$ to $P_{c,s}$ under constant volume condition. However, as P_i is higher than $P_{c,s}$, it causes additional settlement until $P_{c,s}$ equals to P_i following the NCL.

Specimen Kw1.9-0.001 (Figure 5-11) was loaded in a similar manner as Kw1.2-0.01 except that it was inundated near the NCL. Specimen Kw1.9-0.001 was loaded and unloaded twice (A-B-C, C-D-E) to obtain $C_{r,d}$. After the second unloading, the loading history of Specimen Kw1.9-0.001 indicates that $P_{c,d}$ and e_c are located at D. The soil specimen was then reloaded to F where F was chosen such that it is only slightly below the NCL in order to observe the wetting-induced net normal stress change in the elastic region. Specimen Kw1.9-0.001 showed net normal stress relaxation which caused the net normal stress to move from F to G. Specimen Kw1.9-0.001 was then unloaded to H, and then reloaded to J in order to obtain C_c .

Figure 5-11 shows that G is located on the reloading line which indicates that if the specimen was inundated at net normal stress higher than the reloading line, the relaxation causes the reduction in the net normal stress until it reaches the reloading line instead of the unloading line. Such behaviour is beneficial as it causes less net normal stress reduction which indicates less collapse settlement under the free swell condition compared to if the collapse is to take place where void ratio reduces to the unloading line. Therefore, for simplicity and conservative analysis, it is preferred to use the unloading line instead of the reloading line. Specimen Kw1.9-0.001 was then unloaded to K in order to obtain the $C_{r,s}$. Swelling line with a slope $C_{r,s}$

was then constructed to pass through e_c on the NCL ($P_{c,s}, e_c$). Similar to the observation for specimen Kw1.2-0.01, it was found that the swelling line is able to trace the saturated unloading line.

a) Unsaturated compression curve of Specimen Kw9

b) Changing degree of saturation due to the loading and unloading of Specimen Kw9

Figure 5-10 Compression behaviour of Specimen Kw9

Figure 5-11 Specimen Kw1.9-0.001

Based on the observations above, it can be concluded that the intersection between constant water content unloading line and the swelling line determines the transition pressure where inundation pressure which is less than the transition pressure will generate swelling and swelling pressure while inundation pressure which is higher than the transition pressure will cause collapse or relaxation.

5.4. The effect of strain rate under constant water content condition on the compressibility and pore-water pressure response of unsaturated soil

5.4.1. Equilibration between high-capacity tensiometer and unsaturated soils

For each constant water content test with pore-water pressure measurement, the specimen was placed on the top of the HCT. The equilibrium between HCT and the unsaturated soil specimen can only be achieved once the water inside the HCT is connected with the water phase inside the soil specimen. For the kaolin specimen, the HAE disk is slightly moist with a very thin layer of water.

However for the other soil specimens, kaolin paste was used to cover the HAE disk in order to improve the contact condition between the soil specimen and the HCT. Typical equilibration processes types (A, B and C) for kaolin specimen and HCT are shown Figure 5-12.

In most of the cases, the equilibration of the kaolin specimen and the HCT falls into either type A or B which are shown in Figure 5-12a and b, respectively. However, for soils which has an extremely high matric suction such as Specimen Kw23s895 (Figure 5-12c), the negative pore-water pressure keeps decreasing and thus give type C pore-water pressure response.

One of the reasons is that Specimen Kw23s895 has very high matric suction that exceeded the capacity of the HCT. In order to avoid cavitation in the HCT, the equilibration process was stopped and the specimen was further loaded to increase its pore-water pressure.

For the residual soil from Bukit Timah Granite, kaolin paste was used to cover the HAE disk of the HCT. There are three noticeable advantages in using the kaolin paste:

1. It delays the evaporation from the HCT which provides more time for the specimen to be set up.
2. A good contact condition can be easily achieved as the kaolin paste smoothens the rough surface of the specimen.

Very good responses for almost every specimen were obtained except when the specimen's matric suction was higher than the capacity of the HCT as shown in Figure 5-13. The equilibrium time for each specimen also appears to be similar which was around six minutes as shown in Figure 5-13a. Unfortunately, the HCT was cavitating when measuring the pore-water pressure of Specimen BTw15-0.001 and thus, no further pore-water pressure reading can be obtained.

a) Type A

b) Type B

c) Type C

Figure 5-12 Typical equilibration processes for the kaolin specimens (without kaolin paste)

a) Type A

b) Type C

Figure 5-13 Equilibration process for the residual soil specimens (with kaolin paste)

5.4.2. Change in pore-water pressure due to incremental loading under constant water content condition

The results of the CWC-P tests for both kaolin and residual soil specimens are shown in Figure 5-14. Typical results can be observed where under constant water content condition, loading increases the pore-water pressure (mechanical wetting) while unloading decreases the pore-water pressure (mechanical drying).

Figure 5-14 Results for CWC-P test

An increase in pore-water pressure due to mechanical wetting seems to follow a typical trend. In Figure 5-15, Kaolin specimens Kw30s556 and Kw23s895 were unloaded several times prior to their pore-water pressure reaching a positive value. For Kw30s556, a small constant load increment was used in order to obtain a more detailed change in pore-water pressure. However, it appears that when the net normal stress reached about 800 kPa, the HCT no longer works properly as it no longer follows the typical trend which were observed with the other kaolin specimens. This could be due to the prolonged test duration and several unloading stages which were carried. When the loading continued up to 1630 kPa, the HCT suddenly cavitared which confirmed that the HCT gave incorrect reading. However, it is noted that a consistent unloading behaviour of Kw30s556 and Kw23s895 where there is negligible hysteresis between unloading and reloading pore-water pressure responses. In other words, mechanical wetting and mechanical drying do not display any hysteresis.

a) Compression curve

b) Pore-water pressure response

Figure 5-15 Compression curve and pore-water pressure response of kaolin specimens unloaded prior to reaching positive pore-water pressure

The pore-water pressure response during the constant water content loading was observed and can be differentiated into three regions. The three regions are the unsaturated state (Figure 5-16a), the transition state (transition between unsaturated state to the saturated state as illustrated in Figure 5-16b) and the saturated state (Figure 5-16c). In the unsaturated state region, the change in net normal stress increases the degree of saturation without any influx or out flux of water from the soil specimen. Thus, the change in pore-water pressure is mainly due to the change in degree of saturation and to some extent the re-distribution of pore-water pressure. In the transition region, the change in net normal stress causes the soil to change from the unsaturated state to the saturated state (from negative pore-water

pressure to positive pore-water pressure). The excess positive pore-water pressure then dissipated with time. Once the soil has reached the saturated state region, any further loading causes the soil to behave just like a normal saturated soil test where an increase in net normal stress generates excess pore-water pressure which then dissipates with time.

There is a difference between the duration required for the specimen to reach the end of primary compression (t_{end}) and the duration required for the pore-water pressure to reach equilibrium (t_{eq}) for the saturated and unsaturated states. In Figure 5-16, the difference between t_{end} and t_{eq} is indicated as Δt . It is important to ensure that the load is applied until both t_{eq} and t_{end} have been reached. Based on the tests results, it appears that t_{eq} is higher than t_{end} and thus governed the test duration. Further validation of the performance of the HCT is given by the oedometer tests on the residual soil specimens from Bukit Timah Granite (Figure 5-17) which shows identical response. Thus, in order to obtain the correct pore-water pressure, it is important to allow the pore-water pressure to reach equilibrium. Based on the observations for kaolin and residual soil specimens, 20 minutes duration should be sufficient for each loading stage.

5.4.3. Comparison of unsaturated compression curve and change in pore-water pressure response under different strain rates

In order to observe the effect of different loading rates on the unsaturated compression curve and pore-water pressure response, kaolin specimens and residual soil specimens from Bukit Timah Granite were subjected to CWC-P by using oedometer and CRS apparatuses where the strain rates in CRS apparatus are defined in mm/min. The results of kaolin specimens are given in Figures 5-18 and 5-19 while the results of residual soil specimens are given in Figures 5-20 and 5-21.

a) Pore-water pressure response of w47s252 at the 51 kPa net normal stress (unsaturated state region)

b) Pore-water pressure response of Kw30s468 which is loaded at 3260kPa(the transition from unsaturated state to the saturated state region)

c) Pore-water pressure response of Kw42s347 which is loaded at 1630 kPa (saturated state region)

Figure 5-16 Pore-water pressure response observed under constant water content oedometer loading

a) Pore-water pressure response of BTw20s902 at 815 kPa net normal stress (unsaturated state)

b) Pore-water pressure response of BTw20s902 at 3260 kPa net normal stress (transition state)

c) Pore-water pressure response of BTw26s332 at 3260 kPa net normal stress (saturated state)

Figure 5-17 Pore-water pressure response for residual soil specimen observed under constant water content oedometer test

a) Compression curve

a) Pore-water pressure response

Figure 5-18 CWC-P results for kaolin specimens with water content ranged from 47 to 51%

For CRS tests which were carried out for specimen with water content ranging from 47 to 51%, all specimens loaded under the same strain rate are plotted with the same symbol. It appears that even under similar water content condition, the initial matric suction and void ratio of the soil may vary as shown in Figure 5-18a.

The initial negative pore-water pressure ranged from -250 kPa to -350 kPa but some of the soil specimens also showed a significantly low negative pore-water pressure (high matric suction) around -400 to -500 kPa which then gave an untypical pore-water pressure response when the soil specimens were loaded, especially under low net normal stress around less than 30 kPa. At higher net normal stress, the pore-water pressure response seems to converge regardless

of the initial untypical reading. The increase in pore-water pressure was inconsistent and thus may be related to the poor contact condition between the soil specimen and the HCT. As the net normal stress increases, the contact condition improves leading to a more consistent pore-water pressure response.

Once the kaolin specimens reached the fully saturated condition, it appears that a small positive pore-water pressure remained. The reason for such small positive pore-water pressure is due to the undrained condition at the bottom of the specimen which does not allow dissipation of the water pressure at the bottom of the specimen or small measurement error of the HCT pressure transducer.

The difficulties in interpreting the effect of strain rates on the unsaturated compression curve for kaolin specimens with water content ranging from 47% to 51% lie in the disparity of the initial void ratio which caused some differences in the unsaturated compression curves. However, the differences are great only prior to NCL. Once it reaches the NCL, most of the curves show a consistent behaviour where they follow the NCL and the differences become negligible. However, at the end of unloading stage (when the CRS apparatus has stopped moving) for the specimen which was loaded at a strain rate of 0.1 mm/min, the load cell recorded a continuous but limited increase in the net normal stress. This indicates that the movement of the CRS apparatus is higher than the swelling rate of the soil specimen which causes at the end of the unloading stage, the soil keeps confined to swell and thus the additional swelling pressure was recorded by the load cell. Such behaviour may affect the interpretation of the unloading line and therefore must be avoided.

For specimens with water content around 37.5%, only three rates of strain and one incremental loading were used. Thus, the names of the specimens are shown in Figure 5-19. In Figure 5-19a, the unsaturated compression curve of the four specimens seems to be consistent regardless of the strain rates which were applied.

a) Compression curve

b) Pore-water pressure response

Figure 5-19 CWC-P results for kaolin specimens with water content around 37.5%

There are small discrepancies in the pore-water pressure response as shown in Figure 5-19b. Specimen Kw37.5-465-0.1 seems to have slightly lower negative pore-water pressure compared to the other specimens. However, with an increase in the net normal stress, an abrupt increase in the pore-water pressure caused the pore-water pressure response to converge with the other two curves. On the other hand, Specimen Kw37.5-362-0.01 indicates a very similar trend with the rest of the specimens with a relatively constant gap.

Kaolin specimens with water content around 37.5% have less variation in the initial void ratio and thus it is easier to analyse. The unsaturated compression curves of the four specimens showed a very consistent behaviour prior to unloading and therefore do not show any effect

of strain rate on the compression curves. However, when the soil was unloaded, it appears that a slower strain rate gave a smaller change in volume at the beginning of the unloading stage. Once the kaolin specimen was unloaded at net normal stress smaller than 30 kPa, all of the kaolin specimens showed negligible volume change.

Residual soil specimens from Bukit Timah granite with water content around 26% and 18% seems to indicate a consistent result for both its unsaturated compression curve and pore-water pressure response as shown in Figures 5-20 and 5-21. However, the HCT cavitated during the compression test of Specimen BTW18-0.001 and thus no matric suction reading is available for this specimen. BTw18-975-0.001 and BTw18-1039-0.1 were unloaded prior to the pore-water pressure becomes positive. It is shown in Figure 5-21b that the pore-water pressure showed negligible hysteresis during the unloading and reloading stages and is consistent with the behaviour of Specimen Kw30s556 and Kw23s895.

The unsaturated compression curves of the residual soil showed no apparent effect of the strain rates. However, Specimen BTw26-300-0.1 showed similar behaviour and the kaolin specimen which was tested at a strain rate of 0.1 mm/min where at the end of unloading stage, the load cell recorded an increase in the net normal stress up to 100 kPa even after the CRS apparatus has stopped loading the specimen. The load cell also recorded an increase in net normal stress at the end of the unloading stage for the other residual soil specimens but smaller than 10 kPa.

In Figure 5-22, pore-water pressure response during unloading is plotted with void ratio. In Figure 5-22a, Specimens Kw30s556 and Kw23s895 show that there is almost no hysteresis between loading and unloading. In Figure 5-22b, Specimens BTw18-975-0.01 and BTw18-1039-0.1 show no hysteresis at the first unloading but BTw18-975-0.01 shows hysteresis at the 2nd and 3rd unloading. However, it follows a unique line once it reached their e_c . The hysteresis at the 2nd and 3rd unloading are suspected to be due to the re-distribution of the pore-water.

Therefore, when the soil was loaded, the pore-water redistributed again and followed the previous trend.

When the soil specimen was unloaded after the pore-water pressure became positive, the pore-water pressure responses were not consistent for every specimen. Several trends which were observed on all of these specimens are:

1. Pore-water pressure remains positive and shows negligible change during unloading.
2. Pore-water pressure becomes negative and shows significant change during unloading.

a) Compression curve

b) Pore-water pressure response

Figure 5-20 CWC-P results for residual soil specimens with water content around 26%

a) Compression curve

b) Pore-water pressure response

Figure 5-21 CWC-P results for residual soil specimens with water content around 18%

A plausible reason is, as the pore-water pressure became positive during the compression, there will be a change in water content as the soil cannot have degree of saturation more than 100%. Some of the water which flowed out of the soil specimen was absorbed by the dry filter paper and porous stone. When the soil specimen is unloaded, the soil specimen is under mechanical drying as the pore size increases while there is no water which filled up these pores. As a consequence, there will be a sudden increases in the matric suction which attracted the excess water to flow back into the soil. When there is sufficient excess water to be absorbed by the soil, the pore-water pressure will remain as positive.

a) Kaolin specimens

b) Residual soil specimens from Bukit Timah Granite

Figure 5-22 Pore-water pressure response with respect to the change in void ratio

However, where there is insufficient excess water, the pore-water pressure will remain negative. Evaporation on water which flows out from the soil specimen during compression will also affect the amount of water which can flow back into the specimen during the unloading. This phenomenon causes the soil specimen to have higher matric suction when the soil specimen was unloaded. Therefore, when the soil is unloaded after it reaches positive pore-water pressure, there is no certainty on how much water flows back into the soil. This

phenomenon shows the limitation of CWC-P in evaluating the unloading stage for soil specimens which have reached fully saturated condition as it is currently not possible to know the amount of water which flows back into the soil specimen during the unloading.

This phenomenon can be verified by using the observations of Specimen BTw20s902 during the unloading stages as shown in Figure 5-23. In Figure 5-23a, the unloading stage is carried after the specimen is loaded up to 3260 kPa. Specimen BTw20s902 was then unloaded to 815 kPa. During this unloading stage, the pore-water pressure suddenly decreased to -400 kPa. A sudden significant decrease in pore-water pressure is due to the sudden dilation of the soil's pores and re-distribution of pore-water increases the pore-water pressure until it becomes positive (around 50 kPa). The soil was then further unloaded from 815 kPa to 204 kPa. As more water has filled the pores due to the previous unloading and the decrease in net normal stress is not as much as the previous unloading stage, the dilation caused a decrease in pore-water pressure up to -250 kPa which was not as much as the previous unloading. However, the pore-water pressure did not go back to positive value but remained constant at around -150 kPa. This is due to there is no enough water flow back into the specimen. After around 3 hours, the pore-water pressure started to decrease which might be due to either evaporation or re-distribution of the pore-water.

In Figure 5-24, Kaolin specimens with very low water content were compared under different strain rates. It is shown that only the unsaturated compression curve of specimen Kw1.5-0.1 is slightly higher than the rest of the specimens with a constant vertical offset. However, the trend appears to be similar with the other specimens. All of these observations indicate that the effect of strain rates for the loading case is essentially negligible. This can be rationalized as during the constant water content loading, the air is compressed rather than the water. However, as the permeability of air is very high, strain rates will give negligible effect unto the compressibility of soils. The strain rates from the saturated test should governed the test

duration for the unsaturated test as when the soil is compressed and the soil becomes saturated, positive excess pore-water pressure will start to develop and lower strain rate will allow the positive pore-water pressure to dissipate.

a) Unloaded from net normal stress 3260 kPa to 815 kPa

b) Unloaded from 815 kPa to 204 kPa

Figure 5-23 Pore-water pressure response of BTw20s902 during the unloading stage

Figure 5-24 Comparison of different strain rates for kaolin specimens with water content ranging from 1 to 2%

5.5. Chapter summary

The saturated and unsaturated compression curves which were obtained showed a consistent behaviour. Based on the observations for the experiments carried out in this study, several conclusions can be made regarding the compression and wetting-induced behaviour of unsaturated soils. The conclusions are as follows:

1. A decrease in the void ratio will lead to an increase in the preconsolidation pressure.
2. When the soil becomes saturated prior to reaching NCL, the preconsolidation pressure of an initially unsaturated soil is located on the NCL and is referred to as saturated preconsolidation pressure $P_{c,s}$.
3. Provided that the soil remains unsaturated when it reaches the NCL, its unsaturated compression curve will slowly merge into the NCL on further loading as its degree of saturation changes and therefore causes an apparent increase in C_c .
4. The NCL and UCL are parallel under constant degree of saturation condition.

5. Preconsolidation void ratio e_c is the same before and after inundation provided that the inundation pressure is not higher than the $P_{c,s}(e_c)$ and can be represented by Equation (5.2).
6. Provided that the inundation pressure is at the UCL, collapse would cause $P_{c,s}$ to be equal to the inundation pressure and e_c to change from $e_{c,d}$ to $e_{c,s}$ as represented by Equation (5.3).
7. The swelling line with slope $C_{r,s}$ that passes through e_c determines the wetting-induced behaviour of the unsaturated soils. The intersection between swelling line and constant water content unloading line gives the transition pressure P_T . Swelling or swelling pressure P_s occurs when inundation pressure P_i is below P_T while collapse or relaxation occurs when P_i is above P_T .
8. Based on all the values of $C_{r,d}$ and $C_{r,s}$, the value of $C_{r,d}$ is significantly smaller than $C_{r,s}$.
9. The value of $C_{r,d}$ which is taken at net normal stress less than apparent preconsolidation pressure is smaller than $C_{r,d}$ which is taken at net normal stress higher than apparent preconsolidation pressure. This observation is similar for saturated soils.

The performance of the HCT in the unsaturated soil tests was also investigated. The use of kaolin paste improved the contact condition between unsaturated residual soil specimen and the HCT. The comparison between different strain rates and incremental loading on the pore-water pressure and the compressibility of soil were investigated.

The pore-water pressure response due to incremental loading may vary depending on the degree of saturation of the soil specimen. The pore-water pressure response due to incremental loading was classified into three states which are unsaturated state, transition state and saturated state. In the unsaturated state, the pore-water pressure increases and then remains steady once it reaches equilibrium. Based on the observations on the pore-water

pressure response in the unsaturated state for kaolin and residual soils, it appears that 20 minutes duration for each loading stage is sufficient for the unsaturated state. In the transition state, the negative pore-water pressure becomes positive and then dissipates. During saturated state, further loading generates positive excess pore-water pressure which then dissipates. Due to the generation of positive pore-water pressure, conventional 24 hours duration for each loading stage is recommended for both transition and saturated states.

The final change in pore-water pressure was compared between incremental loading and constant rate of strain loading. The observations do not show significant difference between incremental loading and constant rate of strain loading under different strain rates. The unloading and reloading under constant water content condition prior to the pore-water pressure becoming positive rarely show any hysteresis and will always go back into the previous trend prior to unloading once the void ratio reaches the preconsolidation void ratio or void ratio prior to unloading. When the specimen was unloaded after the pore-water pressure becomes positive, the pore-water pressure response shows inconsistent behaviour were at some cases the pore-water pressure will remain positive while in some cases the pore-water pressure will decrease. The reason for such phenomenon depends on the amount of excess water which can flow back into the soils. If there is insignificant water, the pore-water pressure will become negative.

Small differences between data regarding the position of unsaturated compression curve for different strain rates seems to indicate that the effect of strain rate on the unsaturated compression curves appear to be negligible. Strain rate does not significantly affect the compression curve of unsaturated soil as compression mainly occurred at the pores which are filled with air. As the permeability of air is high, the air dissipates instantaneously and thus the primary compression of unsaturated soil is also completed instantaneously.

Very fast unloading strain rate caused the load cell to record an increase in net normal stress after unloading the specimen stopped as the load cell move faster than the soil swells. Such phenomenon is not desired and therefore very fast strain rate should be avoided. As constant water content loading may cause the soil to become saturated, it is important to ensure that the strain rate is slow enough for the saturated condition. Therefore, it is recommended that the strain rate for CWC-P is controlled by the strain rate for the saturated condition.

Chapter 6. Proposed model for estimating compression and wetting-induced volume change of unsaturated soils

6.1. Introduction

A number of methods has been proposed to estimate the volume change of unsaturated soils. However, most of the proposed methods require the use of matric suction control tests or measurement of matric suction which makes the experimental setup expensive and requires long test duration. Often budget and time do not allow the use of such methods in practice. While rapid matric suction measurement is possible by using high-capacity tensiometer, the maximum matric suction which can be measured is still limited to about 1500 kPa. Thus, using matric suction to estimate soil behaviour has a number of limitations and requires further refinements in order to be used in routine engineering practices. In this chapter, a new model is proposed to estimate the volume change of soils due to either change in net normal stress under constant water content condition or change in water content under constant net normal stress condition. The proposed model uses water content as the state variable instead of matric suction. The motivation for proposing the model are to:

1. Enable the use of the conventional oedometer to provide the data.
2. Reduce the time required to obtain the model parameter as it does not require suction measurement/control.

The proposed model uses the shrinkage curve to estimate the change in volume due to the change in water content as the shrinkage curve can be determined easily. Thus, the proposed model is intended to be experimentally cheaper and faster than current models while giving a reasonable estimate of the volume change of unsaturated soils due to different loading paths. The proposed model was constructed based on the experimental observations in this study.

6.2. Derivation of the proposed model

The compression behaviour of unsaturated soils under constant water content condition is not only affected by the net normal stress but also by the initial water content/void ratio due to shrinkage. Therefore, further understanding can be obtained by observing the constant water content compression path on a w versus e plot. The results of the CWC oedometer tests (circle with line) were compiled in Figure 6-1. Under constant water content path, the volume change of the soil does not change its water content and therefore, void ratio decreases until it reaches and then follows the loading line for the fully saturated condition.

Figure 6-1 Constant water content path on the shrinkage curve from the unsaturated oedometer test results

Figure 6-1 contains a number of constant net normal stress contour lines where the net normal stress in kPa is indicated in the legend. The contour lines were constructed from the CWC oedometer test results. There are several observations of these contour lines:

1. The contour line has a similar shape as the shrinkage curve.
2. The slope of the contour line is essentially zero at low water contents.

3. Contour lines with net normal stress higher than 800 kPa indicate that the void ratio decreases with a decrease in water content while contour lines below 800 kPa indicate that void ratio is increasing with an increase in water content.

In Figure 6-1, the relationship between e_c from each test and the initial water content was plotted (e_c is plotted with square symbol). The curve which represents e_c versus w_0 for soil which is compressed from e_0 is referred to as the e_c curve and it indicates two things:

1. For the case where water content is above the intersection between the e_c curve and the saturation line, the soil becomes saturated prior to reaching its apparent preconsolidation pressure as the e_c curve is below the saturation line.
2. The value of e_c for water content below the intersection between e_c curve and saturation line remains constant indicating that e_c is only a function of e_0 for the drying case. Thus, there will be no change in e_c if e_0 does not change. This observation is consistent with the swell and constant volume tests where the inundation does not change the value of e_c if the change in volume does not cause the void ratio to be lower than e_c .

6.2.1. Estimating e_c curve and preconsolidation pressure for case I

For the case where the e_c curve is above the saturation line, the experimental results show that there is an apparent increase in preconsolidation pressure and it is not located on the NCL but on the UCL ($P_{c,d}$) with degree of saturation equals to S_c where S_c is given as:

$$S_c = \frac{w_0 G_s}{e_c} \quad (6.1)$$

Considering that e_c remains constant when e_0 remains constant (shrinkage curve reaching the zero shrinkage line), $C_{r,d}$ must get smaller in order to maintain the value of $P_{c,d}$ increases as water content decreases. This phenomenon is illustrated in Figure 6-2. In Figure 6-2, a dry

specimen with zero water content is subjected to compression and yield on the UCL when the net normal stress is equal to $P_{c,d}$ and e is equal to e_c .

Figure 6-2 Change in $C_{r,d}(S_c)$

In order to reach e_c , the slope of the initial compression line is equal to $C_{r,d}(S_c = 0)$. However, only the value of e_c is of interest in order to construct the e_c curve. Thus, the same value of e_c can be obtained at net normal stress of $P_{c,s}$ on the NCL if the slope of initial compression line equals to $C_{r,d}(S_c=1)$.

Therefore, by assuming that $C_{r,d}(S_c=1)$ remains constant, the construction of the e_c curve is unaffected by the fact that $P_{c,d}$ increases with a decrease in water content for the same e_0 . By using the line with slope of $C_{r,d}(S_c=1)$ and $P_{c,s}$, the e_c curve in Figure 6-1 can thus be constructed as follows:

$$e_c(w_0) = e_0(w_0) - C_{r,d}(S_c = 1) \log \left[\frac{P_{c,s}(w_0)}{P_0} \right] \quad (6.2a)$$

Hence, $C_{r,d}(S_c=1)$ is determined as:

$$C_{r,d}(S_c = 1) = \frac{e_0 - e_c}{\log\left(\frac{P_{c,s}}{P_0}\right)} \quad (6.2b)$$

e_0 can be a function of water content and can be estimated from the shrinkage curve. The slope $C_{r,d}(S_c=1)$ should not be confused with the slope $C_{r,s}$. The slope $C_{r,d}(S_c=1)$ is the slope of the unloading line when the degree of saturation at preconsolidation pressure is equal to 1 but unsaturated along the unloading line as it is under the constant water content condition. The slope $C_{r,s}$ is the slope of unloading line when the soil is always saturated and thus water will always fill the void during the unloading and cause the soil to further expand.

The change in $P_{c,s}$ can be assumed to be as shown in Figure 5-4 and thus can be obtained as follows:

$$P_{c,s}(w_0) = 10^{\frac{e_{N,s} - e_0(w_0)}{C_c - C_{r,d}(S_c=1)}} \quad (6.3a)$$

$$P_{c,s} = 10^{\frac{e_{N,s} - e_c}{C_c}} \quad (6.3b)$$

Equation (6.3a) and (6.3b) are equivalent. However, Equation (6.3a) is useful to give the relationship between $e_0(w)$ and $P_{c,s}$ and thus is used in Equation (6.2a) while Equation (6.3b) is useful to obtain $C_{r,d}(S_c=1)$ in Equation (6.2b). The value of $P_{c,s}(w_0)$ will not increase further once $e_0(w_0)$ no longer decreases.

6.2.2. Estimating UCL under different degrees of saturation

In case II, the unsaturated compression curve does not merge with the NCL. Thus the NCL for unsaturated soil is referred to as the UCL. The C_c of unsaturated soil is commonly assumed to be increasing or decreasing with an increase in matric suction (Alonso et al., 1990, Wheeler and Sivakumar, 1995).

Despite this contradiction, each assumption seems to be supported by experimental results. In this study, Specimens Kw1 and Kw6 indicate that the UCL at constant degree of saturation is parallel to the NCL. The void ratio along the UCL or NCL is also the preconsolidation void ratio and is given as:

$$e_{UCL} = e_c(S_c, P_{c,d}) = e_N(S_c) - C_c \log P_{c,d} \quad (6.4)$$

where $e_N(S_c)$ is the the void ratio on the UCL at $P_{c,d} = 1$ kPa at degree of saturation S_c . This assumption has three advantages:

1. It simplifies the problem as there is only one value of C_c and thus, only e_N is needed to represent the effect of degree of saturation on the UCL.
2. The parameters can be obtained by using standard oedometer test.
3. The phenomenon where UCL gets steeper and then merges with the NCL as net normal stress increases can be modelled by changing the value of e_N with respect to S_c .

Net normal stress which is higher than $P_{c,d}$ causes the soil to yield on different UCLs due to the change in degree of saturation and therefore, $P_{c,d}$ increases to the applied net normal stress. Similar to the NCL, each point located on the UCL is the respective $P_{c,d}$ as shown in Figure 6-3 and can be used to back calculate the value of $e_N(S_{c,i})$ as follows:

$$e_N(S_{c,i}) = e_{UCL}(S_{c,i}, P_{c,d}) + C_c \log P_{c,d} \quad (6.5)$$

where $S_{c,i}$ is the known S_c value, $e_N(S_{c,i})$ is the value of $e_N(S_c)$ at $S_c = S_{c,i}$. By using the final void ratio from the oedometer test for kaolin specimens with C_c obtained from the saturated oedometer test, the values of e_N are shown in Figure 6-4 for the kaolin specimens.

Figure 6-3 e_N as a function of S_r

Figure 6-4 Modelling e_N under different degrees of saturation

From Figure 6-4, it appears that a linear function can be used to relate the value of e_N with S_c .

The linear function is given as:

$$e_N(S_c) = A - BS_c \quad (6.6)$$

When $S_c = 1$, $e_N(S_c) = e_{N,s}$ where $e_{N,s}$ is the void ratio on the NCL when net normal stress = 1 kPa. When $S_c = 0$, $e_N = e_{N,d}$ where $e_{N,d}$ is the void ratio at the UCL when $S_c = 0$. Substituting these two conditions into Equation (6.6), gives:

$$e_{N,d} = A \quad (6.7a)$$

$$e_{N,s} = A - B \quad (6.7b)$$

Substituting Equation (6.7a) into (6.7b) gives:

$$B = e_{N,d} - e_{N,s} \quad (6.7c)$$

Substituting Equation (6.7a) and (6.7c) into Equation (6.6) gives:

$$e_N = e_{N,d} - (e_{N,d} - e_{N,s})S_c \quad (6.8)$$

However, as it may not be easy to completely dry the specimen to $S_c = 0$, it is more useful to use arbitrary $S_{c,i}$ which gives $e_N = e_N(S_{c,i})$. Substituting $S_{c,i}$ and $e_N(S_{c,i})$ into Equation (6.6) gives:

$$e_N(S_{c,i}) = A - BS_{c,i} \quad (6.9a)$$

$$A = e_N(S_{c,i}) + BS_{c,i} \quad (6.9b)$$

Substituting Equation (6.9b) into Equation (6.7b) gives:

$$e_{N,s} = e_N(S_{c,i}) + B(S_{c,i} - 1) \quad (6.10a)$$

$$B = \frac{e_{N,s} - e_N(S_{c,i})}{S_{c,i} - 1} \quad (6.10b)$$

Thus, substituting B in Equation (6.10b) into Equation (6.9b) gives:

$$A = e_N(S_{c,i}) - [e_{N,s} - e_N(S_{c,i})] \frac{S_{c,i}}{1 - S_{c,i}} \quad (6.11)$$

Substituting A in Equation (6.11) and B in Equation (6.10b) into Equation (6.6) gives:

$$e_N(S_c) = e_N(S_{c,i}) + \frac{S_c - S_{c,i}}{1 - S_{c,i}} [e_{N,s} - e_N(S_{c,i})] \quad (6.12)$$

By letting $S_{c,i}$ equals to 0 and $e_N(0)$ equals to $e_{N,d}$, Equation (6.12) becomes Equation (6.8). By substituting the values of $e_N(S_{c,i})$ and $S_{c,i}$ from Specimen Kw1 and $e_{N,s}$ into Equation (6.12), e_N for different S_c can be calculated using Equation (6.12) which are shown in Figure 6-4 as a straight line. In Figure 6-4, the final data points with the highest net normal stress were used in order to ensure that the points are located on the UCL.

When the net normal stress has reached the UCL, S_c is changing with compression due to the change in e_c and therefore $e_N(S_c)$ has to be updated as S_c increases. Given that subscript j is the current calculated e and subscript $j-1$ denotes the previously calculated e , S_r or S_c is updated as:

$$S_r = \min \left(\frac{w_0 \cdot G_s}{e_{j-1}}, 100 \right) \quad (6.13)$$

6.2.3. Estimating apparent preconsolidation pressure and initial compression/reloading line

As observed from the experiments carried out in this study, the value of the preconsolidation void ratio e_c is only affected by e_0 despite $P_{c,d}$ increasing with decreasing water content. This information is valuable in modelling an increase in $P_{c,d}$ with a decreasing water content.

As the UCL is a function of S_c , $P_{c,d}$ is given as the net normal stress which gives void ratio equals to e_c on the UCL. Thus, substituting e_c into e_{UCL} in Equation (6.4) gives:

$$P_{c,d}(S_c) = 10^{\frac{e_N(S_c) - e_c}{C_c}} \quad (6.14)$$

Equation (6.14) is the general form of Equation (6.3b). The value of $e_N(S_c)$ can be determined from Equation (6.12) which can then be substituted into Equation (6.14) to determine the value

of $P_{c,d}(S_c)$. The initial compression curve of the soil is modelled as a line with slope equals to $C_{r,d}$. The slope of the initial compression line $C_{r,d}$ as a function of S_c can then be calculated as follows:

$$C_{r,d}(S_c) = \frac{e_0 - e_c}{\log \left[\frac{P_{c,d}(S_c)}{P_0} \right]} \quad (6.15a)$$

$$C_{r,d}(S_c) = \frac{e_0 - e_N(S_c) + C_c \log P_{c,d}(S_c)}{\log \left[\frac{P_{c,d}(S_c)}{P_0} \right]} \quad (6.15b)$$

where P_0 is the net normal stress at $e = e_0$. The initial void ratio e_0 is known. Alternatively, e_0 can be obtained from the shrinkage curve while e_c can be obtained from either the known loading history or the e_c curve. When $S_c = 1$, Equation (6.15) becomes Equation (6.2b).

The initial compression/reloading line of the unsaturated soil is then given as:

$$e = e_0 - C_{r,d}(S_c) \log \left(\frac{\sigma - u_a}{P_0} \right) \quad (6.16a)$$

Similar to the initial compression/reloading line of the unsaturated soil, the initial compression/reloading line of saturated soil is then given as:

$$e = e_0 - C_{r,s} \log \left(\frac{\sigma - u_w}{P_0} \right) \quad (6.16b)$$

A common problem associated with modelling the compression curve using a bilinear line in a semi-log plot is that the initial compression line or reloading line will overestimate the amount of settlement. The overestimation is due to the hysteresis between the initial compression curve and the unloading curve. In order to resolve the non-linearity of the initial compression curve, a new term C_i which represent the secant slope of the initial compression curve is introduced. The value of C_i is assumed to vary from 0 at net normal stress p equals to 1 kPa,

up to C_r when net normal stress equals to the preconsolidation pressure. Therefore, it is proposed to use interpolation between 0 to C_r to represent the value of C_i . The proposed equation is then given as:

$$C_i(\sigma - u_a) = C_r \left(\frac{\sigma - u_a}{P_c} \right)^{0.5} \quad (6.17)$$

By using C_i in Equation (6.17) to replace $C_{r,d}(S_c)$ and $C_{r,s}$ in Equation (6.16), the hysteresis between initial compression line and unloading line can be taken into account.

6.2.4. Estimating unloading line

In the case of unloading, the unloading line can be calculated as follows:

$$e = e_u + C_{r,d}(S_c) \log \left(\frac{\sigma - u_a}{P_u} \right) \quad (6.18)$$

where e_u is the void ratio prior to unloading. In the case of initial compression/reloading lines, the value of e_0 is always known. Therefore $C_{r,d}(S_c)$ is calculated by using the known value of e_0 while e_c can either be estimated from the e_c curve or from the known loading history.

However, in the case of unloading, e_c is known while e_0 is not known. Moreover, as the soil is loaded under constant water content condition prior to unloading, the degree of saturation of the soil keeps changing as the soil is loaded. Thus, $C_{r,d}(S_c)$ which is used to calculate the initial compression line is different from the $C_{r,d}(S_c)$ of the unloading line as the value of S_c is different.

In order to simplify the problem for the unloading case, $C_{r,d}(S_c)$ is estimated by using the values of e_0 and e_c (Equation 6.15a) while S_c is obtained from the degree of saturation prior to unloading. Thus, $C_{r,d}(S_c)$ changes only due to the change in S_c which causes changes in $P_{c,d}(S_c)$. The value of $P_{c,d}(S_c)$ is calculated using e_0 and e_c from the initial compression curve. Therefore in Equation (6.15), only S_c needs to be updated for different loading histories (e_0 and e_c remains

constant) in order to obtain $C_{r,d}(S_c)$. Figure 6-5 shows the change in $C_{r,d}(S_c)$ with respect to S_c for the Kaolin specimen.

The experimental results show that the slope of the unloading line of unsaturated soil at net normal stress less than its apparent preconsolidation pressure is smaller than the slope of the unloading line of unsaturated soil at net normal stress higher than its apparent preconsolidation pressure. Thus, it is necessary to model the unloading line at net normal stress less than the apparent preconsolidation pressure to avoid unreasonable expansion due to the assumption of a constant slope unloading line.

Figure 6-5 Modelling $C_{r,d}(S_c)$ of kaolin specimen

In order to take into account the reduction of the slope of the unloading line C_r , Schmertmann (1955) equation is used by specifying e_1 and e_2 values as e_u which is void ratio at the unloading and e_c , respectively. The value of R_c which represent the reduction of the slope of unloading line prior to preconsolidation pressure can be calibrated. Thus, C_r prior to e_c is given as:

$$C_r(e_u) = C_r(e_c) \left(\frac{e_c + 1}{e_u + 1} \right)^{R_c} \quad (6.19)$$

6.2.5. Estimating the wetting-induced behaviour of unsaturated soils

Wetting-induced behaviour such as swelling and collapse of unsaturated soils have attracted a lot of attention due to its catastrophic effect. Thus, it is very important to quantify this behaviour. In this section, a method to quantify the wetting-induced behaviour is proposed.

In the proposed method, two important lines governing the wetting-induced behaviour of soils which are the pre-inundation line (bold line) and the post-inundation line (dashed line) are shown in Figure 6-6. The pre-inundation line is essentially the unsaturated compression curve and therefore can be obtained by using the model which has been proposed in the previous section. The post-inundation line describes the upper boundary of the wetting-induced behaviour of unsaturated soils due to full saturation and is unaffected by the initial water content of the soil (Phien-wej et al., 1992). When the post-inundation line is above the pre-inundation line, swelling and swelling pressure will occur. On the other hand, when post-inundation line is below the pre-inundation line, collapse and pressure relaxation will take place.

In Figure 6-6, P_i is the inundation pressure, $e_{s,0}$ is the void ratio on the post-inundation line at $P_i = 1$ kPa, $e_{d,0}$ is the void ratio on the pre-inundation line at $P_i = 1$ kPa, $e_{d,i}$ is the void ratio on the pre-inundation line at P_i , $e_{s,i}$ is the void ratio on the post-inundation line at P_i , P_s is the post-wetting net normal stress, P_T is the transition pressure, $P_{c,s}$ is the saturated preconsolidation pressure at $e = e_c$ and $P_{c,d}$ is the unsaturated preconsolidation pressure at $e = e_c$.

Based on Figure 6-6, there are three different ranges of inundation pressure ($P_i < P_{c,s}$, $P_{c,s} < P_i < P_{c,d}$ and $P_i > P_{c,d}$) which require three different sets of equations. For the case where $P_i < P_{c,s}$, the void ratios at the pre-inundation line $e_{d,i}$ and post-inundation line $e_{s,i}$ are given, respectively, as:

$$e_{d,i} = e_c + C_{r,d}(S_c) \log \left[\frac{P_{c,d}(S_c)}{P_i} \right] \quad (6.20a)$$

$$e_{s,i} = e_c + C_{r,s} \log \left(\frac{P_{c,s}}{P_i} \right) \quad (6.20b)$$

where $C_{r,d}(S_c)$ is obtained from Equation (6.15b) while the value of $P_{c,s}$ can be determined from Equation (6.3) or from Equation (6.14) with $e_N(S_c) = e_{N,s}$ and thus, $P_{c,d}(S_c) = P_{c,s}$. Thus, the amount of swelling/collapse settlement can be determined by substituting e_c in Equation (6.20a) with Equation (6.20b) such that:

$$\Delta e = e_{s,i} - e_{d,i} = C_{r,s} \log \left(\frac{P_{c,s}}{P_i} \right) - C_{r,d}(S_c) \log \left[\frac{P_{c,d}(S_c)}{P_i} \right] \quad (6.21)$$

In the case of constant volume condition, $e_{s,i} = e_{d,i}$ while P_i in Equation (6.20b) equals to swelling pressure P_s . Thus, substituting $e_{d,i}$ in Equation (6.20a) into $e_{s,i}$ in Equation (6.20b), the swelling pressure P_s can be obtained as:

$$P_s = P_{c,s} \left[\frac{P_i}{P_{c,d}(S_c)} \right]^{\frac{C_{r,d}(S_c)}{C_{r,s}}} \quad (6.22)$$

The transition point between swelling and collapse can be obtained by letting Δe in Equation (6.21) equals to zero. The transition point between swelling and collapse is defined using the transition pressure P_T and is given as:

$$P_T = 10^{\frac{C_{r,s} \log P_{c,s} - C_{r,d}(S_c) \log P_{c,d}(S_c)}{[C_{r,s} - C_{r,d}(S_c)]}} \quad (6.23)$$

For $P_{c,s} < P_i < P_{c,d}(S_c)$ (Figure 6-6b), the relaxation pressure P_s can still be determined from Equation (6.22) while the collapse settlement Δe is given as:

$$\Delta e = C_c \log P_{c,s} - C_{r,d}(S_c) \log P_{c,d}(S_c) + [C_{r,d}(S_c) - C_c] \log P_i \quad (6.24)$$

For $P_i > P_{c,d}$ (Figure 6-6c), the amount of collapse settlement is essentially the same as C_c remains constant. Therefore, Δe is given as:

$$\Delta e = e_N(S_c) - e_{N,s} \quad (6.25)$$

Δe in Equation (6.25) can also be interpreted as the amount of the mechanically-based macropores which are collapsible.

When the soil is inundated under constant volume condition on its UCL, the current net normal stress becomes its $P_{c,d}$ and the soil is subjected to relaxation as there will be a decrease in net normal stress until it reaches the NCL. Thus, $P_{c,d}$ decreases until it becomes $P_{c,s}$ with void ratio equals to e_c . Therefore, the post-wetting net normal stress P_s is essentially $P_{c,s}$ and can be easily determined from e_c as follows:

$$P_s = P_{c,s} 10^{\frac{e_{N,s} - e_c}{C_c}} = 10^{\left(\frac{e_{N,s} - e_N(S_c)}{C_c} + \log P_i \right)} \quad (6.26)$$

The calculation steps are given as follows:

1. Determine the values of C_c , $C_{r,s}$, $e_{N,s}$, $C_{r,d}(S_c=1)$, $P_{c,d}$ and e_c
2. Calculate $P_{c,s}$ using either Equation (6.3) or Equation (6.14)
3. Calculate P_T using Equation (6.23)
4. Choose the value of P_i
5. Determine the amount of wetting-induced volume change using Equations (6.21) and either Equation (6.24) or (6.25)
6. Determine the post-wetting net normal stress using either Equation (6.22) or Equation (6.26)

a) $P_i < P_{c,s}$

b) $P_{c,s} < P_i < P_{c,d}$

c) $P_i = P_{c,d}$

Figure 6-6 Wetting-induced behaviour under different inundation pressures

6.2.6. Estimating the actual amount of swelling and collapse prior to $P_{c,s}$

The actual amount of collapse for the case where $P_i > P_{c,d}$ can be estimated by using Equation (6.25). However, a method to estimate the actual amount of swelling and collapse for the case where $P_i < P_{c,s}$ has not been proposed and therefore is proposed in this section.

Based on Figure 6-1, the relationship between void ratio and water content under constant net normal stress has a similar shape as the shrinkage curve under zero net normal stress for the swelling case. Thus, by knowing the maximum swelling from the previous section, it is possible to estimate the actual amount of swelling under different initial water contents by assuming that the same shrinkage curve is applicable for swelling. This assumption is supported by the observation from Fleureau et al. (2002) mentioned in Chapter 2. From Chapter 4, the proposed shrinkage curve equation is:

$$e(w) = e_{\min} + m_1 w + \sum_{i=2}^n T_i(w, w_i, m_{i-1}, m_i, k_i) \quad (6.27)$$

where $e_{\min}$ is the minimum void ratio of the shrinkage curve, m_i is the slope of the linear segment i , w_i is the water content at the intersection between two linear segments i and $i+1$, k_i is the curvature parameter which describes the transition of each linear segment, and n is the number of linear segments of the shrinkage curve. The advantage of Equation (6.27) is that it has a good control over the curvature and it is also able to represent the shrinkage curve with more than two linear segments as shown in Chapter 4. It is assumed that two linear segments are sufficient to describe the swelling curve and therefore $n = 2$. For swelling curve, m_1 can be assumed to be 0 while m_2 is equal to G_s/S_f where G_s is the specific gravity and S_f is the degree of saturation after inundation which is assumed to be 100%. The intersection between zero shrinkage line and saturation line is w_2 and therefore is defined as the apparent shrinkage limit SL' . Thus, Equation (6.27) can be simplified for $n = 2$ as follows:

$$e(w) = e_{\min} + \frac{G_s}{2S_f} \left\langle w + \frac{1}{k} \ln \left\{ \frac{\cosh[k(w - SL')]}{\cosh(kSL')} \right\} \right\rangle \quad (6.28)$$

In the case of swelling, the initial void ratio prior to swelling $e_{d,i}$ under inundation pressure P_i should be the minimum void ratio except when it is higher than its natural minimum void ratio $e_{\min-n}$. Thus, $e_{\min}$ in Equation (6.28) equals to:

$$e_{\min} = \min(e_{\min-n}, e_{d,i}) \quad (6.29)$$

Based on Equation (6.29), SL' is given as:

$$SL' = \frac{e_{\min} \cdot 100}{G_s} \quad (6.30)$$

For some soils especially soils with very low air-entry value, the swelling stop once it reaches the fully saturated condition. The value of parameter k can be calculated from the water content at full saturation condition w_s and the apparent shrinkage limit SL' . Thus, k is given as:

$$k = \frac{1}{w_s - SL'} \quad (6.31)$$

where w_s can be obtained from $e_{s,i}$ which is the void ratio after inundation at inundation pressure P_i such that:

$$w_s = \frac{100 \cdot e_{s,i}}{G_s} \quad (6.32)$$

For the case of collapse where P_i is greater than the $P_{c,d}$, the amount of collapse which is given by Equation (6.25) is linearly related to the change in degree of saturation. As Equation (6.12) is used to estimate the relationship between e_N and S_c , it is assumed that for the case where $P_i < P_{c,d}$, the actual amount of collapse is also a linear function of the degree of saturation. The amount of collapse is given as:

$$e(S_r) = e_{d,i} + \frac{S_r - S_i}{1 - S_i} (e_{s,i} - e_{d,i}) \quad (6.33)$$

where S_r is obtained from Equation (6.13) which is calculated incrementally and S_i is the degree of saturation prior to inundation.

6.2.7. Estimating the amount of shrinkage under different net normal stress

As soil may also be subjected to shrinkage under different net normal stresses, a method to estimate the post compression shrinkage curve is given in this section. Based on the shrinkage test on Specimen 5 which has been outlined in the previous chapter, it is shown that even for soil which has been compressed such that its e_0 is below its $e_{\min-n}$, it is possible for the soil to shrink.

In the previous section, it was shown that e_c , $C_{r,d}$ and $C_{r,s}$ affect the amount of swelling of the soil. Thus, it is possible to assume that the amount of shrinkage is affected by the same parameters. Therefore, for the case where the net normal stress is less than the transition pressure P_T , the amount of shrinkage can be calculated by using Equation (6.21) with $e_{\min}$ calculated as:

$$e_{\min} = \min(e_{\min-n}, e_{s,i} - \Delta e) \quad (6.34a)$$

$$e_{\min} = \min \left\{ e_{\min-n}, e_{s,i} - \left[C_{r,s} \log \left(\frac{P_{c,s}}{P_i} \right) - C_{r,d} (S_c = 0) \log \left[\frac{P_{c,d} (S_c = 0)}{P_i} \right] \right] \right\} \quad (6.34b)$$

where S_c equals to 0 to represent a completely dry condition and is used to obtain $e_N(S_c=0)$, $C_{r,d}(S_c=0)$ and $P_{c,d}(S_c=0)$. These values are used to calculate the upper bound pre-inundation line. For the case where the net normal stress is higher than P_T , the soil specimen has been subjected to very high net normal stress. Thus, the pre-inundation line is above the post-inundation line as the unsaturated soil has become stiffer due to the effect of matric suction

which supports the metastable pores. is higher than the effect of matric suction in reducing the soil volume Thus, it is assumed that there will be no further shrinkage beyond this pressure.

6.3. Summary of the proposed model in estimating the volume change of unsaturated soils

A new model has been proposed to estimate the volume change of unsaturated soil due to:

1. Change in net normal stress under constant water content condition
2. Change in water content under constant net normal stress

The proposed model uses water content and degree of saturation as the state variables to represent the unsaturated condition such that it is possible to obtain the parameters in conventional soil testing laboratory. Several tests which are required for this model are:

1. Saturated compression test (C_c , $e_{N,s}$, $C_{r,s}$),
2. Unsaturated compression test ($C_{r,d}(S_c=1)$, $S_{c,i}$, $e_N(S_{c,i})$, and
3. Shrinkage test (e_0 , $e_{min.n}$, G_s , SL' , k)

The saturated and unsaturated compression tests are used to determine the parameters which are related to the change in volume due to the change in net normal stress under constant water content condition and also the maximum swelling or collapse settlement due to the inundation. Shrinkage test is used to obtain parameters which are related to the change in volume due to the change in water content under constant net normal stress condition.

Saturated oedometer test can be carried out following ASTM-D2435/D2435M (2011). Thus, only shrinkage and unsaturated oedometer test procedures are elaborated in this section. The procedure are given as follows:

1. Cut the soil sample by using a slightly bigger internal diameter ring cutter than the oedometer ring. The purpose is to allow the soil to shrink due to drying such that, the

specimen can fit into the oedometer ring thereafter. In this study, a 6.3 cm internal diameter ring cutter was used to trim the specimen prior to drying and a 5 cm internal diameter oedometer ring was used to further trim the specimen for the oedometer test.

2. Air dry the soil specimen, and measure its volume and weight at regular intervals until there is no further change in the weight of the specimen to construct the shrinkage curve.
3. Cut the specimen using the 5 cm internal diameter oedometer ring once the soil has been dried. As dried soil cracked easily during this process, it is proposed to use cling wrap to wrap the perimeter of the specimen so as to apply a small confining pressure. After that, push the oedometer ring using an even pressure. It is preferred to use a machine such as a soil sample extruder or a hydraulic pump to ensure that a uniform pressure can be exerted on the oedometer ring.
4. Placed the dried specimen in the oedometer apparatus, and then load up to the maximum possible load for the apparatus.
5. Inundate the specimen to let the specimen collapse to NCL.
6. Unload the specimen back to a small net normal stress i.e. 50 kPa in order to obtain the slope of the unloading line at the saturated condition.

By using the above procedures, all of the parameters to model the compression behaviour of unsaturated soils can be obtained with only one additional soil specimen. However, more precise parameters may be obtained by testing soil specimens having several different degrees of saturation in order to obtain a relationship between $e_N(S_c)$ and S_c .

Figure 6-7a illustrates the proposed model. The compression of unsaturated soil can fall into either Case I (Section 5.2.2) or Case II (Sections 5.2.3 and 5.2.4). In Case I, the soil specimen becomes saturated prior to the NCL. Thus, the unsaturated compression curve yields at the

NCL. The change in $P_{c,s}$ is due to the change in $e_{0,d}$ which follows the shrinkage curve while the value of $C_{r,d}(S_c = 1)$ remains constant for both cases I and II. In case II, the soil specimen does not yield on the NCL but on the UCL which is a function of degree of saturation. However, the value of e_c is only a function of $e_{0,d}$ and also the function of $C_{r,d}(S_c = 1)$. Thus, $P_{c,d}$ and $P_{c,s}$ can be estimated from e_c curve and $e_N(S_c)$. The soil shrinks as a result of the change in matric suction. Whenever the matric suction is less than the AEV of the soil, the matric suction has an equal effect as the net normal stress in decreasing soil volume. Therefore, change in matric suction may also cause plastic deformation if the matric suction causes the void ratio of the specimen to go below its past minimum void ratio e_c .

a) Compression model

b) Shrinkage, swelling and collapse model

Figure 6-7 Illustration of the proposed model

However, when the matric suction increases above its AEV, an increase in matric suction is no longer effective in decreasing the soil's volume. As a result, there is a minimum void ratio which

can be attained through shrinkage deformation. When the soil shrinks from the slurry condition, the minimum void ratio is referred to as the natural minimum void ratio $e_{min,n}$ and $e_{min,n}$ is commonly related to the shrinkage limit (Fredlund et al., 2002). Drying soil from slurry caused plastic deformation as the soil's void ratio is always smaller than its previous e_c throughout the shrinkage process until it reaches $e_{min,n}$. Therefore, wetting such a soil will cause the soil to swell by a limited amount as swelling is an elastic volume change as it does not cause the soil's void ratio to decrease below its e_c . When the soil shrinks again after being swelled, it is also considered as elastic behaviour until it reaches its e_c . Thus, when a soil swells from A to B in Figure 6-7b, the soil will also shrink from B to A when it is dried.

Figure 6-7b illustrates the assumption of the proposed model to model the wetting/drying induced volume change of unsaturated soils. In Figure 6-7b, soil specimen which is dried from $e_{N,s}$ (of slurry condition) will have its void ratio e decreasing following the initial shrinkage curve (ISC) until it reaches the $e_{min,n}$. However, for the case where the soil specimen has void ratio below this value (i.e. being consolidated and then unloaded with final void ratio below $e_{min,n}$), it is possible for the soil specimen to have a minimum void ratio e_{min} below $e_{min,n}$ which can be determined from Equation (6.34)

The initial shrinkage curve is due to plastic deformation as drying the soil causes it to have a void ratio which is smaller than its previous void ratio. Thus, it is not possible for the specimen which has been dried to $e_{min,n}$ to return to its $e_{N,s}$ value. Therefore, the soil specimen swells to $e_{0,s}$ instead of $e_{N,s}$ which is determined by the post inundation line. The amount of swelling can be calculated by using Equation (6.24).

However, as the soil is compressed to $P_i = P_{i,1}$ in Figure 6-7, the amount of swelling will decrease as shown by the curve A-B. The shrinkage curve equation can be used to estimate the swelling path and is given by Equation (6.28). For the swelling case from dry condition, the value of e_{min} is defined by Equation (6.29). Equation (6.28) was chosen as the curvature parameter as it can

fit the $e_{s,i}$ on the loading line with water content equal to w_s . The curvature parameter k is given by Equation (6.31). For the soil which is swelled from void ratio below its minimum void ratio, it is assumed that it follows the same shrinkage curve when it is dried as the soil cannot shrink below its past minimum void ratio. Thus, Equation (6.28) gives the shrinkage and swelling path for the particular net normal stress.

When P_i is equal to P_T (transition point), there is no change in volume due to an increase in the water content which is indicated as curve C-D in Figure 6-7b. However, above P_T such as when $P_i = P_{i,2}$, the soil specimen starts to collapse. The collapse path is indicated by curve E-F in Figure 6-7b. The amount of collapse is assumed to be controlled by the degree of saturation of the specimen regardless of the cause i.e. either mechanical wetting or change in water content. For the case when $P_T < P_i < P_{c,d}$, the actual amount of collapse is estimated by using Equation (6.33) while when $P_i > P_{c,d}$, the amount of collapse is estimated by using Equation (6.25).

The flow chart of the proposed model is given in Figure 6-8. In the proposed model, three curves are required to be obtained experimentally which are the shrinkage curve, the constant water content compression curve (CWC) and the saturated compression curve (SCC). The parameters which are obtained from each curve are shown in the shaded box in Figure 6-8. The flow chart which shows the application of the equation to model the CWC test is given in Figure B-1 of Appendix B.

The initial water content w_0 and the net normal stress are the known variables. From the shrinkage curve and w_0 , e_0 and S_0 can be obtained. From e_0 , S_0 and $C_{r,d}(S_c=1)$, the e_c curve can be constructed in order to differentiate whether the soil falls under case I or II. The $C_{r,d}(S_c)$ is determined from Equation (6.15) which uses the value of e_c and $P_{c,d}(S_c)$. For case I, C_c , e_N and $C_{r,d}(S_c)$ are used to determine the value of $P_{c,s}$ while for case II, S_c , $P_{c,d}(S_c)$, $e_N(S_c)$, $C_{r,d}(S_c)$ and $C_i(\sigma-u_a)$ are determined.

Figure 6-8 Unsaturated compression analysis

From all of these parameters, changes in S_r , S_c , $e_N(S_c)$ and e can be obtained. The values of S_r , e , S_c and $e_N(S_c)$ are calculated incrementally as the previously calculated e affects the value of S_r which is used to calculate the value of $e_N(S_c)$ and e . For the case where the unloading takes place after $P_{c,d}$ (for unsaturated soil) or $P_{c,s}$ (for saturated soil), Equation (6.15) is used to recalculate the value of $C_{r,d}(S_c)$ by only changing the value of S_c . For the case of unloading prior to $P_{c,d}$ (for unsaturated soil) or $P_{c,s}$ (for saturated soil), $C_r(e_u)$ from Equation (6.19) is used to replace $C_{r,d}(S_c)$ in Equation (6.18).

6.3.1. Validation of the proposed method

In order to show how the proposed model works, test results on kaolin specimens, residual soils from Bukit Timah Granite (BT) and compacted silty clay (JSC) from Jotisankasa et al. (2007) were used. All of the parameters used for the simulations are given in Table 6-1.

All of the soil data were calculated based on the same procedure. All of the parameters were first loaded from one saturated compression test and one unsaturated compression test (preferably with the lowest water content). From these two soil data, the other soil tests under different initial water contents will be simulated following the exact loading path of each soil specimen.

The shrinkage curve is constructed when the parameters for the shrinkage curve are available. The shrinkage curve is used to estimate the initial void ratio (i.e. kaolin and residual soil from Bukit Timah Granite). When a reasonable shrinkage curve cannot be constructed, the initial void ratio from the soil data is then used. This condition is applicable for data from Jotisankasa et al. (2007).

The value of $C_{r,d}(S_c = 0)$ is shown in Table 6-1 in order to estimate post shrinkage compression curve and thus is only applicable for Kaolin and residual soil from Bukit Timah Granite where the post compression shrinkage curve data are available for verification. In order to obtain a good estimate on the post compression shrinkage curve, initial void ratio prior to drying and the value of e_c were obtained from the experimental data instead of from the simulation. The rest of the parameters are obtained from Table 6-1.

Table 6-1 Parameters for the simulation

Parameter	Kaolin	BT	JSC
$e_N(S_{c,i})$	2.889	1.472	1.183
$S_{c,i}(\%)$	4.879	0.732	3.912
$e_{N,s}$	2.758	1.378	0.936
C_c	0.592	0.279	0.166
$C_{r,d}(S_c=1)$	0.066	0.026	0.021
$C_{r,d}(S_c=0)$	0.061	0.0235	-
R_c	15	15	15
$C_{r,s}$	0.131	0.049	0.06
e_{min}	1.289	0.68	-
G_s	2.66	2.65	2.6
k	0.15	0.3	-
$SL'(\%)$	48	25.66	-

6.3.1.1. Validation using Kaolin specimen

For the kaolin specimen, the saturated parameters from the tests on Specimens KODSat1 to KODSat4 gave similar result. Thus, Specimen KODSat2 was chosen to obtain the saturated compression parameters. Specimen Kw1 was chosen to obtain the unsaturated compression parameters as it provides the data at the lowest degree of saturation. The initial void ratio was

estimated from the shrinkage curve which is the same as the one shown in Figure 6-1. The relationship between $e_N(S_c)$ and S_c which was produced by the spreadsheet program is shown in Figure 6-4. The line which shows the estimated $e_N(S_c)$ was obtained by using data from Specimens KODSat2 and Kw1. The first example which is shown in Figure 6-9 is the estimated compression curve for Specimen Kw1. The data for Specimen Kw1 have certain features such as Specimen Kw1 was unloaded prior to its apparent preconsolidation pressure and after its apparent preconsolidation pressure.

a) Change in S_r due to the change in net normal stress

b) Compression curve

c) w vs e

Figure 6-9 Simulation of Kw1

Figures 6-9a, b and c are arranged such that the x-axis of change in S_r due to the change in net normal stress (Figure 6-9a) and the x-axis of the compression curve Figure 6-9b) coincides while the y-axis of the compression curve coincides with the y-axis of the hydraulic path (Figure 6-9c). The estimated changes in void ratio and degree of saturation seem to match the data points

and the unloading before and after the apparent preconsolidation pressure can be represented very well.

The case where the soil was inundated at net normal stress less than the transition pressure after following a certain loading history is given by the test results of Specimen Kw5 and is shown in Figure 6-10. The results for Specimen Kw5 showed that the unloading prior to preconsolidation pressure can be estimated very well. However, there was inaccuracy in determining e_c prior to the second unloading which gave an inaccurate estimate of swelling. However, the trend of the volume change agrees with the data.

Figure 6-10 Estimation of the change in volume of Kw5 following the same stress path

The reason for the inaccuracy can be seen in Figure 6-4 where the $e_N(S_c)$ of Specimen Kw5 (S_c approximately 13%) is slightly different from the estimated $e_N(S_c)$. The estimated S_r and w

versus e do not seem to be inaccurate which shows that the error from the compression curve was not significant in affecting the degree of saturation.

In order to show the swelling estimation when e_c can be accurately predicted, the final net normal stress in the loading sequence was changed to 1300 kPa from 1630.3 kPa and the result is shown in Figure 6-11. Figure 6-11 shows a good match between the estimated wetting-induced volume change behaviour and the data provided that e_c can be accurately estimated prior to inundation.

Figure 6-11 Estimation of the change in volume of Kw5 by adjusting the net normal stress prior to second unloading from 1630.3 kPa to 1300 kPa

In order to show examples of collapse behaviour, Specimens Kw6 and Kw9 were used to show how the proposed model simulates the volume change of both soil specimens and the results are shown in Figures 6-12 and 6-13. Figure 6-12 shows that the first unloading, second

unloading, the UCL and the volume change after inundation can be modelled very well. Figure 6-13 shows that there is small inaccuracy in estimating the second unloading curve although this inaccuracy does not seem to have any significant effect on the estimation of the degree of saturation. However, the first unloading, the UCL, the amount of collapse and the change in volume after inundation can be estimated very well.

a) Change in S_r due to the change in net normal stress

b) Compression curve

c) w vs e

Figure 6-12 Estimation of the change in volume of Kw6

Following the same procedures, the rest of the behaviour of the other unsaturated soil specimens were estimated. As most of the data fall into case I, the w versus e for each specimen was not plotted as it only shows a vertical line until it meets with the loading line as shown in Figure 6-1. Therefore, only the compression curve and the change in S_r due to the change in net normal stress of each soil specimen are shown in Figures 6-14 and 6-15,

respectively. A shift in the estimation of the void ratio especially on the highest net normal stress is due to the small difference in C_c between specimens.

Based on the results which are shown in Figures 6-9 to 6-15, the proposed models are able to capture the trend and in most of the cases provides a good estimation of the volume change of soils. However a proper estimation of the $e_N(S_r)$ is essential in determining the UCL. When an error in estimating the UCL is large, the error will propagate into the estimation of volume change due to wetting and the position of the unloading line will not be correct although a good trend can still be obtained.

a) Change in S_r due to the change in net normal stress

b) Compression curve

c) w vs e

Figure 6-13 Simulation of Kw9

a) Kw60s40

b) Kw57s110

c) Kw57s81

d) Kw48s287

e) Kw47s252

f) Kw42s347

g) Kw37s399

h) Kw30s556

i) Kw30s468

j) Kw23s895

Figure 6-14 Compression curve of kaolin specimens

a) Kw60s40

b) Kw57s110

c) Kw57s81

d) Kw48s287

Figure 6-15 Change in degree of saturation of the kaolin specimens

To illustrate the estimation of post compression shrinkage curve, test results of Kaolin specimens 1 to 5 from chapter 4 were used. Specimens 1 to 4 were dried and the e_{min-n} is equal to 1.289. The SL' is calculated by using Equation (5.33). The curvature parameter is set to be 0.15 to curve fit the shrinkage curve.

Specimen 5 in Chapter 4 was subjected to net normal stress equals to 2035 kPa under saturated condition prior to unloading. Thus, its $P_{c,s}$ equals to 2035 kPa with $e_c = 0.731$. The soil was then unloaded and the measured final void ratio is equals to 1.179 which is much less than the e_{min-n} . By using parameters from Table 6-1, $e_N(S_c=0)$ was calculated and is equals to 2.896. Substituting $e_N(S_c=0)$, e_c and C_c into Equation (5.17), $P_{c,d}$ equals to 4529 kPa. Substituting all of these parameters into Equation (5.37b) and letting P_1 equals to 1 kPa, the estimated Δe is equal to 0.211 and thus, the e_{min} of the post compression shrinkage curve is 0.968. The SL' was calculated using Equation (5.33) and is equal to 36%. The curvature parameter k_2 for Specimen 5 was obtained from Specimens 1 to 4 and therefore is equal to 0.15. The shrinkage curves for Specimens 1 to 5 were constructed by using Equation (5.31) and are shown in Figure 6-16. Figure 6-16 shows that accurate estimation of the shrinkage curve of Specimen 5 can be obtained.

Figure 6-16 Estimated shrinkage curves of kaolin specimens under different e_0

6.3.1.2. Validation using residual soils from Bukit Timah Granite

Six oedometer tests were conducted on residual soils which are two saturated oedometer test (Specimens BTODSat1 and BTODSat2) and four constant water content oedometer tests (Specimens BTw0.1, BTw1.3, BTw20s902 and BTw26s332). In this section, Specimen BTODSat2 was used to obtain saturated parameters while Specimen BTw0.1 was used to obtain the

unsaturated parameters. The parameters which were used are given in Table 6-1. It is important to note that $e_N(S_{c,i})$ and $e_{N,s}$ are dependent on the chosen value of C_c and the variation of $e_N(S_c)$ with S_c is given in Figure 6-17 while the variation of $C_{r,d}(S_c)$ with S_c is calculated by using Equation (5.18) and is given in Figure 6-18.

Figure 6-17 Variation of $e_N(S_c)$ of Residual soil from Bukit Timah Granite

Figure 6-18 Modelling $C_{r,d}(S_c)$ of Residual soil from Bukit Timah Granite

The second step is to construct the shrinkage curve and e_c curve by using the spreadsheet program. The results are shown in Figure 6-19. Figure 6-19 shows a significant disparity between e_0 for each specimen as residual soil is significantly more inhomogeneous than kaolin.

By using the idealized shrinkage curve, the e_0 of the specimens under different initial water contents were estimated.

Figure 6-19 Shrinkage curve and e_c curve of residual soil from Bukit Timah Granite

The simulated unsaturated compression curves of residual soils from Bukit Timah Granite are given in Figures 6-20 to 6-22. Figures 6-20 and 6-21 show cases for specimens which have a combination of loading, unloading and inundation paths. Specimen BTw0.1 shows a consistent behaviour with Specimen Kw5 which proves the validity of Equation (5.2). However, there seems to be a discrepancy between $e_{N,s}$ which was used for simulation and $e_{N,s}$ of Specimen BTw0.1 which caused a mismatch between the simulated normal compression line and the data. The simulation for Specimen BTw1.3 was more difficult as there was a big discrepancy between the estimated e_0 and the e_0 from the laboratory data which causes the initial compression curve to be located below the laboratory data. However, it appears that this difference is not significant in affecting the change in degree of saturation due to the change in net normal stress. Figure 6-21 shows the results of Specimen BTw1.3 with e_0 obtained from the laboratory data instead of from the shrinkage curve. It was shown that good estimation of the unsaturated compression curve can be obtained. Both of the specimens have shown a good estimation of volume change of Specimens BTw0.1 and BTw1.3, at least qualitatively.

a) Compression curve of BTw0.1

b) Compression curve of BTw1.3

c) w Vs e of BTw0.1

d) w Vs e of BTw1.3

e) σ Vs S_r of BTw0.1

f) σ Vs S_r of BTw1.3

Figure 6-20 Simulation results of Specimens BTw0.1 and BTw1.3

a) Change in S_r due to the change in net normal stress

b) Compression curve

c) w vs e

Figure 6-21 Simulation results for BTw1.3 with e_0 from the laboratory data

Figure 6-22 shows an example of case I. In Figure 6-22, all of the specimens became saturated prior to reaching the NCL. It is shown that the estimated curve is able to estimate the change in volume, S_r and e under different net normal stresses $\sigma-u_a$. However, discrepancy is shown at the high net normal stresses where the void ratio was not accurately estimated. The reason is due to the discrepancy in C_c . These examples showed that the shrinkage curve affects the estimation of soil behaviour significantly. Thus, proper estimation of the shrinkage curve is needed.

Figure 6-22 Simulation results of BTw20s902 and BTw26s332

Estimation of the post compression shrinkage curves for the residual soil specimen from Bukit Timah Granite is given in Figure 6-23. The symbols which are used in Figure 6-23 has the same meaning as the symbols in Figure 4-7. The net normal stresses which were applied on the saturated oedometer test, constant water content oedometer test and constant water content CRS test are different. Saturated oedometer test and constant water content oedometer test are different due to the different oedometer ring dimensions which are 6.3 cm and 5 cm

internal diameter, respectively. The constant rate of strain tests are different due to the different maximum applicable net normal stress that can be given to the soil specimen. However, the value of e_0 after unloading is relatively similar. Thus, the estimation of the post compression shrinkage curve was only carried out using Specimen BTw1.3. Specimen BTw1.3 has $P_{c,s}$ of 3260 kPa with e_c equals to 0.351 and then unloaded. Following the same procedures as the kaolin specimen, the estimated post compression shrinkage curve is shown in Figure 6-23 and it shows a good match.

Figure 6-23 Estimated shrinkage curve of residual soil specimens from Bukit Timah granite for different e_0

6.3.1.3. Validation using data from (Jotisankasa et al., 2007)

Another validation of the proposed method in estimating the compression behaviour of unsaturated soil was conducted using the data from Jotisankasa et al. (2007) who tested a silty clay. The silty clay was created by using static compaction at a dry of optimum water content. The shrinkage curve which was constructed from tests on 10 specimens (7-10-D, 7-10-G, 7-10-H, 7-10-K, 7-10-P, 7-10-SL, 7-10-T, 7-10-U, 7-10-V and 7-10-W) is plotted in Figure 6-24. Figure 6-24 indicates that the data are widely dispersed. Thus, e_c was estimated from the known e_0 and the $C_{r,d}(S_c=1)$ which can be determined from Equation (5.5b).

In order to model the unsaturated compression curve from all of the specimens, only data from Specimens 7-10-SL and 7-10-P were used. By using C_c from Specimen 7-10-SL, $e_N(S_c)$ for each specimen was obtained by using the final data points and are shown in Figure 6-25. Figure 6-25 shows that the value of $e_N(S_c)$ increases linearly with a decrease in S_c which agrees with the observation for the kaolin specimens and also the residual soil specimens. Therefore, it supported the assumption of Equation 5.15. However, it appears that the estimated $e_N(S_c)$ which used data from Specimens 7-10-SL and 7-10-P is above most of the data points. As the estimated $e_N(S_c)$ is above the real $e_N(S_c)$, the proposed model overestimated the void ratio on the UCL. Thus, the sensitivity of the error in estimating $e_N(S_c)$ is evaluated in this section.

Table 6-1 shows the parameters which were used to estimate the volume change of the silty clay. The estimated compression curve and change in degree of saturations are plotted in Figures 6-27 and 6-28, respectively. The proposed method manages to capture the trend of the compression curve very well for soil with higher degree of saturation. The UCL of the compression curve gets steeper until it merges with the NCL. However, as the estimated $e_N(S_c)$ is higher than most of the data points, the estimated unsaturated compression curve overestimates the void ratio of the data points.

Figure 6-24 Shrinkage curve of silty clay specimens from Jotisankasa et al. (2007)

Figure 6-25 Values of $e_N(S_c)$ for silty clay specimens from Jotisankasa et al. (2007)

Figure 6-26 Values pf $C_{r,d}(S_c)$ for silty clay specimens from Jotisankasa et al. (2007)

a) 7-10-D

b) 7-10-G

c) 7-10-H

d) 7-10-T

e) 7-10-U

f) 7-10-V

g) 7-10-W

h) 7-10-P

i) 7-10-K

Figure 6-27 Estimation of the compression curve for data from Jotisankasa et al. (2007)

a) 7-10-D

b) 7-10-G

c) 7-10-H

d) 7-10-T

Figure 6-28 Change in degree of saturation for data from Jotisankasa et al. (2007)

6.4. Comparison between proposed method and Barcelona basic model (BBM)

A number of models have been proposed in the past to model the elasto-plastic behaviour of soils. Most of these models are variants of the Barcelona basic model (BBM) and use matric suction to represent the unsaturated state of the soil. The difference between most of these models have been given in Chapter 2. In this section, the proposed model is compared with the BBM to show the advantages, disadvantages and similarities between the two models. The comparison focuses mainly on volume change as the proposed model is only able to model the volume change of unsaturated soils while BBM can also be used to model the change in shear strength of unsaturated soils due to different loading histories.

6.4.1. State variables and required equipment

The biggest difference between the proposed models and the BBM is the state variables which are used to represent the unsaturated condition. BBM used two stress-state variables which are net normal stress and matric suction. On the other hand, the proposed model used water content and degree of saturation to represent the unsaturated state of soils.

The difference in the state variables gives a very significant difference in the efforts to obtain the parameters for the model. As BBM used matric suction as the state variable, it requires sophisticated equipment to do suction control and suction measurement. Suction control test usually requires a very long time to do the test as it is important to maintain the matric suction at the desired matric suction after additional loading/unloading is applied. On the other hand, as the proposed method uses water content and degree of saturation, conventional oedometer apparatus is sufficient to obtain all of the model parameters and a one-day loading duration (or less) can be used for each loading. Thus, the proposed model can be considered to be very efficient in terms of the required apparatus and ease of obtaining the parameters.

6.4.2. Assumption on model parameters

There are a number of differences in the proposed model with BBM due to the different state variables which are used and the assumptions on the soil behaviour. The one to one comparison between the assumptions in the BBM and in the proposed models is given in Table 6-2. It is important to note that in the BBM, the x-axis is in natural log scale while in the proposed model, the x-axis is in log scale. Thus, different symbols are used to represent some of the parameters. For the sake of convenience in discussing the differences in this section, the slope of the UCL is referred to as C_c , the intercepts of the NCL and UCL with net normal stress equals to 1 kPa are referred as $e_{N,s}$ and $e_{N,d}$, the slopes of unloading-reloading line under unsaturated and saturated conditions are referred to as $C_{r,d}$ and $C_{r,s}$, respectively. When the parameter is a function of another parameter, the controlling variables are given in the brackets.

The basis of BBM (or any of the BBM variants) is that C_c is a function of matric suction s . In order to obtain $C_c(s)$, two additional parameters, r and β_a , are needed. In order to obtain these two parameters, two to three constant matric suction tests are usually conducted. In the BBM, it is important to assume that $C_c(s)$ is decreasing due to an increase in matric suction as it affects the determination of $P_{c,d}(s)$. If $C_c(s)$ increases with an increase in matric suction, there will be no increase in the $P_{c,d}$. This assumption causes $e_N(S_c)$ to be lower than $e_{N,s}$. This assumption is not valid for soils which are formed under dry condition such as loess. As mentioned in Chapter 2, loess is formed under dry condition such that the void ratio at low net normal stress can be much higher than the void ratio at liquid limit. Thus, loess is prone to collapse when it is wetted even when the net normal stress is only 1 kPa. Therefore, for loess soil, $e_N(S_c)$ must be higher than $e_{N,s}$.

Table 6-2 Comparison between BBM and proposed model

Parameter	BBM	Proposed model
UCL	$C_c(s) = C_c(s=0)[(1-r)\exp(-\beta_a s) + r]$	$C_c(S_r) = C_c$ $e_N(S_c) = e_N(S_{c,i}) + \frac{S_c - S_{c,i}}{1 - S_{c,i}} [e_{N,s} - e_N(S_{c,i})]$
yield curve	$\left[\frac{P_{c,d}(s)}{P_{ref}} \right] = \left(\frac{P_{c,s}}{P_{ref}} \right)^{\frac{[C_c(s=0) - C_{r,s}]}{[C_c(s) - C_{r,s}]}}$	$P_{c,d}(S_c) = 10^{\frac{e_N(S_c) - e_c}{C_c}}$
URL	$C_{r,d}(s) = C_{r,s}$	$C_{r,d}(S_c) = \frac{e_0 - e_c}{\log \left[\frac{P_{c,d}(S_c)}{P_0} \right]}$ e_0, e_c and $P_{c,d}(S_c)$ are determined from one specimen so that $C_{r,d}(S_c)$ is only a function of S_c .
swell/shrink	For elastic deformation: $de = -\kappa_s \frac{ds}{s + P_{atm}}$ For plastic deformation: $de = -\lambda_s \frac{ds}{s + P_{atm}}$	for $P_i < P_{c,s}$ $\Delta e = C_{r,s} \log \left(\frac{P_{c,s}}{P_i} \right) - C_{r,d} \log \left(\frac{P_{c,d}}{P_i} \right)$ $e(w) = e_{min} + \frac{G_s}{2S_f} \left\langle w + \frac{1}{k} \ln \left\{ \frac{\cosh[k(w - SL')]}{\cosh(kSL')} \right\} \right\rangle$ $k = \frac{1}{w_s - SL'}; w_s = \frac{100e_{s,i}}{G_s}$ for $P_{c,s} < P_i < P_{c,d}$ $\Delta e = C_c \log P_{c,s} - C_{r,d} \log P_{c,d} + (C_{r,d} - C_c) \log P_i$ $e(S_r) = e_{d,i} + \frac{S_r - S_i}{1 - S_i} (e_{s,i} - e_{d,i})$ for $P_i > P_{c,d}$ $\Delta e = e_N(S_r) - e_{N,s}$

In the proposed model, C_c is constant under constant degree of saturation. However e_N is a function of degree of saturation. Therefore, UCL which is constant under constant degree of saturation seems to get steeper if the degree of saturation increases during constant water content loading. Thus, the proposed model is able to represent the condition where the UCL is getting steeper until it merges with the NCL (when the change in degree of saturation is high) and a condition where the UCL never joins or remains constant (when the change in degree of saturation is low).

The proposed model is also able to explain the reason for loess soil which collapses at very low net normal stress by using the value of $e_N(S_c)$ which is higher than $e_{N,s}$. In the proposed model, the value of e_c is also not affected by swelling and thus wetting under low net normal stress

does not change the value of e_c . This phenomenon has been proven by the experiments which were conducted and is an important part of this study. This behaviour is intuitively correct as the minimum particle arrangement which is represented by e_c should be due to the maximum combined effect between net normal stress and matric suction. Therefore, any combination of net normal stress and matric suction which does not cause the soil to compress less than e_c should not cause the soil to yield. As most of the previously proposed models relied on the preconsolidation pressure or yield suction, the estimation of the apparent preconsolidation pressure becomes difficult. This behaviour is beneficial and is used as an important assumption to calculate the values of $P_{c,s}$ and $P_{c,d}$.

The difference in the LC curve or yield curve between BBM and proposed model is shown in Figure 6-29. The term yield curve is preferred as the curve is actually showing how the preconsolidation pressure of the soil increases as the matric suction increases or water content decreases rather than indicating the starting point where the soil will collapse upon wetting (which is referred as transition pressure in the proposed model). In BBM, the yield curve is a function of the matric suction. As matric suction changes, the apparent preconsolidation pressure increases. However, there is a unique yield curve for different saturated preconsolidation pressures. The saturated preconsolidation pressures (according to how the yield curve equation was derived) must be obtained by inundating the soil specimen at the desired matric suction which is also used to obtain the unsaturated compression curve under the reference net normal stress (prior to compression). In the proposed model, the yield curve is a function of initial water content. Changes in water content cause changes in e_c and S_e which then affect the preconsolidation pressure. Thus different preconsolidation pressures can be obtained for different initial conditions (e_0 and w_0). The shape of the yield curve is then dependent on whether it falls under case I or II and the maximum $P_{c,d}$ is given when w_0 is equals to 0. Figure 6-29b was constructed using Kaolin specimens and the parameters are given in Table 6-1.

The value of C_r is assumed to be constant in the BBM. Such assumption has been proven to be incorrect as the value of C_r is different when there is change in water content and under constant water content condition. As a consequence, in BBM the change in volume due to the change in matric suction is not dependent on C_r but requires additional parameters which are κ_s and λ_s to calculate the volume change of the soil due to the change in matric suction (i.e. shrinkage and swelling). The two parameters remain constant under different net normal stresses. It is quite common to increase the net normal stress to reduce the amount of swelling and therefore this assumption is unlikely to be true. Thus, these parameters must be calibrated at a specific net normal stress which is desired and must not be used for other net normal stresses. In order to obtain these two parameters, controlled suction test is required.

Figure 6-29 Comparison of the yield curve

In the proposed model, the value of $C_{r,d}$ is used to represent the unloading line under constant water content condition while $C_{r,s}$ is used to represent the unloading line when there is a change in water content and the soil remains saturated during unloading. The value of $C_{r,d}$ is also a function of the degree of saturation at the preconsolidation void ratio (S_c) and is calibrated by using one set of experimental data and is assumed to be only a function of S_c regardless of e_c and e_0 for the case of unloading. These parameters are not only affecting the unloading behaviour of soils but also the possible boundary of shrink and swell under different

net normal stresses. The proposed model used shrinkage curve to model the change in void ratio due to the change in water content or degree of saturation.

6.5. Chapter summary

Based on the experiments which were performed, a model was proposed to estimate the volume change of unsaturated due to the change in net normal stress and due to the change in water content. The proposed model used water content as a state variable instead of matric suction which is conventionally used in elasto plastic models such as the Barcelona basic model (BBM) so that the parameters can be easily calibrated.

In the proposed model, the preconsolidation void ratio e_c has an important role. The value of e_c is used to represent the yield condition due to net normal stress and matric suction. The value of e_c remains constant in the case of swelling. As the value of e_c is not always available, it is proposed to estimate e_c from e_c curve. The e_c curve is constructed by using the shrinkage curve and also the slope of the unloading line $C_{r,d}(S_c=1)$ under constant water content condition with preconsolidation degree of saturation equals to 1. Based on the e_c curve and saturation line, it is possible to determine whether the unsaturated compression curve of the soil merges into the normal compression line (case I) or crosses the normal compression line (case II).

In the proposed model, it is assumed that the unsaturated compression curve UCL is parallel to the normal compression line NCL under constant degree of saturation with slope equals to C_c . The void ratio e_N of the UCL at net normal stress equals to 1 kPa is linearly related to the preconsolidation degree of saturation S_c . The apparent preconsolidation pressure $P_{c,d}$ can thus be estimated by using e_c , C_c , $e_N(S_c)$ and S_c . The mechanical wetting can also be represented in the proposed model as the change in degree of saturation during the compression will cause the UCL to slowly merge into the NCL.

The slope of the unloading line prior to inundation was shown to be much smaller compare to the unloading line after inundation and this difference is considered in the proposed model. This phenomenon has a significant effect on the determination of the wetting-induced behaviour and shrinkage of unsaturated soil. The reduction of the slope of the unloading line when the unloading is carried out prior to its preconsolidation pressure and the hysteresis between the unloading and the reloading lines are also taken into account. This is one of the main difference between the proposed model and BBM as in the BBM, the slope of unloading line is always taken as constant regardless of its matric suction and preconsolidation pressure.

Shrinkage curve is used to estimate the change in volume due to the change in water content. The shrinkage curve is used as there is negligible hysteresis between swelling and shrinkage. In the proposed model, the loading history is taken into account to estimate the change in volume due to the change in water content. In the BBM, the slope of change in volume due to the change in natural log of change in matric suction is assumed to be constant as it does not take into account the effect of net normal stress and the loading history on the shrinkage and swelling behaviour.

The proposed model has been validated by using test on Kaolin, residual soil from Bukit Timah Granite and statically compacted silty clay from Jotisankasa et al. (2007). The proposed model was able to estimate the change in volume due to the change in net normal stress and water content with a good accuracy.

Chapter 7. Conclusion and recommendations

7.1. Conclusion

An investigation on the compression, shrinkage and wetting induced behaviour of unsaturated soils has been conducted in this study. Several important experimental observations in this study are related to:

1. Relationship between compression, shrinkage, swelling and collapse.
2. Change in matric suction due to loading under constant water content condition.
3. The effect of strain rates on the compression curve and pore-water pressure response.
4. Change in soil-water characteristic curve (SWCC) and shrinkage curve due to the change in density or void ratio.

Based on these experimental observations:

1. A new model has been proposed to estimate the change in volume due to change in water content (shrinkage, swelling and collapse) and net normal stress (compression).
2. A new equation has been proposed to fit SWCC and shrinkage curve with any number of linear segments. The proposed equation has been extended to take into account the effect of density on the SWCC-w- and to estimate the post compression shrinkage curve.

7.1.1. Observations on the volume change of unsaturated soils due to the change in net normal stress and water content

Shrinkage, swelling, compression behaviour and SWCC under different void ratios of unsaturated soils have been investigated in this study. The important observations are:

1. Drying the soil specimen increases its preconsolidation pressure.

2. The unsaturated compression curve does not cross the NCL (Case I) if the soil specimen reaches the fully saturated condition prior to the NCL.
3. The unsaturated compression curve yields at its unsaturated compression line (UCL) if the degree of saturation of the soil specimen has not reached 100% when it crosses the NCL which is similar to the commonly observed behaviour for specimens which are loaded under constant matric suction test.
4. The UCL is parallel to the NCL under constant degree of saturation condition and gets closer to the NCL when the degree of saturation approaches 1.
5. Slope of the unloading line under constant water content condition $C_{r,d}$ is smaller than the slope of unloading line under saturated condition $C_{r,s}$.
6. The preconsolidation void ratio e_c is due to the maximum combined effect between net normal stress and matric suction. Thus, inundation at low net normal stress for soil with void ratio higher than e_c will not affect the value of e_c . The determination of e_c is useful as it simplifies the estimation of apparent preconsolidation pressure.
7. Inundation causes the soil specimen to swell when the net normal stress is less than its transition pressure and to collapse when the net normal stress is higher than its transition pressure. The transition pressure is determined by the preconsolidation void ratio, the saturated and apparent preconsolidation pressure, slope of unloading line under saturated condition and slope of unloading line under constant water content condition.
8. Swelling line with slope equals to the slope of unloading line under saturated condition $C_{r,s}$ which passes through the preconsolidation void ratio can be used to estimate the amount of swelling.

Based on these observations, a model has been developed to estimate the volume change of unsaturated soils due to the change in net normal stress under constant water content

condition and volume change due to the change in water content under constant net normal stress. There are two advantages of the proposed model over existing models:

1. The parameters do not rely on matric suction but use degree of saturation and water content. Thus, the proposed model is suitable for laboratories with limited equipment as it only requires the oedometer apparatus.
2. Only two specimens are required to calibrate the model. The first specimen is used for saturated test, and the second specimen is used for shrinkage test and unsaturated oedometer test. Therefore, it is suitable for routine engineering practice.

A spreadsheet program was developed to model the volume change of unsaturated soils. The spreadsheet program was used to validate the proposed method by using tests conducted in this study for kaolin and residual soils from Bukit Timah Granite. Tests on silty clay from Jotisankasa et al. (2007) were also used in the validation. It was shown that the proposed model is able to capture the compression, shrinkage and wetting-induced behaviour of unsaturated soils very well.

Several Journal and conference papers related to the volume change of unsaturated soils due to the change in net normal stress and water content have been published in Wijaya et al. (2014), Wijaya et al. (2015) and Wijaya and Leong (2015b).

7.1.2. General equation for SWCC and shrinkage curve

The SWCC-w of unsaturated soils can be either unimodal or bimodal and is affected by its density. Based on the experiments carried out in this study, the SWCC-w has a constant virgin drying line for both coarse-grained and fine-grained soils. This behaviour is beneficial in modelling the effect of density on SWCC. However, most of the equations which are used to model the effect of density on SWCC rely on equation which used parameters that cannot be

obtained graphically. Such equations commonly requires more than one SWCC to calibrate the parameters and the values of the parameters depend on the curve fitting algorithm used.

In this study, a new equation to fit a multi-linear segments curve has been proposed to model unimodal and bimodal SWCCs. There are two advantages of the proposed equation:

1. The proposed equation is able to fit unimodal and bimodal SWCCs.
2. The proposed equation account for the effect of density on the unimodal SWCC based on gravimetric water content (SWCC-w) and requires only one SWCC-w to estimate the SWCC-w of the soil at different densities
3. The proposed equation uses graphically obtainable parameters. Thus, it does not require any curve fitting algorithm and the parameters have unique values.

The proposed equation has also been used to model the shrinkage curve of the soil. The proposed equation has been shown to be able to describe the various type of shrinkage curves including the structural shrinkage curve. The proposed equation can also estimate the post compression shrinkage curve.

The work described above have been written up in various publications (Wijaya and Leong, 2014, Leong and Wijaya, 2015, Wijaya and Leong, 2015a, Wijaya and Leong, 2016).

7.1.3. High-capacity tensiometer and constant water content tests

The high-capacity tensiometer (HCT) has been shown to perform very well in measuring the change in matric suction during the constant water content test. Contact condition and evaporation are perhaps the biggest issues in using the HCT. Thus, several suggestions were proposed for conducting constant water content compression test with pore-water pressure measurement as follows:

1. Using a kaolin paste with water content between its plastic and liquid limits can be used to minimise evaporation from the HCT and also to improve the contact condition between the high-capacity tensiometer and the soil specimen.
2. Placing the HCT at the bottom of the modified oedometer apparatus provided stable pore-water pressure readings.
3. Using the HCT to monitor the test duration for incremental loading to ensure reliable results. The next loading stage should be carried only when there is negligible change in pore-water pressure or when the pore-water pressure indicates a sudden decrease in pore-water pressure accompanied with an increase in volume change (which indicates shrinkage due to evaporation).

The constant water content tests conducted in this study indicated that an increase in the degree of saturation due to mechanical wetting increases the pore-water pressure and insignificant hysteresis between mechanical wetting and drying was observed. It was also shown that there is a unique relationship when void ratio is plotted with the change in negative pore-water pressure during loading and unloading. The experiments also showed that the effect of strain rate on the compression curve and pore-water pressure is negligible compare to the effect of the initial void ratio. When an unsaturated soil is loaded to the fully saturated condition, it is recommended that the strain rate used thereafter should be the strain rate for saturated test.

7.1.4. Significant implication on engineering practice

The research has significant implication on the engineering practices as follows:

1. The research has shown the interaction between change in net normal stress and water content with the volume change of unsaturated soils. Thus, shrinkage, swelling, collapse and compression behaviour should be seen as interrelated behaviour instead of independent behaviour.

2. The proposed test method and model to investigate the volume change behaviour of unsaturated soil due to the changes in net normal stress and water content allow the possibility of incorporating unsaturated soil mechanics into practice. The proposed test method uses conventional laboratory equipment and requires short test duration. Thus it is readily implementable in most laboratories.
3. The proposed SWCC model in this thesis allows estimation of SWCC of soils with different initial densities by using only one specimen. Thus, it significantly reduces the cost and the number of specimens required for the modelling.

7.2. Recommendation for the future works

Due to the limited candidature period, it was not possible to explore all interesting related research. Thus, the following recommendations are given for future work:

1. In the current study, shrinkage test was only carried out before and after the compression test. Therefore, the post-compression shrinkage curve always corresponded to a net normal stress of 1 kPa. This issue was not checked for other net normal stresses as it requires precise control over the gravimetric water content which is currently not possible with the current oedometer set-up.
2. The proposed model was calibrated with kaolin and residual soil specimens. It is thus important to validate the proposed model with other soils, especially for expansive soils and collapsible soils.
3. As the proposed model has been investigated using soils consolidated from slurry, it is important to investigate whether the proposed model can be applied to compacted soil as well.
4. To extend the proposed model to model the change in shear strength due to the change in water content as well.

5. The proposed equation for the SWCC can also be used for the wetting SWCC. Thus, the proposed equation can be extended to model a complete drying-wetting SWCC.
6. The effect of density on the drying SWCC has been observed. However, the effect of density on the wetting SWCC curve might be different. Thus, it is important to investigate the effect of density on the wetting SWCC. The proposed equation can then be extended for the wetting SWCC to take into account the effect of density.
7. The effect of density on the bimodal SWCC remains to be investigated. By conducting SWCC tests on the bimodal soils with different initial densities, it is possible to extend the proposed equation to estimate the SWCCs of bimodal soils with different initial densities.
8. The proposed equation for the SWCC and shrinkage curve uses graphically obtainable parameters which have a unique value. Thus, it is possible to obtain correlations of these parameters with basic soil properties. This will enable the proposed equation to be used as predictive equation for the SWCC and shrinkage curve.

Appendix A. Test and apparatus compliance

Compression test using either oedometer or CRS apparatus needs to account for the compliance of the apparatus. To take account of the compliance of the apparatus, the soil specimen was replaced with a steel disk and then subjected to the similar loading path as in the tests (ASTM-D4186, 2006, ASTM-D2435/D2435M, 2011). The compression of the apparatus was then measured. The compliance curves of the oedometer and the CRS apparatus are shown in Figures A-1a and b. There are small differences between the loading compliance curve and the unloading compliance curve. The effect of using only the loading compliance curve in correcting the compression curve was compared when both loading and unloading compliance curves were used to correct the compression curve.

Figure A-1 Compliance curve of oedometer and CRS apparatus

Specimen Kw1 was selected for the comparison as it has the least deformation which makes it the most seriously affected test by the compliance of the apparatus. In Figure A-2, it is shown that irrespective of either the loading compliance curve or the loading and unloading compliance curve was used, it has a negligible effect on the compression curve. Thus, only the loading compliance curve was used to correct the compression curves of all the tests reported in this study.

Figure A-2 Comparison between corrected and uncorrected compression curve of Specimen Kw1

Appendix B Visual basic application code used in this study

In this study, a spreadsheet program in Visual Basic Application (Vba) code was developed to model the soil behaviour. There are two main codes which are used:

1. Function, and
2. Subroutine

When the codes are very complex, userform is used in order to make the use of Vba code becomes convenient. The Visual basic application (Vba) code developed is presented in this Appendix. However, only the functions are given as the presentation of the subroutines requires the layout of the spreadsheet. Due to the number of worksheets which are used, it is not possible to show all the worksheets.

General equation for SWCC and shrinkage curve

The general equation for the soil-water characteristic curve (SWCC) and the shrinkage curve which is derived in this study is presented below where the Vba code for the transition function is given:

Function Ti(x, x0, xi, mh, mi, ki)

"Transition function

'x = axis which can be either gravimetric water content (shrinkage curve) or matric suction (SWCC)

'x0 = minimum x axis value which is arbitrary. Can be 0 (shrinkage curve) or 1 kPa (SWCC)

'mh = slope of segment i-1

'mi = slope of segment i

'xi = x value of intersection between segment i and segment i-1

'ki = parameter which represent the curvature

If ki <> "" Then

Ti = 0.5 * (mi - mh) * ((x - x0) + (1 / ki) * Log(Cosh(ki * (x - xi)) / Cosh(ki * (xi - x0))))

End If

End Function

By using the transition function, the Vba code for the SWCC with any number of linear segments is given as follows:

```
Function SWCCwGeneralEquation(ws, suc, suc1, sucI, m1, mi, k, n)
'ws = saturated gravimetric water content
'suc = matric suction
'suc1 = arbitrary minimum matric suction which can be taken as 1 kPa
'sucI = matric suction at the intersection between segment i and segment i-1
'm1 = slope of the initial drying line
'mi = slope of segment i
'k = curvature parameter between segment i and segment i-1
'n = number of parameters

If n = 0 Then
SWCCwGeneralEquation = 0
Else
ReDim b(2 To n)

For r = 2 To n
b(r) = Ti(Log10(suc), Log10(suc1), Log10(sucI(r)), mi(r - 1), mi(r), k(r))
Next r
p = WorksheetFunction.Transpose(b)
fi = WorksheetFunction.Sum(p)
SWCCwGeneralEquation = ws - Log10(suc / suc1) * m1 - fi
End If

If SWCCwGeneralEquation < 0 Then
SWCCwGeneralEquation = 0
End If

End Function
```

Following the same principle, the Vba code for the shrinkage curve is given as follows:

```
Function ShrinkageCurveGeneral(w, emin, wi, m1, mi, k, n)
'w = water content
'emin = minimum void ratio
'wi = water content at the intersection between segment i and segment i-1
'm1 = slope of the first segment
'mi = slope of segment i
'k = curvature parameter between segment i and segment i-1
'n = number of segment

If n = 1 Then
ShrinkageCurveGeneral = emin
ElseIf n > 1 Then
ReDim b(2 To n)

For r = 2 To n
b(r) = Ti(w, 0, wi(r), mi(r - 1), mi(r), k(r))
Next r
p = WorksheetFunction.Transpose(b)
fi = WorksheetFunction.Sum(p)
ShrinkageCurveGeneral = emin + w * m1 + fi
ElseIf n < 1 Then
ShrinkageCurveGeneral = 0
End If

End Function
```

When it is desired to take into account the effect of density on SWCC, the estimation of SWCC for different densities as a function of void ratio is given as follows:

```
Function SWCCeEquation(ws1, e, s, Gs, suc, suc1, AEV0, suc3, m1, m2, m3, k2, k3)
'ws1 = saturated gravimetric water content of the known SWCC
'e = void ratio of the estimated SWCC
's = degree of saturation which is taken normally as 100%
'Gs = specific gravity
'suc = matric suction
'suc1 = minimum matric suction of the x axis which correspond to ws
'AEV0 = intersection between initial drying line and virgin drying line of the known SWCC
'suc3 = residual matric suction
'm1 = slope of the initial drying line
'm2 = slope of the virgin drying line
'm3 = slope of the residual line
'k2 = curvature parameter between segment 2 and segment 1
'k3 = curvature parameter between segment 3 and segment 2

ws = e * s / Gs
f1 = ws - Log10(suc / suc1) * m1
suc2 = AEVf(AEV0, ws1, ws, m2, m1)

VirginDryingLine = Ti(Log10(suc), Log10(suc1), Log10(suc2), m1, m2, k2)
ResidualLine = Ti(Log10(suc), Log10(suc1), Log10(suc3), m2, m3, k3)
SWCCeEquation = f1 - VirginDryingLine - ResidualLine

If SWCCeEquation < 0 Then
SWCCeEquation = 0
End If

End Function
```

Alternatively, when it is desired to estimate the SWCC for different gravimetric water contents, a similar code is given as follows:

```
Function SWCCwsEquation(ws1, ws2, suc, suc1, AEV0, suc3, m1, m2, m3, k2, k3)
'ws1 = saturated gravimetric water content of the known SWCC
'ws2 saturated gravimetric water content of the estimated SWCC
'Gs = specific gravity
'suc = matric suction
'suc1 = minimum matric suction of the x axis which correspond to ws
'AEV0 = intersection between initial drying line and virgin drying line of the known SWCC
'suc3 = residual matric suction
'm1 = slope of the initial drying line
'm2 = slope of the virgin drying line
'm3 = slope of the residual line
'k2 = curvature parameter between segment 2 and segment 1
'k3 = curvature parameter between segment 3 and segment 2

ws = ws2
f1 = ws - Log10(suc / suc1) * m1
suc2 = AEVf(AEV0, ws1, ws, m2, m1)

VirginDryingLine = Ti(Log10(suc), Log10(suc1), Log10(suc2), m1, m2, k2)
ResidualLine = Ti(Log10(suc), Log10(suc1), Log10(suc3), m2, m3, k3)
SWCCwsEquation = f1 - VirginDryingLine - ResidualLine

If SWCCwsEquation < 0 Then
SWCCwsEquation = 0
End If
```

End Function

Modelling compression and wetting induced behaviour of unsaturated soil

In order to model the compression behaviour of unsaturated soils under constant water content condition, several relationships are needed. A complete flowchart of the estimation of the constant water content compression is given in Figure B-1.

Figure B-1 Flow chart of the proposed model in estimating constant water content compression

Spreadsheet program for the proposed method

To model the volume change behaviour of unsaturated soil, a spreadsheet program was developed. The spreadsheet program has two userforms which are userform 1 as shown in Figure B-2a and userform 2 as shown in Figures B-2b and c. Userform 1 is divided into three sections which are sections A, B and C as shown in Figure B-3.

Userform1 section A (Figure B-3a) is used to load the soil data. “Export saturated parameter” and “Export unsaturated parameter” command buttons in section A are used to export the related parameters from the selected soil data to the input textbox of the model parameters located in section B. In userform 1 section B (Figure B-3b), option buttons are given to choose whether e_0 and e_c are to be obtained from the laboratory data or to be estimated by using the shrinkage curve and e_c curve, respectively. The model parameters such as $e_N(S_{c,i})$, $S_{c,i}$, C_c , G_s , w_0 , $e_{N,s}$, $C_{r,d}$, $C_{r,s}$ and R_c can be input in this textbox or from the command buttons in section A. The plot of $e_N(S_c)$ and S_c is obtained only from the void ratio with the highest net normal stress from all of the data which were loaded in order to ensure that the void ratio is located on the UCL. However, the estimated $e_N(S_c)$ which is shown as a bold line in this plot is obtained only from the two selected specimens.

a) Spreadsheet program userform 1

a) Spreadsheet program userform 2 (input for shrinkage curve)

b) Spreadsheet program userform 2-net normal stress for interpolation
Figure B-2 Spreadsheet program for the proposed method

Data summary

Author:

Specimen:

☒ Load data from file
☐ Load data from folder

Construct shrinkage curve and ec curve

Specimen K0dSat1

Lab data	
Saturated	
Cc	0.5815
Cr,s	0.12
Gs	2.66
Unsaturated	
eNi	2.7320
Sri	100
Cr,d(Sc=1)	0.1
w0	58.322

Export saturated parameter

Export unsaturated parameter

K0dSat1

K0dSat2
K0dSat3
K0dSat4
Kw1
Kw5
Kw6
Kw9
Kw23s895
Kw30s468
Kw30s556
Kw37s399
Kw42s347
Kw47s252
Kw48s287
Kw57s110
Kw57s81
Kw60s40

a) Userform1 section A

Initial compression curve

P0 (kPa)

e0

☐ Determine from the lab data
☒ Determine from shrinkage curve

ec

☐ Determine from the lab data
☒ Determine from ec curve

Model parameters

eNi(Sci) 2.889 ☐ Check to get water content from the lab data

Sci 4.879 ☐ w0

eNs 2.758 ☐ Show loading curve

Cc 0.592 ☒ Show unloading-reloading curve

Cr,d(Sc=1) 0.066 Rc

Cr,s 0.131

Update

b) Userform1 section B

Analyse results

loading sequence

The estimated compression curve will be constructed following the loading sequence in an incremental form. Same net normal stress in sequence indicate hydraulic change.

load (kPa)

Step

☐ Load sequence from data
☒ Wetting ☐ Drying

51.5(C/WC)
1630.3(C/WC)
51.5(C/WC)
3260(C/WC)

Generate results

Update parameter and generate results

Do analysis for all data

Shrinkage curve

ec curve

Saturation line

Open/close spreadsheet

Shut down

c) Userform1 section C

Figure B-3 Sections in userform1

In section C (Figure B-3c), the net normal stress can be simulated. When the same net normal stress is given, the program will simulate either drying (the soil's water content decreases until degree of saturation equals to 0) or wetting path (the soil's water content increases until degree of saturation equals to 1). "Load sequence from data" check box is given such that the same stress path from the laboratory data is used. In order to estimate the unsaturated compression curve of different soil data, choose the soil data at section A and then click "update parameter and generate results" to use w_0 laboratory data from each specimen or click "generate results" to simulate different water contents. There are three figures which show different state variables relationship which are net normal stress versus void ratio, net normal stress versus degree of saturation and gravimetric water content versus void ratio. When it is desired to analyse the entire data, "Do analysis for all data" button is clicked.

In userform2, parameters which are related to the shrinkage curve such as e_{min} , SL' , G_s , k , can be input (Figure B-2b). Another feature of userform 2 is to construct a contour line from the laboratory data based on the net normal stress. In Figure B-2c, net normal stress starting from 50 kPa, which is doubled each time, i.e. 50, 100, 200, 400, 800 and 1600 kPa, are selected to construct the contour lines. The contour lines are obtained by interpolating data from oedometer tests for different void ratios and water contents. For example, contour line with net normal stress equal to 100 kPa means every water content and void ratio which is subjected to net normal stress 100 kPa is connected. The shrinkage curve is used to estimate the initial void ratio. However, when the shrinkage curve cannot be constructed, the initial void ratio from the laboratory data is then used. The option to choose whether to estimate the initial void from the shrinkage curve or to obtain it from the data is given in userform 1 section B (Figure B-3b).

Vba codes for modelling compression and wetting induced behaviour of unsaturated soil

In this section, the Vba code which is used to calculate some of the important relationships shown in Figure B-1 are given.

In order to estimate the value of $e_N(S_c)$, the Vba code is:

```
Function eNInt(Sri, eNs, eNi, Sr)
    'Sri = known preconsolidation degree of saturation Sc
    'eNs = eN under saturated condition
    'eNi = known eN(Sc) with Sc = Sri
    'Sr = desired Sc for estimation
    If Sri < 100 Then
        eNInt = eNi + (Sr - Sri) / (100 - Sri) * (eNs - eNi)
    Else
        eNInt = eNs
    End If
End Function
```

In order to obtain $P_{c,d}(S_c)$ by using $e_N(S_c)$, e_c and C_c , the following code is used:

```
Function PcFromec(eN, Cc, ec)
    'eN = the intercept of NCL and UCL
    'Cc = slope of the normal compression line. Always get this from saturated test
    'ec = preconsolidation void ratio
    PcFromec = 10 ^ ((eN - ec) / Cc)
End Function
```

The unsaturated compression curve is then given can be modelled as follow:

```
Function WijayaCompressionModel(w0, Gs, e0, ec, Cc, eNs, eNi, Sri, p, Sr, P0)
    'w0 = initial water content
    'Gs = specific gravity
    'ec = preconsolidation void ratio
    'Cc = normal compression line
    'eNs = void ratio of NCL at net normal stress equal to 1 kPa
    'eNi = arbitrary eN from the oedometer test
    'Sri = arbitrary Sr from the oedometer test
    'P = current net normal stress

    Sc = w0 * Gs / ec
    eNPcd = eNInt(Sri, eNs, eNi, WorksheetFunction.Min(Sc, 100))
    PcdSc = PcFromec(eNPcd, Cc, ec)
    If e0 <> ec Then
        Crd = (e0 - ec) / WorksheetFunction.Log10(PcdSc / P0)
    End If
    Ci = (p / PcdSc) ^ 0.5 * Crd

    NLModel = e0 - Ci * WorksheetFunction.Log10(p / P0)
    UCL = eNInt(Sri, eNs, eNi, Sr) - Cc * WorksheetFunction.Log10(p)
    If p < PcdSc Then
```

```

WijayaCompressionModel = NLModel
ElseIf p >= PcdSc Then
WijayaCompressionModel = UCL
End If

```

End Function

For the case where the soil is unloaded, the value of $C_{r,d}$ depends on whether the unloading is obtained prior to $P_{c,d}(S_c)$ or after $P_{c,d}(S_c)$. The following code is used to model $C_{r,d}$:

```

Function CrSchmertmann(Cr, ec, eu, Rc)
'eu = e at unloading
'ec = et at preconsolidation void ratio

If eu > ec Then
CrSchmertmann = Cr * ((ec + 1) / (eu + 1)) ^ Rc
Else
CrSchmertmann = Cr
End If

```

End Function

Thus, unloading can be simply modelled by using the following code:

```

Function WijayaUnloading(ef, pf, p, Cr, ec, Rc)
'ef = void ratio prior to unloading
'pf = net normal stress prior to unloading
'p = current net normal stress
'Cr = slope of unloading line after preconsolidation pressure
'ec = preconsolidation void ratio
'Rc = parameter which is used to reduce the value of Cr when unloading is taken prior to preconsolidation pressure

WijayaUnloading = ef - CrSchmertmann(Cr, ec, ef, Rc) * WorksheetFunction.Log10(p / pf)

End Function

```

When the soil is inundated, the maximum wetting-induced volume change is given as follows:

```

Function SwellingCapacityLinearModel(Crs, Crd, Cc, Pcs, Pcd, p)
'Crs = slope of the unloading line under saturated condition when water can migrate into the soil
'Crd = slope of the unloading under constant water content condition and is a function of Sc
'Cc = slope of the NCL
'Pcs = preconsolidation pressure
'Pcd = apparent preconsolidation pressure
'p = current net normal stress

If p >= Pcd Then
de = Cc * Log10(Pcs / Pcd)
ElseIf p > Pcs And p < Pcd Then
de = Cc * Log10(Pcs) - Crd * Log10(Pcd) + (Crd - Cc) * Log10(p)
ElseIf p < Pcs Then
de = Crs * Log10(Pcs) - Crd * Log10(Pcd) - (Crs - Crd) * Log10(p)
End If

SwellingCapacityLinearModel = de

End Function

```

When the actual change in volume due to the change in water content is desired, the following code is used:

```
Function WettingInducedVolumeChange(Crs, Crd, Cc, eNs, Sri, eNi, Sr, p, emin, edi, Gs, k2, w, ec, Sr0, ec0,
e0, P0, wc)
'CrS = slope of the unloading line under saturated condition when water can migrate into the soil
'CrD = slope of the unloading line under constant water content condition and is a function of Sc
'Cc = slope of the NCL
'eNs = intercept of the NCL
'Sri = arbitrary Sr from the oedometer test
'eNi = arbitrary eN from the oedometer test
'Sr = current degree of saturation
'p = current net normal stress
'emin = minimum void ratio
'edi = void ratio prior to wetting
'Gs = specific gravity
'k2 = curvature parameter of the shrinkage curve
'w = current water content
'ec = preconsolidation void ratio
'Sr0 = degree of saturation prior to wetting
'ec0 = preconsolidation void ratio prior to wetting
'e0 = initial void ratio
'P0 = net normal stress at e0
'wc = water content at ec

Sc = wc * Gs / ec
Pcs = PcFromec(eNs, Cc, ec)
Pcd = 10 ^ ((eNInt(Sri, eNs, eNi, Sc) - ec) / Cc)
dp = WorksheetFunction.Round((p - Pcd), 2) ^ 2
If dp < 0.00001 And dp >= 0 Then
p = Pcd
End If
de = SwellingCapacityLinearModel(Crs, Crd, Cc, Pcs, Pcd, p)

esi = edi + de
If de > 0 And p < Pcd Then
WettingInducedVolumeChange = ActualSwelling(Crs, Crd, Cc, eNs, Sri, eNi, Sr0, p, emin, edi, Gs, k2, w,
ec)
ElseIf de < 0 And p < Pcd Then
WettingInducedVolumeChange = edi + (Sr - Sr0) / (100 - Sr0) * (esi - edi)
ElseIf de <= 0 And p >= Pcd Then
WettingInducedVolumeChange = eNInt(Sri, eNs, eNi, Sr) - Cc * Log10(p)
End If

End Function
```

When drying is taking place under different net normal stresses, the following code can be used:

```
Function ShrinkageCurveUnderDifferentNetNormalStress(Crs, Crd, Cc, ec, eNs, Sri, eNi, Sr, p, esi, eminN,
Gs, k2, w)
'CrS = slope of the unloading line under saturated condition when water can migrate into the soil
'CrD = slope of the unloading line under constant water content condition and is a function of Sc
'Cc = slope of the NCL
'ec = preconsolidation void ratio
'eNs = Intercept of the NCL
'Sri = known preconsolidation degree of saturation Sc
'eNi = known eN(Sc) with Sc = Sri
'Sr = the simulated Sc
'p = net normal stress
```

```

'esi = void ratio prior to drying
'eminN = natural minimum void ratio
'Gs = specific gravity
'k2 = curvature parameter of the shrinkage curve
'w = current gravimetric water content

Pcs = PcFromec(eNs, Cc, ec)
eNdi = eNInt(Sri, eNs, eNi, 0)
Pcd = PcFromec(eNdi, Cc, ec)
de = SwellingCapacityLinearModel(Crs, Crd, Cc, Pcs, Pcd, p)
edi = esi - de
emin = WorksheetFunction.Min(edi, eminN)
w2 = emin * 100 / Gs
m1 = 0
m2 = Gs / 100
If de > 0 Then
ShrinkageCurveUnderDifferentNetNormalStress = ShrinkageCurveN2(emin, w2, m1, m2, k2, w)
Else
ShrinkageCurveUnderDifferentNetNormalStress = esi
End If

End Function

```

- (Da), U. D. O. T. A. (1983) Technical Manual T M 5-818-7, Foundations in Expansive soils. Washington, DC.
- Abelev, Y. M. (1948) The essentials of designing and building on macroporous soils In Proceedings of Stroivoenmorizdat, pp. 203.
- Acikel, A. S. & Mancuso, C. (2010) Use of CO₂ gas to improve saturation of high capacity tensiometer (HCT) In Proceedings of Unsaturated soils (Buzzi, O., Fityus, D., and Sheng, D. (eds)). Taylor & Francis group.
- Aitchison, G. D. (1961) Relationship of Moisture and Effective Stress Functions in Unsaturated Soils. Pore Pressure and Suction in Soils Conf.:47-52.
- Aitchison, G. D. (1965) Moisture equilibria and moisture changes in soils beneath covered areas. A symposium in print, Aitchison (eds) Australia: Butterworths:278.
- Al-Obaidi, Q. J., Ibrahim, S. & Schanz, T. (2013) Evaluation of Collapse Potential Investigated from Different Collapsible Soils. In Multiphysical Testing of Soils and Shales. (Laloui, L., and Ferrari, A. (eds)) Springer Berlin Heidelberg, pp. 117-122.
- Alonso, E. E. (1993) Unsaturated soil: Recent developments and application - constitutive model of unsaturated soils. Civil Engineering European courses programme of continuing education. Barcelona.
- Alonso, E. E., Gens, A. & Josa, A. (1990) A constitutive model for partially saturated soil. *Géotechnique* 40(3)(405-430).
- Alonso, E. E., Pereira, J. M., Vaunat, J. & Olivella, S. (2010) A microstructurally based effective stress for unsaturated soils. *Géotechnique* 60(12):913-925.
- Alonso, E. E., Pinyol, N. M. & Gens, A. (2013) Compacted soil behaviour: initial state, structure and constitutive modelling. In *Géotechnique.*), vol. 63, pp. 463-478.
- Alonso, E. E., Vaunat, J. & Gens, A. (1999) Modelling the mechanical behaviour of expansive clays. *Engineering Geology* 54:173-183.
- Arvidsson, J. & Keller, T. (2004) Soil precompression stress: I. A survey of Swedish arable soils. *Soil and Tillage Research* 77(1):85-95.
- Assallay, A. M., Jefferson, I., Rogers, C. D. F. & Smalley, I. J. (1998) Fragipan formation in loess soils: development of the Bryant hydroconsolidation hypothesis. *Geoderma* 83(1-2):1-16.
- Astm-D422-63 (2007) Standard Test Method for Particle-Size Analysis of Soils.
- Astm-D854-10 (2010) Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer.
- ASTM-D2435/D2435m (2011) Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.).
- ASTM-D3877 (2008) Standard test methods for one dimensional expansion, shrinkage, and uplift pressure of soil-lime mixtures. ASTM Standard.
- ASTM-D4186 (2006) Standard Test Method for One Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading.).
- ASTM-D4253-14 (2014) Standard test methods for minimum index density and unit weight of soils and calculation of relative density. ASTM International, West Conshohocken, PA, 2014 04(08).
- ASTM-D4318-10 (2010) Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
- ASTM-D4546 (2008) Standard test methods for one-dimensional swell or collapse of cohesive soils. ASTM Standard.
- ASTM-D4829 (2011) Standard test method for expansion index for soils. ASTM Standard.
- ASTM-D5333 (2003) Standard test method for measurement of collapse potential of soils. West Conshohocken, PA, ASTM Book of Annual Standards.

- ASTM-D6836-02 (2008) Standard test method for determination of Soil Water Characteristic Curve for Desorption Using Hanging Column, Pressure Extractor, Chilled Mirror Hygrometer, or Centrifuge, ASTM Book of Annual Standards.
- ASTM-D6913 (2009) Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, ASTM.
- Ayadat, T. & Hanna, A. M. (2012) Assessment of soil collapse prediction methods. *IJE* 25(1):19-26.
- Baker, R. & Frydman, S. (2009) Unsaturated soil mechanics: Critical review of physical foundations. *Engineering Geology* 106(1–2):26-39.
- Bao, C., Gong, B. & Zhan, L. (1998) Keynote Lecturer, Properties of Unsaturated Soils and Slope Stability of Expansive Soils. In *Proceedings of the 2nd International Conference on Unsaturated Soils (UNSAT 98)*, .), International Academic, Beijing, China, vol. 1, pp. 71-98.
- Barden, L., Madedor, A. O. & Sides, G. R. (1969) Volume change characteristics of unsaturated clays. *J. Soil Mech. Fdn Engng, Am. Soc. Civ. Engrs* 95 SM1(33-51).
- Becker, D. E., Crooks, J. H. A., Been, K. & Jefferies, M. G. (1987) Work as a criterion for determining in situ and yield stresses in clays. *Canadian Geotechnical Journal* 24(4):549-564.
- Benites, L. A. (1968) Geotechnical properties of the soils affected by piping near the Beson area, Cochise County, Arizona.) University of Arizona, Tucson.
- Berli, M., Casini, F., Attinger, W., Schulin, R., Springman, S. M. & Kirby, J. M. (2015) Compressibility of Undisturbed Silt Loam Soil—Measurements and Simulations. *Vadose Zone Journal* 14(8).
- Bishop, A. W. (1959) The principle of effective stress Lecture delivered in Oslo, Norway. *Teknisk Ukeblad* 39(106):859-863.
- Bjerrum, L. (1967) Engineering Geology of Norwegian Normally-Consolidated Marine Clays as Related to Settlements of Buildings. In *Géotechnique.*), vol. 17, pp. 83-118.
- Blatz, J. A. & Graham, J. (2000) A system for controlled suction in triaxial tests. *Géotechnique* 50(4):465-469.
- Blatz, J. A. & Graham, J. (2003) Elastic-plastic modelling of unsaturated soil using results from a new triaxial test with controlled suction. *Géotechnique* 53(1):113-122.
- Braudeau, E., Costantini, J. M., Bellier, G. & Colleuille, H. (1999) New Device and Method for Soil Shrinkage Curve Measurement and Characterization. *Soil Sci. Soc. Am. J.* 63(3):525-535.
- Bronswijk, J. J. B. (1991) Relation between Vertical Soil Movements and Water-Content Changes in Cracking Clays. *Soil Sci. Soc. Am. J.* 55(5):1220-1226.
- Bs-1377-2 (1990) Methods of test for soils for civil engineering purposes Classification tests. BSI.
- Burger, C. A. & Shackelford, C. D. (2001) Evaluating dual porosity of pelletized diatomaceous earth using bimodal soil-water characteristic curve functions. *Canadian Geotechnical Journal* 38(1):53-66.
- Burland, J. B. & Ridley, A. (1966) The importance of suction in soil mechanics. State of the art lecture In *Proceedings of Twelfth Southeast Asian Geotechnical Conference*, pp. 27-49.
- Burton, G. J., Pineda, J. A., Sheng, D., Airey, D. W. & Zhang, F. (2016) Exploring one-dimensional compression of compacted clay under constant degree of saturation paths. *Géotechnique* 66(5):435-440.
- Burton, G. J., Sheng, D. & Airey, D. (2014) Experimental study on volumetric behaviour of Maryland clay and the role of degree of saturation. *Canadian Geotechnical Journal* 51(12):1449-1455.

- Chertkov, V. Y. (2000) Modeling the pore structure and shrinkage curve of soil clay matrix. *Geoderma* 95(3–4):215-246.
- Chertkov, V. Y. (2003) Modelling the shrinkage curve of the soil clay pastes. *Geoderma* 112(1-2):71-95.
- Chin, K. B., Leong, E. C. & Rahardjo, H. (2010) A simplified method to estimate the soil-water characteristic curve. *Canadian Geotechnical Journal* 47(12):1382-1400.
- Clemence, S. P. & Finbarr, A. O. (1981) Design considerations for collapsible soils. *J. Geotech. Eng. Div., Am. Soc. Civ. Eng* 107:305-317.
- Clevenger, W. A. (1956) Experiences with loess as foundation materials. *Journal of soil mechanics and Foundation Division* 82(3):1025-1026.
- Cornelis, W. M., Corluy, J., Medina, H., Diaz, J., Hartmann, R., Van Meirvenne, M. & Ruiz, M. E. (2006a) Measuring and modelling the soil shrinkage characteristic curve. *Geoderma* 137(1-2):179-191.
- Cornelis, W. M., Corluy, J., Medina, H., Hartmann, R., Van Meirvenne, M. & Ruiz, M. E. (2006b) A simplified parametric model to describe the magnitude and geometry of soil shrinkage. *Eur. J. Soil. Sci.* 57:258-268.
- Crawford, C. B. (1964) Interpretation of the consolidation test. *Journal of the Soil Mechanics and Foundations Division* 90(5):87-102.
- Croney, D., Coleman, J. D. & Black, W. P. M. (1958) Movement and distribution of water in soil in relation to highway design and performance. Washington, DC, pp. 226-252.
- Cui, K., Défossez, P., Cui, Y. J. & Richard, G. (2010) Quantifying the effect of matric suction on the compressive properties of two agricultural soils using an osmotic oedometer. *Geoderma* 156(3–4):337-345.
- Cui, Y., Tang, A., Mantho, A. & De Laure, E. (2008) Monitoring Field Soil Suction Using a Miniature Tensiometer.
- Cui, Y. J., Nguyen, X. P., Tang, A. M. & Li, X. L. (2013) An insight into the unloading/reloading loops on the compression curve of natural stiff clays. *Applied Clay Science* 83–84:343-348.
- Cunningham, M. R., Ridley, A. M., Dineen, K. & Burland, J. B. (2003) The mechanical behaviour of a reconstituted unsaturated silty clay. *Geotechnique* 53(2):183-194.
- Dane, J. H., Cassel, D. K., Davidson, J. M., Pollans, W. L. & Quisenberry, V. L. (1983) Physical characteristics of soil in the southern region — Troup and Lakeland series. In *Southern Cooperative Series Bulletin*. Alabama Agricultural Experiment Station, Auburn University, Auburn, Ala., vol. 262.
- De Marsily, G. (1986) Quantitative Hydrogeology. In *Groundwater Hydrology for Engineers*. Academic, Sand Diego, Calif.
- Denisov, N. Y. (1951) The engineering properties of loess and loess loams In *Proceedings of Gosstroizdat*.
- Derbyshire, E. & Mellors, T. W. (1988) Geological and geotechnical characteristics of some loess and loessic soils from China and Britain: A comparison. *Engineering Geology* 25(2–4):135-175.
- Durner, W. (1994) Hydraulic conductivity estimation for soils with heterogeneous pore structure. *Water Resources Research* 30(2):211-223.
- Esteban, F. (1990) Caracterización experimental de la expansividad de una roca evaporítica.) Universidad de Canabria, Santander (In Spanish), vol. Ph.D thesis.
- Feda, J. (1964) Colloidal activity, shrinking and swelling of some clays. In *Proceedings Soil Mechanics Seminar*., pp. 531–546.
- Feda, J. (1988) Collapse of loess upon wetting. *Engineering Geology* 25(2–4):263-269.
- Feda, J., Boháč, J. & Herle, I. (1993) Compression of collapsed loess: Studies on bonded and unbonded soils. *Engineering Geology* 34(1–2):95-103.

- Fleureau, J. M., Verbrugge, J. C., Huergo, P. J., Correia, A. G. & Kheirbek-Saoud, S. (2002) Aspects of the behaviour of compacted clayey soils on drying and wetting paths. *Canadian Geotechnical Journal of Agricultural Science* 39(6):1341-1357.
- Fredlund, D. G. (1973) Volume change behaviour of Unsaturated Soils.) University of Alberta, Edmonton, vol. Ph. D, pp. 490.
- Fredlund, D. G. (1979) Second Canadian Geotechnical Colloquium: Appropriate Concepts and Technology for Unsaturated Soils. *Can. Geotech. J.* 16.
- Fredlund, D. G. & Gan, J. K. M. (1995) The Collapse Mechanism of a Soil Subjected to One-Dimensional Loading and Wetting. In *Genesis and Properties of Collapsible Soils*. (Derbyshire, E., Dijkstra, T., and Smalley, I. (eds)) Springer Netherlands, vol. 468, pp. 173-205.
- Fredlund, D. G., Hasan, J. U. & Filson, H. L. (1980) The prediction of total heave In *Proceedings of Fourth international conference on expansive soils*, pp. 1-17.
- Fredlund, D. G. & Morgenstern, N. R. (1976) Constitutive Relations for Volume Change in Unsaturated Soils. *Can. Geotech. J.* 13(3):261-276.
- Fredlund, D. G. & Morgenstern, N. R. (1977) Stress state variables for unsaturated soils. John Wiley & Sons, New York:560p.
- Fredlund, D. G. & Rahardjo, H. (1987) Soil mechanics principles for highway engineering in arid regions. pp. 1-11.
- Fredlund, D. G. & Rahardjo, H. (1993) Soil mechanics for unsaturated soils. New York : Wiley, 1993.
- Fredlund, D. G., Stone, J. & Stianson, J. (2011) Obtaining unsaturated soil properties for high volume change oil sands material. *Proceedings of the 5th Asia Pacific conference on unsaturated soils-Pattaya, Thailand* 1:415-420.
- Fredlund, D. G. & Xing, A. (1994) Equations for the soil-water characteristic curve. *Canadian Geotechnical Journal* 31(4):521-532.
- Fredlund, M. D. (2000) The role of unsaturated soil property functions in the practice of unsaturated soil mechanics.) University of Saskatchewan, Saskatoon, vol. Ph. D.
- Fredlund, M. D., Wilson, G. W. & Fredlund, D. G. (2002) Representation and estimation of the shrinkage curve In *Proceedings of 3rd International Conference on Unsaturated Soils, UNSAT 2002*, pp. 145-149.
- Frydman, S. & Baker, R. (2009) Theoretical Soil-Water Characteristic Curves Based on Adsorption, Cavitation, and a Double Porosity Model. *International Journal of Geomechanics* 9(6):250-257.
- Fung, Y. C. (1977) A first course in continuum mechanics. 2nd edition. Prentice-Hall, Englewood Cliffs, New Jersey:340p.
- Futai, M. M. & Almeida, M. S. S. (2005) An experimental investigation of the mechanical behaviour of an unsaturated gneiss residual soil. *Geotechnique* 55(3):201-213.
- Futai, M. M., De Almeida, M. S. S., Conciani, W., Silva & Filho, F. C. (2002) Experimental and theoretical evaluation of plate load test in collapsible soil. In *Proc. 3rd Int. Conf. on Unsaturated Soils (UNSAT 2002)*. Swets and Zeitlinger, Lisse, Recife, Brazil,, pp. 721-726.
- Gallage, C., Kodikara, J. & Uchimura, T. (2013) Laboratory measurement of hydraulic conductivity functions of two unsaturated sandy soils during drying and wetting processes. *Soils and Foundations* 53(3):417-430.
- Gallipoli, D., Wheeler, S. J. & M., K. (2003) Modelling the variation of degree of saturation in a deformable unsaturated soil. *Géotechnique* 53(1):105-11.
- Gens, A., Sánchez, M. & Sheng, D. (2006) On constitutive modelling of unsaturated soils. *Acta Geotechnica* 1(3):137-147.
- Geology.Com (2015) Expansive soil and expansive clay - The hidden force behind basement and foundation problems, See Geology.Com/Articles/Expansive-soil.shtml (Ascend).

- Gibbs, H. J. (1961) Properties which divide loess and dense uncemented soils. Washington D.C.
- Gibbs, H. J. & Bara, J. P. (1962) Predicting surface subsidence from basic soil tests. ASTM Spec. Tech. Publ. 332:231-246.
- Giraldez, J. V., Sposito, G. & Delgado, C. (1983) A General soil volume change equation: I. The two-parameter model. *Soil Sci. Soc. Am. J.* 47(3):419-422.
- Gitirana, G. F. N., Jr. & Fredlund, D. (2004) Soil-Water Characteristic Curve Equation with Independent Properties. *Journal of Geotechnical and Geoenvironmental Engineering* 130(2):209-212.
- Goh, S. G., Rahardjo, H. & Leong, E. C. (2010) Shear Strength Equations for Unsaturated Soil Under Drying and Wetting. *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 136(4):594-606.
- Gould, S., Rajeev, P., Kodikara, J., Zhao, X.-L., Burn, S. & Marlow, D. (2012) A new method for developing equations applied to the water retention curve. *Soil Science Society of America Journal* 76(3):806-814.
- Gould, S. J. F., Kodikara, J., Rajeev, P., Zhao, X. L. & Burn, S. (2011) A void ratio – water content – net stress model for environmentally stabilized expansive soils. *Canadian Geotechnical Journal* 48(6):867-877.
- Grabowska-Olszewska, B. (1988) Engineering geological problems of loess in Poland. *Engineering Geology* 25(2-4):177-199.
- Gray, W. G. & Hassanizadeh, S. M. (1991) Paradoxes and realities in unsaturated flow theory. *Water Resources Research* 27:1847-1854.
- Groenevelt, P. H. & Grant, C. D. (2001) Re-evaluation of the structural properties of some British swelling soils. *European Journal of Soil Science* 52(3):469-477.
- Groenevelt, P. H. & Grant, C. D. (2002) Curvature of shrinkage lines in relation to the consistency and structure of a Norwegian clay soil. *Geoderma* 106(3-4):235-245.
- Guan, Y. & Fredlund, D. G. (1997) Use of the tensile strength of water for the direct measurement of high soil suction. *Canadian Geotechnical Journal* 34(4):604-614.
- Haines, W. B. (1930) Studies in the physical properties of soil - V: The hysteresis effect in capillary properties and the modes of water distribution associated therewith. *Journal of Agricultural Science* 20:97-116.
- Hamberg, D. J. & Nelson, J. D. (1984) Prediction of floor slab heave In *Proceedings of The 5th international conference expansive soils*, pp. 137-217.
- Handy, R. L. (1973) Collapsible Loess in Iowa. *Soil Science Society of America Journal* 37(2):281-284.
- He, L., Leong, E. C. & Elgamal, A. (2006) A Miniature Tensiometer for Measurement of High Matric Suction. (Miller, G. A. (ed)) 147 edn. Reston Va, [Great Britain], American Society of Civil Engineers, vol. 2, pp. 1897-1907.
- Hilf, J. W. (1956) An Investigation of pore-water pressure in compacted cohesive soils.) Tech. Memo. No. 654, U. S. Dep. of the Interior, Bureau of Reclamation, Design and Construction Div., Denver, CO, vol. Ph. D, pp. 654.
- Ho, D. Y. F. & Fredlund, D. G. (1982) Strain rates for unsaturated soil shear strength testing In *Proceedings of Asian geotechnical conference* vol. 1, pp. 787-803.
- Holtz, R. D., Kovacs, W. D. & Sheahan, T. C. (2011) An Introduction to Geotechnical Engineering. 2nd edn., Pearson Education, Inc.
- Honda, M. (2000) Research on the prediction methods for the mechanical behaviours of unsaturated soils.) Kobe University, Kobe, Japan, vol. Ph.D.
- Houlsby, G. T. (1997) The work input to an unsaturated granular material. In *Géotechnique.*), vol. 47, pp. 193-196.
- Howayek, A. E., Huang, P.-T., Bisnett, R. & Santagata, M. C. (2011) Identification and Behavior of collapsible soils. Publication FHWA/IN/JTRP-2011/12. Joint Transportation Research

- Program, Indiana Department of Transportation and Purdue University., West Lafayette, Indiana.
- Hoyos, L., Pérez-Ruiz, D. & Puppala, A. (2012) Modeling Unsaturated Soil Response under Suction-Controlled True Triaxial Stress Paths. *International Journal of Geomechanics* 12(3):292-308.
- Huang, S., Barbour, S. L. & Fredlund, D. G. (1998) Development and verification of a coefficient of permeability function for a deformable unsaturated soil. *Canadian Geotechnical Journal* 35(3):411-425.
- Hung, V. Q. (2003) Uncoupled and coupled solutions of volume change problems in expansive soils. In Department of Civil and Geological Engineering.) University of Saskatchewan, Saskatoon, SK, Canada, vol. Ph. D.
- Hunt, A. G. (2004) Comparing van Genuchten and Percolation Theoretical Formulations of the Hydraulic Properties of Unsaturated Media. *Vadose Zone Journal* 3(4):1483-1488.
- Imhoff, S., Da Silva, A. P. & Fallow, D. (2004) Susceptibility to Compaction, Load Support Capacity, and Soil Compressibility of Hapludox. *Soil Sci. Soc. Am. J.* 68(1):17-24.
- Iwata, S., Tabuchi, T. & Warkentin, B. P. (1995) *Soil-Water Interaction: Mechanism and Application*, 2. 2 edn. France, Loviuser Pub.
- Jamiolkowski, M., Ladd, C. C., Germaine, J. T. & Lancellotta, R. (1985) New developments in field and laboratory testing of soils. In Proc. 11th Int. Conf. on Soil Mech. and Found. Engrg.) A. A. Balkema, Rotterdam, The Netherlands, vol. 1, pp. 57-154.
- Jennings, J. E. B. (1961) A Revised Effective Stress Law for Use in the Prediction of the Behaviour of Unsaturated Soils. *Pore Pressure and Suction in Soils Conf.*(26-30).
- Jennings, J. E. B. & Burland, J. B. (1962) Limitations to the Use of Effective Stresses in Partly Saturated Soils. *Géotechnique* 12 no. 2:125-144.
- Jennings, J. E. B. & Knight, K. (1957a) The additional settlement of foundation due to collapse of sandy soils on wetting. *Proc. 4th Int. Conf. on Soil Mech. and Found. Engrg.* 1:316-319.
- Jennings, J. E. B. & Knight, K. (1957b) The prediction of total heave from the double-oedometer test. *Transactions of the South African Institution of Civil Engineering* 7:285-291.
- Jommi, C. (2000) Remarks on the constitutive modelling of unsaturated soils In *Proceedings of Experimental evidence and theoretical approaches in unsaturated soils* (eds A. Tarantino and C. Mancuso), pp. 139-153.
- Jones, D. E. & Holtz, W. G. (1973) *Expansive Soils-The Hidden Disaster*. Civil. Eng., ASCE(New York):87-89.
- Jotisankasa, A. (2005) Collapse behaviour of a compacted silty clay. In *Civil and Environmental Engineering.*) Imperial College, London, vol. Ph.D, pp. 422.
- Jotisankasa, A., Ridley, A. & Coop, M. (2007) Collapse Behavior of Compacted Silty Clay in Suction-Monitored Oedometer Apparatus. *Journal of Geotechnical and Geoenvironmental Engineering* 133(7):867-877.
- Kabbaj, M., Tavenas, F. & Leroueil, S. (1988) In situ and laboratory stress-strain relationships. In *Géotechnique.*), vol. 38, pp. 83-100.
- Karube, D., Kato, S., Hamada, K. & Honda, M. (1996) The relationship between the mechanical behavior and the state of pore-water in unsaturated soil. *Geotechnical Engineering Journal, JSCE* 535(3):83-92.
- Kayadelen, C. (2008) The consolidation characteristics of an unsaturated compacted soil. *Environmental geology* 54:325-334.
- Khalili, N., Geiser, G. & Blight, G. E. (2004) Effective Stress in Unsaturated Soils: Review with New Evidence. *International Journal of Geomechanics* 4(2):115-126.
- Khalili, N. & Khabbaz, M. H. (1998) A unique relationship for χ for the determination of the shear strength of unsaturated soils. *Géotechnique* 48(5):681-687.

- Kim, D. J., Vereecken, H., Feyen, J., Boels, D. & Bronswijk, J. J. B. (1992) On the Characterization of Properties of an unripe marine clay soil. 1. Shrinkage processes of an unripe marine clay soil in relation to physical ripening. *Soil Sci.* 153(6):471-481.
- Klausner, Y. (1991) *Fundamentals of continuum mechanics of soils*. New York, Springer-Verlag.
- Knodel, P. C. (1981) Construction of large canal on collapsing soils. *J. Geotech. Eng. Div., Am. Soc. Civ. Eng* 107:79-94.
- Kodikara, J. (2012) New framework for volumetric constitutive behaviour of compacted unsaturated soil. *Canadian Geotechnical Journal* 49:1227-1243.
- Kohgo, Y. (2003) Review of constitutive models for unsaturated soils and initial-boundary value analysis. In *Proceedings of the 2nd Asian Conference on unsaturated soils*. (Karube, D., Iizuka, A., Kato, S., Kawai, K., and Tateyama, K. (eds)) *UNSAT-ASIA 2003*, Osaka-Japan.
- Kohgo, Y., Nakano, M. & Miyazaki, T. (1993) Theoretical aspects of constitutive modelling for unsaturated soils. *Soils and Foundations* 33(4):49-63.
- Koorevaar, P., Menelik, G. & Dirksen, C. (1983) *Elements of soil physics*. Developments in soil science. New York, Elsevier.
- Ladd, C. C., Foott, R., Ishihara, K., Schlosser, F. & Poulos, H. G. (1977) Stress-deformation and strength characteristics. In *Proc., 9th Int. Conf. on Soil Mech. and Found. Engrg.*, pp. 421-494.
- Larson, W. E., Gupta, S. C. & Useche, R. A. (1980) Compression of Agricultural Soils from Eight Soil Orders. *Soil Sci. Soc. Am. J.* 44(3):450-457.
- Le, T. T., Cui, Y.-J., Muñoz, J. J., Delage, P., Tang, A. M. & Li, X.-L. (2011) Studying the stress-suction coupling in soils using an oedometer equipped with a high capacity tensiometer. *Frontiers of Architecture and Civil Engineering in China* 5(2):160-170.
- Lee, I. M., Sung, S. G. & Cho, G. C. (2005) Effect of stress state on the unsaturated shear strength of a weathered granite. *Canadian Geotechnical Journal* 42(2):624-631.
- Leong, E.-C., Tripathy, S. & Rahardjo, H. (2003) Total suction measurement of unsaturated soils with a device using the chilled-mirror dew-point technique. In *Géotechnique.*, vol. 53, pp. 173-182.
- Leong, E. C., Lee, C. C. & Low, K. S. (2009) An Active Control System for Matric Suction Measurement. *Soils and Foundation* 49(5):807-811.
- Leong, E. C. & Rahardjo, H. (1997a) Permeability functions for unsaturated soils. *Journal of Geotechnical & Geoenvironmental Engineering* 123(12):1118.
- Leong, E. C. & Rahardjo, H. (1997b) Review of soil-water characteristic curve equations. *Journal of Geotechnical & Geoenvironmental Engineering* 123(12):1106-1117.
- Leong, E. C., Widiastuti, S. & Rahardjo, H. (2013) Estimating wetting-induced settlement of compacted soils using oedometer test. *Geotechnical Engineering Journal of the SEAGS & AGSSEA* 44(1).
- Leong, E. C. & Wijaya, M. (2015) Universal soil shrinkage curve equation. *Geoderma* 237–238(0):78-87.
- Leroueil, S., Kabbaj, M., Tavenas, F. & Bouchard, R. (1985) Stress-strain-strain rate relation for the compressibility of sensitive natural clays. In *Géotechnique.*, vol. 35, pp. 159-180.
- Leroueil, S., Samson, L. & Bozozuk, M. (1983a) Laboratory and field determination of preconsolidation pressures at Gloucester. *Canadian Geotechnical Journal* 20(3):477-490.
- Leroueil, S., Tavenas, F., Samson, L. & Morin, P. (1983b) Preconsolidation pressure of Champlain clays. Part II. Laboratory determination. *Canadian Geotechnical Journal* 20(4):803-816.
- Li, X. (2009) Dual-porosity structure and Bimodal Hydraulic property functions for unsaturated coarse granular soils. In *Civil Engineering.* Hong Kong University of Science and Technology, Hong Kong, vol. Ph.D.

- Li, X., Li, J. H. & Zhang, L. M. (2014) Predicting bimodal soil–water characteristic curves and permeability functions using physically based parameters. *Computers and Geotechnics* 57(0):85-96.
- Lin, Z. (1995) Variation in Collapsibility and Strength of Loess with Age. In *Genesis and Properties of Collapsible Soils*. (Derbyshire, E., Dijkstra, T., and Smalley, I. (eds)) Springer Netherlands, vol. 468, pp. 247-265.
- Lin, Z. G. & Wang, S. J. (1988) Collapsibility and deformation characteristics of deep-seated loess in China. *Engineering Geology* 25(2–4):271-282.
- Lloret, A. & Alonso, E. E. (1985) State Surfaces for Partially Saturated Soils. In *Proc. 11th Int. Conf. Soil Mech. Found. Eng.*, San Francisco, pp. 557-562.
- Lloret, A., Villar, M. V., Sa'Nchez, M., Gens, A., Pintado, X. & Alonso, E. E. (2003) Mechanical behaviour of heavily compacted bentonite under high suction changes. *Géotechnique* 53(1):27-40.
- Lourenço, S. D. N., Gallipoli, D., Toll, D. G. & Evans, F. D. (2009) On the measurement of water pressure in soils with high suction tensiometers. *Geotech. Testing J.* 32(6).
- Lu, N. (2008) Is Matric Suction a Stress Variable? *Journal of Geotechnical and Geoenvironmental Engineering* 134(7):899-905.
- Lu, N. (2011) INTERPRETING THE “COLLAPSING” BEHAVIOR OF UNSATURATED SOIL BY EFFECTIVE STRESS PRINCIPLE. In *Multiscale and Multiphysics Processes in Geomechanics*. (Borja, R. (ed)) Springer Berlin Heidelberg, pp. 81-84.
- Lu, N. & Likos, W. (2006) Suction Stress Characteristic Curve for Unsaturated Soil. *Journal of Geotechnical and Geoenvironmental Engineering* 132(2):131-142.
- Lu, N. & Likos, W. J. (2004) *Unsaturated soil mechanics* Hoboken, N.J. : J. Wiley, c2004.
- Lytton, R. L. (1994) Prediction of movement in expansive clay In *Proceedings of In vertical and horizontal deformations of foundations and embankments* (Yeung, A., and Fealio, G. (eds)). ASCE vol. 40, pp. 1827-1845.
- Mallants, D., Tseng, P.-H., Toride, N., Tinunerman, A. & Feyen, J. (1997) Evaluation of multimodal hydraulic functions in characterizing a heterogeneous field soil. *Journal of Hydrology* 195(1–4):172-199.
- Mantho, A. T. (2005) *Echanges sol-atmosphère application à la sécheresse.* Ecole Nationale des Ponts et Chaussées, French, vol. Ph. D.
- Marinho, F. a. M., Take, W. A. & Tarantino, A. (2005) State of the art presentation on tensiometer and axis-translation technique In *Proceedings of International Symposium on Advanced Experimental Unsaturated Soil Mechanics*.
- Matyas, E. L. & Radhakrishna, H. S. (1968) Volume Change Characteristic of Partially Saturated Soils. *Géotechnique* 18 No. 4:432-448.
- Mbonimpa, M., Aubertin, M. & Bussire, B. (2006) Predicting the unsaturated hydraulic conductivity of granular soils from basic geotechnical properties using the modified Kovacs (MK) model and statistical models. *Canadian Geotechnical Journal* 43(8):773-787.
- McGarry, D. & Malafant, K. W. J. (1987) The analysis of volume change in unconfined units of soils. *Soil Sci. Soc. Am. J.* 51:290-297.
- McKeen, R. G. (1992) A model for predicting expansive soil behavior In *Proceedings of Proceedings of the 7th international conference on expansive soils* vol. 1, pp. 1-6.
- Mendes, J. & Buzzi, O. (2013) New insight into cavitation mechanism in high capacity tensiometers based on high-speed photography. *Can. Geotech. J.* 50:550-556.
- Mesri, G. & Choi, Y. K. (1985) The uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship In *Proceedings of Proc, 11th Int. Conf. on Soil Mech. and Found. Engrg.* A. A. Balkema vol. 1.
- Morgenstern, N. R. (1979) Properties of Compacted Soils. *Proc. of the 6th pan-american Conf. Soil Mech. Found. Eng.* 3.

- Mun, W. & McCartney, J. S. (2015) Compression mechanisms of unsaturated clay under high stresses. *Canadian Geotechnical Journal* 52(12):2099-2112.
- Munoz, J., Castelblanco, Delage, P., Pereira, J. M. & Cui, Y. J. (2011) Some aspect of the compression and collapse behaviour of an unsaturated natural loess. *Geotechnique Letters* 1.
- Murray, E. J. & Sivakumar, V. (2010) *Unsaturated Soils. A fundamental interpretation of soil behaviour.* United Kingdom, Wiley-Blackwell.
- Nash, D. F. T., Sills, G. C. & Davison, L. R. (1992) One-dimensional consolidation testing of soft clay from Bothkennar. In *Géotechnique.*, vol. 42, pp. 241-256.
- Nelson, J. D., Chao, K. C., Overton, D. D. & Nelson, E. J. (2015) *Foundation Engineering for Expansive soils.* New York, NY, John Wiley & Sons, Inc.
- Ng, C. W. W., Cui, Y., Chen, R. U. I. & Delage, P. (2007) The axis-translation and osmotic techniques in shear testing of unsaturated soils: A comparison. *Soils and Foundations* 47(4):675-684.
- Nishimura, T., Koseki, J. & Rahardjo, H. (2011) Effect of density for sand on hysteresis in soil-water characteristic curve In *Proceedings of Unsaturated soils: Theory and Practice* (Jotisankasa, A., Sawangsuriya, Soralump, and Mairaing (eds)).
- Nuth, M. & Laloui, L. (2008) Advances in modelling hysteretic water retention curve in deformable soils. *Computers and Geotechnics* 35(6):835-844.
- O'sullivan, M. F. (1992) Uniaxial compaction effects on soil physical properties in relation to soil type and cultivation. *Soil and Tillage Research* 24(3):257-269.
- Oliveira, O. M. & Marinho, F. a. M. (2008) Suction equilibration time for a high capacity tensiometer. *Geotech. Testinnng J.* 31(1).
- Olsen, P. A. & Haugen, L. E. (1998) New model of the shrinkage characteristic applied to some Norwegian soils. *Geoderma* 83:67-81.
- Othmer, H., Diekkruger, B. & Kutilek, M. (1991) Bimodal porosity and unsaturated hydraulic conductivity. *Soil Science* 152(3):139-149.
- Peng, X. & Horn, R. (2007) Anisotropic shrinkage and swelling of some organic and inorganic soils. *European Journal of Soil Science* 58(1):98-107.
- Peng, X. & Horn, R. (2013) Identifying Six Types of Soil Shrinkage Curves from a Large Set of Experimental Data. *Soil Sci. Soc. Am. J.* 77(2):372-381.
- Peng, X., Horn, R. & Smucker, A. (2007) *Pore Shrinkage Dependency of Inorganic and Organic Soils on Wetting and Drying Cycles Current Address: Institute of Soil Science, Chinese Academy of Sciences, Nanjing, 210008, People's Republic of China All rights reserved. No part of this periodical may be reproduced or transmitted in any form or by any means, electronic or mechanical, including photocopying, recording, or any information storage and retrieval system, without permission in writing from the publisher. Permission for printing and for reprinting the material contained herein has been obtained by the publisher. Soil Sci. Soc. Am. J.* 71(4):1095-1104.
- Pham, H. Q. (2005) A volume-mass constitutive model for unsaturated soils. In *Civil and Geological Engineering.*) University of Saskatchewan, Saskatoon, vol. Doctor of Philosophy, pp. 532.
- Pham, H. Q. & Fredlund, D. G. (2008) Equations for the entire soil-water characteristic curve of a volume change soil. *Canadian Geotechnical Journal* 45(4):443-453.
- Pham, H. Q., Fredlund, D. G. & Barbour, S. L. (2005) A study of hysteresis models for soil-water characteristic curves.
- Phien-Wej, N., Pientong, T. & Balasubramaniam, A. S. (1992) Collapse and strength characteristics of loess in Thailand. *Engineering Geology* 32(1-2):59-72.
- Philip, J. R. (1986) Linearized unsteady multidimensional infiltration. *Water Resources Research* 22(12):1717-1727.
- Prikonski, V. A. (1952) Gruntoedenia-Vtroarid Chast In *Proceedings of Gosgeolzdat.*

- Rahardjo, H. (1990) The study of Undrained and Drained Behaviour of Unsaturated Soil. In Department of Civil Engineering.) University of Saskatchewan, Saskatoon, vol. Ph.D.
- Rahardjo, H., Heng, O. B. & Choon, L. E. (2004) Shear strength of a compacted residual soil from consolidated drained and constant water content triaxial tests. *Canadian Geotechnical Journal* 41(3):421-436.
- Rampino, C., Mancuso, C. & Vinale, F. (2000) Experimental behaviour and modelling of an unsaturated compacted soil. *Canadian Geotechnical Journal* 37:748-763.
- Rassam, D. W. & Cook, F. J. (2002) Predicting the shear strength envelope of unsaturated soils. *Geotechnical Testing Journal* 25(2):215-220.
- Rassam, D. W. & Williams, D. J. (1999) Bearing capacity of dessicated tailings. *Journal of Geotechnical and Geoenvironmental Engineering* 125(7):600-610.
- Richards, B. G. (1966) The Significance of Moisture Flow and Equilibrium in Unsaturated Soils in Relation to the Design of Engineering Structures Built on Shallow Foundations in Australia. Symp. on Permeability and Capillary, Amer. Soc. Testing Materials, Atlantic City, NJ.
- Ridley, A. M. & Burland, J. B. (1993) A new instrument for the measurement of soil moisture suction. *Géotechnique* 43(2):321-324.
- Ridley, A. M. & Burland, J. B. (1999) Use of the tensile strength of water for the direct measurement of high soil suction: Discussion. *Canadian Geotechnical Journal* 36(1):178-180.
- Rijtema, P. E. (1965) An analysis of actual evapotranspiration. Wageningen, C. F. a. P. a. D.
- Rogers, C. D. F., Dijkstra, T. A. & Smalley, I. J. (1994) Hydroconsolidation and subsidence of loess: Studies from China, Russia, North America and Europe : In memory of Jan Sajgalik. *Engineering Geology* 37(2):83-113.
- Romero, E. (2013) A microstructural insight into compacted clayey soils and their hydraulic properties. *Engineering Geology* 165:3-19.
- Romero, E., Gens, A. & Lloret, A. (1999) Water permeability, water retention and microstructure of unsaturated compacted Boom clay. *Engineering Geology* 54(1-2):117-127.
- Romero, E., Vecchia, G. D. & Jommi, C. (2011) An insight into the water retention properties of compacted clayey soils. *Géotechnique* 61(4):313-328.
- Ross, P. J. & Smettem, K. R. J. (1993) Describing Soil Hydraulic Properties with Sums of Simple Functions. *Soil Science Society of America Journal* 57(1):26-29.
- Salager, S., El Youssoufi, M. S. & Saix, C. (2010) Definition and experimental determination of a soil-water retention surface. *Canadian Geotechnical Journal* 47(6):609-622.
- Salager, S., Ferrari, A., Nuth, M. & Laloui, L. (2011) Investigations on water retention behaviour of deformable soils In *Proceedings of Unsaturated Soils-Alonso & Gens (eds)*. Taylor & Francis Group, pp. 485-490.
- Salager, S., Nuth, M., Ferrari, A. & Laloui, L. (2013) Investigation into water retention behaviour of deformable soils. *Canadian Geotechnical Journal* 50(2):200-208.
- Satyanaga, A., Rahardjo, H., Leong, E.-C. & Wang, J.-Y. (2013) Water characteristic curve of soil with bimodal grain-size distribution. *Computers and Geotechnics* 48(0):51-61.
- Schmertmann, J. H. (1955) The undisturbed consolidation behavior of clay. *Transactions of the American Society of Civil Engineers* 120:1201-1233.
- Sheahan, T. & Watters, P. (1997) Experimental Verification of CRS Consolidation Theory. *Journal of Geotechnical and Geoenvironmental Engineering* 123(5):430-437.
- Sheng, D. (2011) Review of fundamental principles in modelling unsaturated soil behaviour. *Computers and Geotechnics* 38(6):757-776.
- Sheng, D., Fredlund, D. G. & Gens, A. (2008) A new modelling approach for unsaturated soils using independent stress variables. *Canadian Geotechnical Journal* 45:511-534.

- Sheng, D. & Zhou, A.-N. (2011) Coupling hydraulic with mechanical models for unsaturated soils. *Canadian Geotechnical Journal* 48(5):826-840.
- Shuai, F. & Fredlund, D. G. (1998) Model for the simulation of swelling-pressure measurements on expansive soils. *Canadian Geotechnical Journal* 35(1):96-114.
- Silva, F.C. F., Almeida, M. S. S. & Futai, M. M. (2002) Critical state modelling of unsaturated soils. In *Proceeding of the third international conference on unsaturated soil.*, Recife, Brazil, pp. 133-137.
- Sivakumar, V., Tan, W. C., Murray, E. J. & McKinley, J. D. (2006) Wetting, drying and compression characteristics of compacted clay. In *Géotechnique.*, vol. 56, pp. 57-62.
- Sivakumar, V. & Wheeler, S. J. (2000) Influence of compaction procedure on the mechanical behaviour of an unsaturated compacted clay. Part 1: wetting and isotropic compression. *Géotechnique* 50(4):359-368.
- Skempton, A. W. (1960) Significance of Terzaghi's concept of effective stress In *Proceedings of From theory to practice in soil mechanics* (Bjerrum, L., Cassagrande, A., Peck, R. B., and Skempton, A. W. (eds)). Wiley.
- Smettem, K. R. J. & Kirkby, C. (1990) Measuring the hydraulic properties of a stable aggregated soil. *Journal of Hydrology* 117(1-4):1-13.
- Smith, C. W., Johnston, M. A. & Lorentz, S. (1997) Assessing the compaction susceptibility of South African forestry soils. II. Soil properties affecting compactibility and compressibility. *Soil and Tillage Research* 43(3-4):335-354.
- Stannard, D. I. (1992) Tensiometer-theory, construction and use. *ASTM International* 15(1).
- Suklje, L. (1957) The analysis of the consolidation process by the isotaches method. In *Proc., 4th Int. Conf. on Soil Mech. and Found. Engrg.*, pp. 200-206.
- Sullivan, R. A. & McClelland, B. (1969) Predicting heave of buildings on unsaturated clay. In *Proceedings of the second international conference on expansive clay soils.* Texas A & M Press, pp. 404-420.
- Sun, D. A., Matsuoka, H., Yao, Y. P. & Ichihara, W. (2000) Three-dimensional elastoplastic model for unsaturated soils. In *Proceedings of the 1st Asian Conference on Unsaturated Soils (UNSAT-ASIA 2000)*. (Rahardjo, H., Toll, D. G., and Leong, E. C. (eds)), Singapore, pp. 153-158.
- Tarantino, A. (2009) A water retention model for deformable soils. In *Géotechnique.*, vol. 59, pp. 751-762.
- Tarantino, A. & De Col, E. (2008) Compaction behaviour of clay. *Géotechnique* 58(3):199-213.
- Tarantino, A. & Mongiovi, L. (2002) Design and construction of tensiometer for direct measurement of matric suction. In *Proc. 3rd Int. Conf. on Unsaturated Soils, Recife.*, vol. 1, pp. 319-324.
- Tarantino, A. & Tombolato, S. (2005) Coupling of hydraulic and mechanical behaviour in unsaturated compacted clay. *Géotechnique* 55(4):307-317.
- Tariq, A. & Durnford, D. S. (1993) Analytical volume change model for swelling clay soils. *Soil Sci. Soc. Am. J.* 57(5):1183-1187.
- Taylor, D. W. (1942) *Research on consolidation of clays.* Cambridge, Mass., Massachusetts Institute of Technology.
- Tekinsoy, M. A., Kayadalan, C., Keskin, M. S. & Soylemaz, M. (2004) An Equation for Predicting Shear Strength Envelope with Respect to Matric Suction. *Computer and Geotechnics* 31(7):589-593.
- Terzaghi, K. (1936) The Shear Resistance for Saturated Soils. *1st Int. Conf. Soil Mech. Found. Eng.* 1:54-56.
- Toll, D. G. (1990) A framework for unsaturated soil behaviour. *Géotechnique* 40(1):31-44.
- Tuffour, Y. A. (1984) Shear strength of partly saturated clays. In *Faculty of Civil Engineering.* Technion, Israeli Institute of Technology, Haifa, Israel, vol. M. Sc.

- Vaid, Y. P., Robertson, P. K. & Campanella, R. G. (1979) Strain rate behaviour of Saint-Jean-Vianney clay. *Canadian Geotechnical Journal* 16(1):34-42.
- Van Genuchten, M. T. (1980) A Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils. *Soil Sci. Soc. Am. J.* 44(5):892-898.
- Vecchia, G. D. & Romero, E. (2013) A fully coupled elastic–plastic hydromechanical model for compacted soils accounting for clay activity. *International Journal for Numerical and Analytical Methods in Geomechanics* 37(5):503-535.
- Wang, L. B. & Frost, J. D. (2004) Dissipated strain energy method for determining preconsolidation pressure. *Canadian Geotechnical Journal* 41(4):760-768.
- Watabe, Y., J.P., L. & Leroueil, S. (2006) Probabilistic modelling of saturated/unsaturated hydraulic conductivity for compacted glacial tills. *Géotechnique* 56(4):273-284.
- Wheeler, S. J., Sharma, R. S. & Buisson, M. S. R. (2003) Coupling of hydraulic hysteresis and stress-strain behaviour in unsaturated soils. In *Géotechnique.*, vol. 53, pp. 41-54.
- Wheeler, S. J. & Sivakumar, V. (1995) An elasto-plastic critical state framework for unsaturated soil. *Géotechnique* 45(1)(35-53).
- Wijaya, M. & Leong, E. C. (2014) Modelling shrinkage behaviour of soft soils. In *Softsoils 2014*. (Rahardjo, P., and Bakar, I. (eds)), Bandung, vol. 2.
- Wijaya, M. & Leong, E. C. (2015a) Estimation of Soil Shrinkage Curve In *Proceedings of Unsaturated soils: Research & Applications*. CRC Press/Balkema vol. 1, pp. 785-789.
- Wijaya, M. & Leong, E. C. (2015b) Swelling and Collapse of Unsaturated Soils Due to Inundation Under One-Dimensional Loading. *Indian Geotechnical Journal*:1-13.
- Wijaya, M. & Leong, E. C. (2016) General equation for unimodal and bimodal soil water characteristic curve. *Soils and Foundations*.
- Wijaya, M., Leong, E. C. & Rahardjo, H. (2014) CRS compression tests of an unsaturated kaolin with pore-water pressure measurement In *Proceedings of Unsaturated soils: Research & Applications* (Khalili, N., Russell, A. R., and Khoshghalb, A. (eds)). CRC Press Taylor & Francis vol. 2.
- Wijaya, M., Leong, E. C. & Rahardjo, H. (2015) Effect of shrinkage on air-entry value of soils. *Soils and Foundations* 55(1):166-180.
- Williams, J. & Shaykewich, C. F. (1969) An evaluation of the polyethylene glycol (P.E.G.) 6000 and P.E.G. (20,000) in the osmotic control of soil water matric potential. *Canadian Journal of Soil Science* 49(3):397-401.
- Wilson, G. V., Jardine, P. M. & Gwo, J. P. (1992) Modeling the Hydraulic Properties of a Multiregion Soil. *Soil Science Society of America Journal* 56(6):1731-1737.
- Wong, H., Morvan, M. & Branque, D. (2010) A 13-parameter model for unsaturated soil based on bounding surface plasticity. *Journal of rock mechanics and geotechnical engineering* 2(2):135-142.
- Xu, Y. (2004) Bearing capacity of unsaturated expansive soils. *Geotechnical and geological engineering* 22(4):611-625.
- Yang, Y. (1989) Study of mechanism of loess collapse. *Science in China (Ser. B)* 32(5):604-617.
- Yoshida, R. T., Fredlund, D. G. & Hamilton, J. J. (1983) The prediction of total heave of a slab-on-grade floor on Regina clay. *Canadian Geotechnical Journal* 20(1):69-81.
- Zhai, Q. & Rahardjo, H. (2012) Determination of soil–water characteristic curve variables. *Computers and Geotechnics* 42(0):37-43.
- Zhang, L.-M. & Chen, Q. (2005) Predicting Bimodal Soil–Water Characteristic Curves. *Journal of Geotechnical and Geoenvironmental Engineering* 131(5):666-670.
- Zhang, X. (2016a) Failure of divide-and-conquer approach in the characterization of unsaturated soils In *Proceedings of Unsaturated soil mechanics from theory to practice* (Chen, Z., Wei, C., Sun, D. A., and Xu, Y. (eds)). CRC Press/Balkema.

- Zhang, X. (2016b) Problems associated with determination of yield curves for unsaturated soils In Proceedings of Unsaturated soil mechanics from theory to practice (Chen, Z., Wei, C., Sun, D. A., and Xu, Y. (eds)). CRC Press/Balkema.
- Zhang, X., Alonso, E. E. & Casini, F. (2016a) Explicit formulation of at-rest coefficient and its role in calibrating elasto-plastic models for unsaturated soils. *Computers and Geotechnics* 71:56-68.
- Zhang, X., Liu, J. & Li, P. (2010) Determining the Shapes of Yield Curves for Unsaturated Soils by Modified State Surface Approach. *Journal of Geotechnical and Geoenvironmental Engineering* 136(1):239-247.
- Zhang, X., Lu, H. & Li, L. (2016b) Use of oedometer equipped with high suction tensiometer to characterize unsaturated soils. *Transportation research record: Journal of the transportation research board* No 2578:58-71.
- Zhang, X. & Lytton, R. L. (2009) Modified state-surface approach to the study of unsaturated soil behavior. Part I: Basic concept. *Canadian Geotechnical Journal* 46(5):536-552.
- Zhou, A.-N., Sheng, D., Sloan, S. W. & Gens, A. (2012a) Interpretation of unsaturated soil behaviour in the stress–saturation space: II: Constitutive relationships and validations. *Computers and Geotechnics* 43(0):111-123.
- Zhou, A.-N., Sheng, D., Sloan, S. W. & Gens, A. (2012b) Interpretation of unsaturated soil behaviour in the stress – Saturation space, I: Volume change and water retention behaviour. *Computers and Geotechnics* 43(0):178-187.
- Zhou, A. N., Sheng, D. & Carter, J. P. (2012c) Modelling the effect of initial density on soil-water characteristic curves. In *Géotechnique*.), vol. 62, pp. 669-680.
- Zhou, W.-H., Yuen, K.-V. & Tan, F. (2014) Estimation of soil–water characteristic curve and relative permeability for granular soils with different initial dry densities. *Engineering Geology* 179(0):1-9.
- Zur, B. (1966) Osmotic control of the matric soil-water potential: I. Soil-water system. *Soil Science* 102(6):394-398.