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SUCTION AND STRENGTH OF UNSATURATED SOILS

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School of Civil and Environmental Engineering

2020

Suction and Strength of Unsaturated Soils

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School of Civil and Environmental Engineering

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

2020

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

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A/Prof. E.C. Leong

Authorship Attribution Statement

This thesis contains material from 3 paper(s) published in the following peer-reviewed journal(s) / 3 from papers accepted at conferences in which I am listed as an author.

Chapter 4 is published as:

- 1) Bulolo, S., Leong, E. C., & Abuel-Naga, H (2021). "Filter paper method for suction measurement using electrical resistivity". *Géotechnique Letters*.

The contributions of the co-author are as follows:

- Prof Leong provided the initial research direction and edited the manuscript.
- I conducted the experiments, co-designed the research procedures with Prof Leong, analysed the data and prepared the manuscript drafts.

Chapter 5 is published as:

- 2) Bulolo, S., & Leong, E. C. (2020). Osmotic Oedometer Using Sodium Chloride Solution and Reverse Osmosis Membrane. *Geotechnical Testing Journal*. <https://doi.org/10.1520/GTJ20190403>.
- 3) Bulolo, S., & Leong, E. C. (2019). Osmotic consolidation of expansive soil. In *7th Asia-Pacific Conference on Unsaturated Soils* (pp. 256–260). Nagoya, Japan

The contributions of the co-author are as follows:

- Prof Leong provided the initial research direction and edited the manuscript.
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- 2) Bulolo, S., & Leong, E. C. (2020). Discussion of “From Basic Particle Gradation Parameters to Water Retention Curves and Tensile Strength of Unsaturated Soils.” *International Journal of Geomechanics* (Under review).

The contributions of the co-authors are as follows:

- Prof Leong provided the initial research direction and edited the manuscript.

- I conducted some of the experiments, co-designed the research procedures with Prof Leong, collected the data from the literature. The model calibration and evaluation were done by me. I also prepared the manuscript drafts and analysed the data.
- Dr Kizza conducted some of the experiments.

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- 3) Bulolo, S., & Leong, E. C. (2021). “Constant suction direct shear tests – experiment, analysis, and shear strength equation”. Geotechnique (Under Review).

The contributions of the co-author are as follows:

- Prof Leong provided the initial research direction and edited the manuscript.
- I conducted the experiments, co-designed the research procedures with Prof Leong, analysed the data and prepared the manuscript drafts.

A manuscript is ready for submission to a journal paper for the remaining part of Chapter 6.

21/June/2021

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Date

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Sam Bulolo

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Summary

The shear strength of soil provides a measure of the maximum stress a soil can carry without failure and its behaviour is important in addressing numerous engineering problems such as bearing capacity and slope stability. The current practice assumes that soils are either saturated or dry. However, soils near the ground surface or in arid and semi-arid regions are normally unsaturated. It is, therefore, important to assess the shear strength of soil in an unsaturated condition.

There are established test procedures; however, they require sophisticated expensive equipment, expertise and long testing durations which has hindered the practical application of the unsaturated soil mechanics in practice. This is the main motivation for this study which aims to investigate new approaches and techniques that address time, availability, accuracy, and cost associated with the current techniques. The study focuses on the experimental use of conventional apparatuses to improve on the existing approaches for estimating unsaturated soil shear strength. This approach will encourage the application of unsaturated shear strength concepts in engineering practice.

Two major aspects of unsaturated soils are studied: suction and strength. These are investigated using reconstituted clay and compacted residual soil. The clay was kaolin – bentonite mixture. The residual soil was obtained from one of the major geological formations of Singapore, that is Bukit Timah Granite.

For suction measurements, the filter paper method in contact and non-contact modes and under initially dry and initially wet conditions were studied. The study observed that if a strict protocol is observed, using filter paper technique is still very useful for suction measurement. The study has also demonstrated that the electrical resistivity of the filter paper can be used to infer the suction. This was only applied to non-contact filter paper method because the electrical resistivity is affected by the salinity of the water. The equilibration times for initially dry and initially wet filter papers were found to be within the same ranges with the initially wet filter paper showing slightly lower equilibration time at high suctions. However, the equilibrium time of initially wet filter paper at high suctions requires more experimental data to support the observations in this study. Good agreement between suction measured using the filter paper method on the as-compacted soil specimens with suction estimated from the corresponding SWCC of the soil specimen was observed. Finally, no filter paper contamination was observed in this study, and this suggests that the sacrificial filter papers in the contact method may not be needed.

The osmotic technique of controlling matric suction using reverse osmosis membrane and sodium chloride solutions has been evaluated. Furthermore, the volume change of soil that occurs due to a difference in salt concentration in the pore water has also been investigated in this study. The study observed that the osmotic technique is still valuable in controlling matric suction of soil if a correct combination of the salt solution and membrane is used. It was observed that the osmotic technique is easy to implement, cheap and higher matric suctions can be easily achieved within a reasonably shorter time as compared to other methods of matric suction control. The study also observed a greater change in volume under matric suction application as compared to changes in salt concentration of the pore water. Hence, matric suction plays a bigger role than the osmotic suction towards the shear strength development in unsaturated soils.

Constant-suction direct shear test for unsaturated soils was reviewed. Data were collated from the literature and re-analysed resulting in the development of a new shear strength equation. The study observed that interpreting the constant-suction direct shear tests by plotting the stress path helps to identify the failure line especially in cases where there is no peak or maximum shear stress with shear displacement. The study also observed that it is better to plot corrected shear stress versus corrected net normal stress for constant-suction direct shear tests with a failure line at a gradient of $\tan \phi'$. For each matric suction, a higher net normal stress can cause the void ratio of soil to decrease and hence ϕ' to increase. The study also proposed a new shear strength equation using suction stress to account for the contribution of matric suction to the shear strength of unsaturated soils.

The shear strength of compacted residual soils was determined using constant water content direct shear tests and interpreted using the extended Mohr-Coulomb failure criterion for unsaturated soils. From constant water content tests, shear strength of unsaturated soils can be obtained by fitting tangent lines inclined at $\tan \phi'$. Considering the data usually available from constant water content direct shear tests, the study proposed a separate expression of suction stress to account for the contribution of matric suction to shear strength. Using the suction stress expression, it was demonstrated that the matric suction of compacted soil can be estimated from CW direct shear tests. The study further observed that as-compacted soils give lower $\tan \phi^b$ values compared to soils dried from wet of optimum compacted condition and hence are useful for determining preliminary design values for conservative design in unsaturated soils.

Indirect tensile strength tests were conducted using the Brazilian tensile strength tests on compacted fine-grained soils. It observed that the compaction method (dynamic or static) does not affect the tensile strength of unsaturated soils. It was also observed that the tensile strength of soil increases with reduction in the degree of saturation, remains almost constant then decreases at very low degrees of saturation. The study observed that the tensile strength of specimens dried from the wet of optimum compacted condition is higher than those of as-compacted specimens at comparable water contents. The study finally evaluated recent tensile strength models incorporating SWCC for unsaturated soils and a new tensile strength model was developed for both coarse and fine-grained soils.

List of Publications

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1. Bulolo, S., & Leong, E. C. (2020). Osmotic Oedometer Using Sodium Chloride Solution and Reverse Osmosis Membrane. *Geotechnical Testing Journal*. <https://doi.org/10.1520/GTJ20190403>
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3. Bulolo, S., & Leong, E. C. (2018). Shear strength of compacted residual soils via constant water content direct shear tests. In *7th International Conference on Unsaturated Soil Mechanics* (pp. 1–6). Hong Kong.

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List of symbols/abbreviations

Symbol /Abbreviation	Definition	Units
A	Cross sectional area of a soil specimen	Square millimetres (mm ²)
a	Fredlund and Xing (1994) SWCC model fitting parameter	Kilopascals (kPa)
a _{aw}	specific air-water interfacial area per pore volume	Square centimetre / cubic centimetre (cm ² /cm ³)
A ₀	Initial cross-sectional area	Square millimetres
α	van Genuchten (1980) model fitting parameter	1/kilopascal (kPa ⁻¹)
AEV	Air-entry value of soil	Kilopascals (kPa)
ASTM	American Society for Testing and Materials	-
BS	British Standards	-
BT	Bukit Timah	-
BTS	Brazilian Tensile Strength	-
c _{app}	Apparent Cohesion	Kilopascals (kPa)
c'	Effective soil cohesion	Kilopascals (kPa)
CBR	California Bearing Ratio	-
C _c	Coefficient of correlation	-
CH	High plasticity clay	-
CMDP (WP4)	Chilled Mirror Dewpoint	-
CS	Constant suction test	-
C _u	Coefficient of uniformity	-
CW	Constant water content test	-
CWDS	Constant water content direct shear test	-
d	Diameter of filter paper	Millimetres (mm)
DC	Direct current	Amperes (A)
DEM	Discrete Element Modelling	-
E	Total energy	Joules (J)
e	void ratio	-
e ₀	Initial void ratio	-
EC	Electrical conductivity	Ohm-metre (Ω.m)

Symbol /Abbreviation	Definition	Units
e_{min}	Minimum void ratio	-
ER	Electrical resistivity	Ohm-metre ($\Omega.m$)
FP	Filter Paper	-
FX	Fredlund and Xing	-
g	Gravitational acceleration	Metres / second squared (m/s^2)
G_s	Specific gravity	-
GPa	Giga pascal	-
GSD	Grain size distribution	-
$G(\theta_0)$	Geometric factor	-
γ_w	Specific weight of water	Kilonewtons / cubic metre (kN/m^2)
HAE	High Air Entry	-
HAEV	High Air Entry Value	-
HCT	High Capacity Tensiometer	-
ISO	International Organization for Standardization	-
KB	Kaolin Bentonite	-
LVDT	Linear Variable Differential Transformer	-
m	A fitting parameter related to residual water content	-
MH	Silt of high plasticity	-
ML	Silt of low plasticity	-
m_π	Osmotic coefficient of volume change.	1/kilo pascal (kPa^{-1})
m_v	Coefficient of volume change	1/kilo pascal (kPa^{-1})
M_w	Mass of water	Grammes (g)
OCR	Over Consolidation Ratio	-
μ	Viscosity of a fluid	Pascal – second (Pa.s)
n	van Genuchten (1980) model fitting parameter	-
p	Pressure	Kilopascals (kPa)
PEG	Polyethylene Glycol	-

Symbol /Abbreviation	Definition	Units
PES	Polyether sulphonated synthetic	-
PH	Potential of Hydrogen	-
PI	Plasticity Index	Percentage (%)
PL	Plastic Limit	Percentage (%)
$P_s(\sigma^s)$	Suction stress	Kilopascals (kPa)
PSD	Particle Size Distribution	-
PVC	Polyvinyl chloride	-
θ	Normalized volumetric water content	-
θ	volumetric water content	-
θ_s	Saturated volumetric	Percentage (%)
R	Universal gas constant	Energy per temperature increment per mole ($L \cdot Pa \cdot K^{-1} \cdot mol^{-1}$)
r	Distance from centre of circular tube from concentric cylindrical surface	Millimetres (mm)
REV	Representative Elementary Volume	-
RH	Relative Humidity	Grammes per cubic metre (gm^{-3})
<i>RMSE</i>	Root mean square error	-
RO	Reverse Osmosis	-
ρ	Resistivity	Ohmmeter ($\Omega \cdot m$)
S	Degree of saturation	-
S_e	Effective degree of saturation	-
S_r	Residual degree of saturation	-
SWCC	Soil Water Characteristic Curve	-
s-SWCC	Soil Water Characteristic curve based on degree of saturation	-
w-SWCC	Soil Water Characteristic curve based on gravimetric water content	-
θ -SWCC	Soil Water Characteristic curve based on volumetric water content	-

Symbol /Abbreviation	Definition	Units
T_k	Absolute temperature	Degrees Celsius ($^{\circ}\text{C}$)
T_s	Surface tension	Newton per Meter (N/m)
t	Time	Hours (hr)
τ	Shear strength	Kilonewtons per metre squared (kN/m^2)
u_a	Pore-air pressure	Kilopascals (kPa)
UHCT	Ultra-High Capacity Tensiometer	-
USCS	Unified Soil Classification System	-
USDA	United States Department of Agriculture	-
u_w	Pore-water pressure	Kilopascals (kPa)
$u_a - u_w$	Matric suction	Kilopascals (kPa)
$\bar{u}_v$	Partial pressure of pore-water vapor	Kilopascals (kPa)
$\bar{u}_{v0}$	Saturation pressure of water vapor over a flat surface of pure water at the same temperature	Kilopascals (kPa)
VG	Van Genuchten	-
w	Gravimetric water content	Percentage (%)
WEV	Water Entry Value	Kilopascals (kPa)
WRC	Water Retention Curve	-
χ	Bishop's effective stress parameter	-
ρ_d	Dry density	Megagrammes per cubic metre (Mg/m^3)
ψ	Soil suction	Kilopascals (kPa)
ψ_r	Suction corresponding to residual water content	Kilopascals (kPa)
σ	Total normal stress	Kilopascals (kPa)
σ'	Effective normal stress	Kilopascals (kPa)
σ_t	Tensile stress	Kilopascals (kPa)
$\sigma - u_a$	Net normal stress	Kilopascals (kPa)

Chapter 1: Introduction

1.1: Background

The shear strength of soil provides a measure of the maximum stress a soil can carry without failure and is important in all geotechnical engineering problems. The current practice assumes that soils are either saturated or dry. However, most soils near the ground surface, either in-situ or compacted, are in an unsaturated condition. It is, therefore, important to assess the shear strength of soil in the unsaturated condition. However, the application of unsaturated soil mechanics to practice is still a challenge due to the long testing duration and relatively high costs required to obtain unsaturated soil properties (Fredlund et al. 2012).

The shear strength of unsaturated soils is dependent on the net confining stress and negative pore-water pressure (suction). The suction is dependent on the degree of saturation and the relationship can be described using the soil-water characteristic curve. In fact, the soil-water characteristic curve has been often used to estimate hydraulic and shear strength properties of unsaturated soils (Fredlund et al. 2012). However, actual soil tests are still preferred especially for shear strength.

The shear strength parameters of unsaturated soils have been investigated using modified triaxial and direct shear apparatuses (Ho and Fredlund 1982, Fredlund et al. 1987, Gan and Fredlund 1988, Escario et al. 1989, Rahardjo et al. 1995, Kato et al. 2011, Schnellmann et al. 2013, Tang et al. 2018). The modification done to the conventional triaxial, or direct shear apparatuses is either for suction control or suction measurement during shearing. However, the triaxial test presents many problems such as difficulty in preparing uniform specimens, long drainage path and the low permeabilities of both the soil and ceramic disk. Triaxial tests have been used to test soils of moderately low permeability, but excessive time may be required if used on soils with low permeability (Ho and Fredlund, 1982).

Escario and Sáez (1986), and Gan and Fredlund (1988) show that control and measurement of suction cannot be effectively maintained during shearing with an increase in net normal stress. Furthermore, Zhang (2015) shows that the stress path of unsaturated soils cannot be controlled effectively in a constant-suction triaxial test during shearing.

The most used suction control technique in laboratory tests is the axis-translation technique (Hilf, 1956). However, the axis-translation technique has drawn criticism that it does not represent the natural condition (Burland and Ridley 1966, Marinho et al. 2005, Vanapalli and Nicotera 2008). The use of the osmotic technique to control suction has several advantages. It induces negative pore-water

condition in the soil specimen representing the natural condition but commonly used cellulosic membrane and PEG solution in the osmotic technique suffer from degradation with time. This makes the technique less effective when the test is conducted over a long period. In addition, it is generally assumed that at equilibrium, the matric suction of the soil specimen is equal to the applied osmotic suction neglecting the membrane effect and stress history of the soil specimen. As a result, alternative osmotic solutions and membranes that can resist degradation need to be investigated.

Constant water content tests have been used to reduce the test duration for unsaturated soils (Bishop and Donald 1961, Satija 1978, Leong et al. 2013, Rasool et al. 2015, Marinho et al. 2016, Jayasundara et al. 2019). The constant water content test represents an atmospheric pressure condition in the field since the pore-air pressure is drained. Lu and Likos (2006) in the formulation of suction stress characteristic curve also explained how shear strength testing using conventional laboratory equipment can be simplified using constant water content test as suction stress can be expressed in terms of water content.

Notwithstanding the disadvantages of the direct shear test (Terzaghi et al. 1996), large shear displacements can be obtained in two directions. Moreover, the thin specimen used reduces the drainage path and testing time for soils of low permeability in the direct shear box as compared to the triaxial setup. For any applied matric suction, the direct shear test requires between 24 – 48 hours for suction equilibration whereas triaxial test requires between 1 to 2 weeks (Gallage and Uchimura 2016). Therefore, the time required for the specimen to attain suction equilibrium before shearing in the triaxial test is longer as compared to the direct shear test.

The use of constant water content in the conventional direct shear apparatus for testing unsaturated soils is still relatively new and different interpretations of the shear strength can be found in the literature. For such tests, suction measurements have been performed before, during and after the test. While the use of constant water content requires matric suction to be measured, research has shown that when a small displacement is adopted, there is self-equilibration and suction is most likely constant (Heitor et al. 2013). Similarly, Tarantino and El Mountassir (2013) demonstrated that a conservative estimate of the unsaturated soil shear strength can be obtained using conventional apparatus without suction control or suction monitoring and constant water content condition. It is, therefore, attractive to investigate the use of constant water content test using a conventional direct shear apparatus to determine the shear strength properties of unsaturated soils for use in practice.

The presence of a tension zone in a cohesive soil was first reported by Terzaghi (1943) who acknowledged the role of tension in soils and how tensile stresses trigger slope failures. Soils near

the ground surface experience multiple drying and wetting cycles according to weather and climatic conditions. As soil dries, it experiences shrinkage and tensile cracks. These cracks affect the integrity of soil structures such as slopes, dams, and embankments in terms of permeability and strength. Cracks in the soil provide an easy pathway for rainwater infiltration and such phenomenon is commonly encountered in slopes and earth structures. Tensile cracks are highly influenced by the tensile strength of soils (Morris et al. 1992, Trabelsi et al. 2011, Vaniceek 2013, Shi et al. 2014, Li et al. 2019, Tang et al. 2019). Research shows that the matric suction of the unsaturated soils mainly influences the tensile strength of unsaturated soils (De Souza Villar et al. 2009, Yin and Vanapalli 2018).

Although tensile strength testing for soil is complex, the effect of cracking in earth structures has attracted several researchers to conduct tensile strength testing of unsaturated soils. Conventional shear and compression test apparatuses cannot measure tensile strength of soils. This has led to new developments in the methodology, configuration, and laboratory test apparatus to evaluate the tensile strength of soils. Several empirical (Kim and Hwang 2003, Munkholm and Kay 2014, Yin and Vanapalli 2018, Wang et al. 2020) and theoretical (Fisher 1926, Morris et al. 1992, Lu et al. 2009, Varsei et al. 2016) models have been developed to estimate the tensile strength of soils. Tensile strength models based on the macromechanics approach compared to the micromechanical approach are more attractive for practical applications (Yin and Vanapalli, 2018). However, the tensile strength models proposed are for either coarse-grained or fine-grained soils. No existing tensile strength model applies to both coarse-grained and fine-grained soils.

1.2: Objectives and Scope

This study aims to investigate the suction and strength properties of unsaturated soils using conventional laboratory apparatus. The scope of this research is summarised as follows:

1.2.1: Suction measurement and control

Several methods for suction measurements are in use but not all methods apply to the whole suction range. Literature shows that the filter paper method is economical, easily accessible, easy to use, relatively accurate and can be used to measure a wide range of suction. The test method has been standardized in ASTM D5298 - 16 however, the test results are operator dependent, different calibration curves have been developed and the precision and bias for reporting results have not been determined. Therefore, the filter paper method is re-evaluated with emphasis on equilibration time, experimental setup, and the calibration curves.

Current suction control techniques are evaluated and the osmotic technique of controlling matric suction is closely examined.

1.2.2: Strength measurements:

Constant suction test is easier to analyse compared to constant water content test as both net normal and suction stresses are controlled in constant suction test. Hence, analysis of constant suction (CS) direct shear tests is first relooked. This study investigates the use of conventional direct shear apparatus to evaluate the shear strength of unsaturated soils using constant water content (CW) tests. While CW direct shear test ideally requires the measurement of suction during the test, this study investigates the use of independent suction measurement before and after the test.

Tensile strength is important in earth structures. Existing tensile strength models formulated under the framework of unsaturated soil mechanics is only either for fine or coarse-grained soils. In this study, tensile strength models especially those developed using unsaturated soil mechanics are evaluated.

1.3: The organization of the thesis

This thesis consists of eight chapters and four appendices:

Chapter One summaries the background, significance, objectives, and scope of the study.

Chapter Two reviews the literature related to unsaturated soil mechanics. It highlights the experimental techniques and theoretical developments of the current state of knowledge in unsaturated soil mechanics and in particular, topics relevant to this thesis. Finally, the research gaps in the literature are identified.

Chapter Three presents the soil materials, specimen preparation, equipment and apparatus used. The test methods are also presented together with any deviations made from the standards.

Chapter Four contains results and discussion on suction measurements and soil-water characteristic curves of the test specimens.

Chapter Five presents results and discussion on the use of osmotic technique to control matric suction and proposes a new osmotic technique to control suction. Effects of osmotic consolidation are also investigated.

Chapter Six presents a re-analysis of constant suction direct shear tests and proposed a new shear strength equation. In addition, results and interpretation of constant water content, direct shear tests are presented.

Chapter Seven presents results and discussion of tensile strength of compacted soils and proposes a new tensile strength model that applies to both fine and coarse-grained soils.

Chapter Eight presents the conclusions of this study together with the recommendations for further research.

Finally, appendices containing either primary data excessive of what could be presented in the main thesis or secondary data to the discussions in the thesis.

Chapter 2: Literature review

2.1: Introduction

In this chapter, the basic concepts of unsaturated soil mechanics are reviewed first, followed by the water retention behaviour, soil suction control and measurement, and strength of unsaturated soils. The experimental techniques and theoretical developments associated with unsaturated soil mechanics that are relevant to the thesis are reviewed. Finally, the research gaps are identified to justify the need for this study.

2.2: Unsaturated soil mechanics

Soils encountered in geotechnical engineering are grouped according to fluids in the pore space within the soil particles. Pore spaces can be either filled or partially filled with liquid (typically water). Therefore, soils with pore spaces full of water are referred to as saturated and when air voids exist in pore spaces, they are referred to as unsaturated. As a result, it is well known that saturated soils have two phases that are solids and liquid whereas unsaturated soils contain an air phase making it three phases for unsaturated soils. The presence of water in the pore results in the pore-water pressures.

Pore-water pressures in the soil become increasingly negative during the dry season resulting in substantive stability of steep slopes. However, rain infiltration during the wet season causes the pore-water pressure to increase and leads to a reduction in the soil's shear strength. Occasionally this infiltration can cause the soil to become fully saturated and all its additional strength from the negative pore-water pressures is lost. The decrease in shear strength will result in ground movements and finally trigger landslides.

Figure 2-1 shows pore-water pressure profiles represented by lines 1, 2 and 3. For a covered ground surface in which any vertical water flow is prevented by the ground surface cover, negative pore-water pressures exist and under hydrostatic conditions with respect to the groundwater table (i.e., line 1). If the cover is removed from the ground, evaporation would cause an upward moisture flow which makes the pore-water pressures more negative (i.e., line 2), while infiltration would cause a downward water flow and makes the pore-water pressures less negative (line 3), (Fredlund et al. 2012).

In tropical regions, natural deposits from the ground surface to several metres below are unsaturated, i.e., the pore-water pressures are negative. The depth of soils with negative pore-water pressures depends on the local climatic conditions. Two extreme assumptions are considered in classical soil mechanics, i.e., soils are either fully saturated or dry.

Figure 2-1 Static equilibrium and steady-state flow conditions in the zone of negative porewater pressures (from Fredlund et al., 2012).

In most geotechnical infrastructural designs and analyses, the unsaturated zone (above the water table) is usually neglected due to the complexity of unsaturated soil mechanics. This results in unrealistic or over designs since the actual field conditions are not considered in the design. Most of the soils encountered in civil engineering works – backfill behind retaining walls, fill material for dams and embankments are compacted and are unsaturated. To incorporate unsaturated soil mechanics in design, engineers need to understand how to determine unsaturated soil properties. The general behaviour of soils can be explained using stress state variables and this is briefly explained in the next section.

2.2.1: Stress state variables of soils

It is generally accepted that two phases are associated with saturated or dry soils, soil particles as solid phase and either liquid phase or gas phase. The mechanical behaviour of saturated soils can be described by a single stress-state variable, i.e., effective stress, $\sigma' = (\sigma - u_w)$, where σ is the normal stress and u_w is the pore-water pressure (Casagrande 1964). Unsaturated soil has three phases, the pore-water pressure being negative in comparison to pore-air pressure. Fredlund and Morgenstern (1977) proposed that the soil-water interface (“contractible skin”) can be treated as a fourth phase for stress analysis within the context of multiphase continuum mechanics. Until today, according to Wijaya 2017, there are disagreements on stress variable(s) concerning:

- Field and laboratory quantification of the stress state variables.
- The transformation from saturated to unsaturated soil mechanics.
- Use of material properties as part of the stress state variables.
- Evaluation of strength, compressibility, and permeability.

The shear strength of unsaturated soils is usually represented by two major approaches that are:

- (i) Single effective stress approach (e.g., Aitchison and Donald 1956, Bishop 1959, Aitchison 1961) and,
- (ii) Two independent stress-state variables approach (e.g., Barden et al. 1969, Fredlund and Morgenstern 1977).

Each approach was developed based on different assumptions and has advantages and disadvantages as discussed in the following sections. A third approach, *the modified stress variable* proposed by Houlsby (1997) will be briefly discussed in the subsequent section.

2.2.1.1: Single effective stress approach

Terzaghi (1936) proposed an effective stress equation reproduced in Equation 2-1 by considering a single stress-state equation for saturated soils.

$$\sigma' = (\sigma - u_w) \quad (2-1)$$

where,

σ' = effective stress, σ = total stress and u_w = pore water pressure.

Equation (2-1) has been extended to unsaturated soil mechanics by (Aitchison and Donald 1956, Bishop 1959, Aitchison 1961). The most common equation proposed by Bishop (1959) for effective stress of unsaturated soils is shown in Equation (2-2). The effective stress for unsaturated soils consists of net normal stress, $(\sigma - u_w)$ and suction, $(u_a - u_w)$ with a coefficient, χ . The coefficient, χ has values ranging from 0 to 1 depending on soil type and the degree of saturation.

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \quad (2-2)$$

where,

u_a = pore air pressure.

The advantage of using a single effective stress approach is that there is a direct transformation from unsaturated to saturated state following the classical soil mechanics approach.

However, the use of Equation (2-2) in unsaturated soils has been considered inappropriate as χ is estimated from soil properties. Coleman (1962) pointed out that χ is related to soil structure and not

volumetric parameters. Fredlund and Morgenstern (1977) argued that variables used to describe stresses should be independent of the material properties. Morgenstern (1979) observed that the value of χ was different and not unique when determined for shear strength and volume change. In addition, Morgenstern (1979) pointed out that χ parameter can be more than one, mainly in the low suction range. Lastly, Bishop et al. (1960) and Jennings and Burland (1962) reported that the parameter χ is hard to determine and quantify experimentally.

Jennings and Burland (1962) argued that the effective stress principle does not apply to unsaturated soils because it does not explain the collapse process upon wetting. They supported the argument by conducting consolidation tests on unsaturated soils and reported that upon inundation of the specimens, they observed a decrease in the soil volume instead of an increase in volume and a reduction in the effective stress. However, Lu (2011) re-interpreted the experimental evidence arguing that the collapse phenomenon cannot be explained by the effective stress concept. Lu (2011) argues that Jennings and Burland (1962) restricted the interpretation of the volumetric changes and stress state before failure without considering post-failure behaviour. Lu (2011) observes from the re-interpretation that all Mohr circles exceeded the limit of Mohr-Coulomb envelope indicating that failure would occur in all cases. Lu (2011) then concluded that Jennings and Burland (1962) oedometer test data was wrongly interpreted because upon inundation, there was a drastic decrease in suction stress, leading to a drastic decrease in effective stress and shear failure.

Until today, despite these debates, the effective stress concept is still used. Khalili and Khabbaz (1998) proposed an empirical relationship for χ which is purportedly sufficient for practical purposes. Hence, additional experimental evidence is required to support the use of the χ parameter.

2.2.1.2: Independent stress-state variables approach

Following the unsuccessful application of the effective stress approach, the independent stress-state variables approach was proposed. (Coleman 1962, Fredlund and Morgenstern 1977) suggested describing the behaviour of unsaturated soils using a combination of any two of the three stress variables. i.e., σ , u_w , u_a . The possible three combinations of the stress-state variables are:

- (i). $(\sigma - u_a)$ and $(u_a - u_w)$,
- (ii). $(\sigma - u_w)$ and $(u_a - u_w)$ and,
- (iii). $(\sigma - u_a)$ and $(\sigma - u_w)$.

where σ is the total stress, u_a is the pore-air pressure and u_w is the pore-water pressure.

From the above three combinations, the net normal stress, $(\sigma - u_a)$ and matric suction, $(u_a - u_w)$ have been the most commonly used stress state variables to describe the behaviour of unsaturated soils

(Fredlund et al., 2012). Matyas and Radhakrishna (1968) presented the concept of “state parameters” in defining the volume change and when represented on a three-dimensional graphical surface as well as the degree of saturation against the net normal stress and matric suction, they observed that two independent constitutive relations were required for the full description of the volume-mass behaviour. Similarly, Barden et al. (1969) also suggested using separate components of net normal stress and matric suction to examine the volume change in unsaturated soils. Additionally, Zhang and Lytton (2006) concluded that if pore-air pressure influences the volume change then a complete set of stress-state variables for unsaturated soils must include pore-air pressure besides net normal stress and matric suction. Tarantino et al. (2000) presented experimental results from the axis translation technique to support the assumption that two effective stresses control the mechanical response of unsaturated soils.

One of the practical advantages of the stress-state variables approach is that u_a , is referenced to the atmospheric pressure and is equal to 0 kPa and thus net normal stress is equivalent to σ . Therefore, when soil approaches full saturation, u_a is approximately equal to u_w and matric suction becomes zero, and consequently, the net normal stress changes into effective stress for saturated soils. In addition, the stress-state variables can be experimentally tested, theoretically justified, and measured in engineering practice (Fredlund et al., 2012). Generally, the effect of one stress-state variable on unsaturated soil is investigated with the other stress-state variable kept constant.

Nonetheless, researchers such as (Khalili and Khabbaz, 1998) have pointed out that using two stress-state variables approach needs extensive testing and long test durations. Furthermore, the test apparatus required to investigate unsaturated soil properties is often sophisticated and very expensive.

For this reason, there is a need for more studies to make testing of unsaturated soils more accessible in terms of cost and time so that the two stress-state variables approach can be widely used in practice.

2.2.1.3: The modified stress variable approach

In the modified stress approach, soil properties are combined with the stress-state variables to give a modified stress variable. This approach tries to combine both the effective stress and independent stress approaches by examining the manner how the net normal stress and matric suction affect soil properties. Houlsby (1997) proposed the concept of modified stress (Equation 2-3a) based on the work conjugate principle for unsaturated soils and modified suction (Equation 2-3b) by multiplying the matric suction with the instantaneous porosity of the soil specimen. Jommi (2000) defined modified stress as skeleton stress and suggested that the degree of saturation represents the volume fraction and should not be considered as a constitutive parameter. Researchers such as (Wheeler et

al. 2003, Tarantino and De Col 2008) adopted the concept and Wheeler et al. (2003) pointed out that other than suction influence, the stress behavior is directly influenced by the degree of saturation.

$$\sigma^* = (\sigma - u_a) + S_r s \quad (2-3a)$$

$$s^* = n_i s \quad (2-3b)$$

where: σ^* = average skeleton stress, s^* = modified suction, n_i = instantaneous porosity and S_r = degree of saturation, s = matric suction. It should be noted that s^* replaces S_r in Equation 2-3a.

Alonso et al. (2013) model the behaviour of unsaturated soils using constitutive stress σ^{**} and effective suction, s^{**} as illustrated in Equations (2.4a) and (2.4b).

$$\sigma^{**} = (\sigma - u_a) + s^{**} \quad (2-4a)$$

$$s^{**} = S'_r s \quad (2-4b)$$

Tarantino and El Mountassir (2013) adopted this approach instead of the independent stress state variable approach because of the reduction in time and the use of conventional equipment in testing. However, the inclusion of soil property in the definition of stress state variable constitutes a limitation for practical reasons. As a result, attempts in this study have been made to use conventional equipment and the modified stress variable approach to investigate the shear strength of unsaturated soils.

Gens (1996) has shown that the behaviour of compacted soil can be satisfactorily reproduced by an elastoplastic constitutive law.

2.2.2: Soil suction.

Soil suction is a vital stress variable for the characterisation of unsaturated soils. A combination of matric and osmotic suction gives the total suction which is generally referred to as soil suction.

2.2.2.1: Components of soil suction.

According to Aitchinson (1964), the different components of soil suction can be defined as:

Matric or capillary component of free energy; “In suction terms, it is the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with the soil water, relative to the partial pressure of the water vapour in equilibrium with a solution identical in composition with the soil water.”

Osmotic (or solute) component of free energy; “In suction terms, it is the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with a solution identical in composition with the soil water, relative to the partial pressure of water vapour in equilibrium with free pure water.”

Total suction or free energy of the soil water; “In suction terms, it is the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with the soil water, relative to the partial pressure of water vapour in equilibrium with free pure water.”

The definitions indicate that the free energy of the soil water corresponds to total suction whereas the matric and osmotic suction are components of the free energy. This can be mathematically represented by Equation 2.5.

$$\psi = (u_a - u_w) + \pi \quad (2-5)$$

where: ψ = total suction, $(u_a - u_w)$ = matric suction and π = osmotic suction

Richards (1965) presented how the free energy of the soil water (soil suction) can be expressed in terms of the partial vapour pressure in the soil water using a thermodynamic relationship as shown in Equation 2.6.

$$\psi = -\frac{RT_K}{v_{w0}\omega_v} \ln\left(\frac{\bar{u}_v}{\bar{u}_{v0}}\right) \quad (2-6)$$

where: ψ = soil suction, R = universal molar gas constant [i.e., 8.31432J/(mol K)], T_K = absolute temperature, v_{w0} = specific volume of water or the inverse of the density of water, ω_v = molecular mass of water vapour, $\bar{u}_v$ = partial pressure of pore-water vapour and $\bar{u}_{v0}$ = saturation pressure of water vapour over a flat surface of pure water at the same temperature.

2.2.2.2: Suction measurement.

The practical application of unsaturated soil mechanics requires either measuring or controlling soil suction. The measurement of soil suction until today is still a big challenge for engineers and scientists due to several limitations.

Suction measurements can be divided into direct and indirect methods. The direct method of measurement involves the direct measurement of negative pore-water pressures whereas indirect methods involve the use of other measured material parameters like water content, resistivity, or relative humidity to infer the suction (Agus and Schanz, 2007).

Table 2-1 shows the various techniques available for measuring the different suctions of soil, each with its advantages, limitations, and specific suction range.

Table 2-1: Techniques for soil suction measurement (Ridley and Wray 1996; Masrouri et al. 2008; Bulut and Leong, 2008; Vanapalli et.al, 2008).

Method	Suction component	Technique	Suction Range (kPa)	Approx. Equilibration time
Indirect	Matric	Contact filter paper	All	7 days-
		Thermal conductivity sensors	0 - 1500	hours-days
		Electrical conductivity sensors	50 - 1500	6-50 hours
		Time domain reflectometry	0 - 1500	Seconds-minutes
		Psychrometer technique	100 - 10000	~1hr
	Total	Relative humidity sensor	100 - 8000	~1hr
		Chilled-mirror hygrometer technique	150 - 30000	10 minutes
		Non-contact filter paper method	All	7 days-
		Squeezing technique	0 -1500	minutes-hours
	Osmotic	Saturation extract technique	0 - 1500	minutes-hours
Direct	Matric	Axis translation technique	0 - 1500	minutes-hours
		Tensiometer / Suction probe	0 – 1500	minutes-hours

Of all the methods summarised in Table 2-1, the most frequently used methods are the *filter paper*, *tensiometer*, *axis translation* and *chilled mirror hygrometer*. A brief explanation of the methods is presented below though full details of soil suction measuring techniques can be found in (Fredlund and Rahardjo 1993, Leong et al. 2003, Bulut and Leong 2008, Fredlund et al. 2012).

(a) Filter paper technique.

Gardner (1937) presented the concept of soil suction measurement using filter paper since placing a soil specimen with a material that absorbs water will cause an exchange of moisture until an equilibrium is almost attained. Several researchers such as (Al-Khafaf and Hanks 1974, Pham and Fredlund 2005, Marinho and Oliveira 2006, Bulut and Leong 2008, Marinho and da Silva Gomes 2012, Bicalho et al. 2015, Leong et al. 2016) have contributed to expanding the general understanding of the filter paper technique.

The filter paper method is an economical, easily accessible, and easy to use method to measure total or matric suction (Leong et al. 2016) and is standardised in ASTM D5298-16. ASTM D5298 - 16 suggests that the time for equilibration should be at least seven days to allow sufficient time for

equilibration. In addition to that, it also reports a possibility of potential organism growth or biological decomposition when filter papers are exposed for more than 14 days to a moist and warm environment. ASTM D5298-16 further suggests the use of sacrificial filter papers, but these are associated with longer equilibration times.

The filter paper method can cover a wide range of suction but has poor accuracy due to difficulty in handling (Houston et al. 1994, Harrison and Blight 1998). When there is direct contact between the filter paper and the soil sample, pore water flow occurs by capillary until equilibrium is achieved, and *matric suction* is measured. If no direct contact exists, the flow of only water vapor occurs (i.e., relative humidity), and *total suction* is measured. The accuracy of the filter paper measurement depends on the calibration curve, and thus the accurate determination of the moisture content of the filter paper is very important.

The quality of filter paper may differ from batch to batch and affect the calibration curve. Hamblin (1981) examined the calibration curve of Whatman No. 42 filter paper produced two years apart and found that the relationships were almost identical. Leong et al., (2002) found the calibration curves obtained for Whatman No. 42 filter papers are almost identical and in agreement with the data of Fawcett and Collis-George (1967). However, Likos and Lu (2002) found that the calibration curves for seven different sets of Whatman No. 42 filter paper used in the non-contact mode vary.

Marinho and da Silva Gomes (2011) reported that contact condition affects the equilibration time of the filter paper. The lesser the contact, the longer the equilibration time making the equilibration of the non-contact filter paper method for suction measurement impossible. Leong et al. (2016) assessed the effect of equilibration time on the calibration curves of initially dry and initially moist Whatman No. 42 filter papers. They concluded that for the non-contact initially dry filter paper, the total suction calibration curve must be used and the calibration curve of non-contact initially moist filter paper is time-dependent. Bulut and Wray (2005) suggested that the filter paper water characteristic curve may change as a result of contact conditions in a pressure plate or within a soil specimen in the contact mode compared with the non-contact mode. Power et al. (2008) investigated the effect of contact stress on the calibration curve of the Whatman No. 42 filter paper in the contact mode and found that contact stress has a significant effect. They recommended that contact stress of 1 kPa will provide sufficient contact without significantly affecting the calibration curve of the filter paper.

Chemical pre-treatment of filter papers to avoid a microbial attack of the filter papers before use has been suggested by researchers (Gardner, 1937; McQueen and Miller, 1968; Al-Khafaf and Hanks, 1974). However, some researchers report that pre-treatment is not necessary (Hamblin 1981,

Chandler and Gutierrez 1986), and alters their absorption characteristics (Marinho and Oliveira, 2006). The latter argues that this might require a different calibration curve.

Recommendations have been made to include the hysteresis effect (where the moisture retention characteristics differ between wetting and drying of a porous material) in the calibration process of filter papers by some researchers (e.g Al-Khafaf and Hanks, 1974; Ridley, 1995). It is worth noting that there is little experimental data to quantify the effect of hysteresis in the filter paper.

Furthermore, Leong et al., (2002) reported the hysteresis to be between 1 and 5% of the filter paper water content which is relatively small within the low suction range. Therefore, an initially dry filter paper is commonly used, and it is assumed that the measurement follows the wetting path hence the hysteresis effect is eliminated. Table 2.2 shows some of the Calibration curves for Whatman No.42 and Schleicher and Schuell No.589 filter papers.

The filter paper method is simple to use but the determination of moisture content of the filter paper requires extra precautions. The handling of the filter paper must follow a strict protocol to avoid accidental moisture transfer. The recommendation is to use gloved hands and picking up the filter paper for moisture content determination using a pair of tweezers (Bulut et al. 2001). The filter paper should be transferred as quickly as possible into a waiting moisture content can and capped. The filter paper should be weighed with an analytical balance to an accuracy of 0.0001g. Such an analytical balance is costly.

In addition, the moisture content of the filter paper is usually only determined once after it has been presumed to be in equilibrium with the soil suction. Hence to overcome the limitations of measuring moisture content of the filter paper, other parameters such as electrical resistivity (ER) of the filter paper should be explored to infer the suction. As electrical resistivity is affected by the salinity of the water, this can only be applied to non-contact filter paper method.

Table 2-2: Calibration curves for Whatman No.42 and Schleicher and Schuell No.589 filter papers

Whatman No.42 filter paper			
Reference	Suction type Matric/Total	Unit	Suction equation
Hamblin (1981)	All	MPa	$\ln\psi = 2.397 - 3.683\ln w_f$
Chandler and Gutierrez (1986)	All	pF	$\psi = 5.85 - 0.0622 w_f, w_f > 47$
Greacen et al. (1987)	All	kPa	$\log\psi = 5.327 - 0.0780 w_f, w_f < 45.3$ $\log\psi = 2.413 - 0.0135 w_f, w_f > 45.3$
Leong et al. (2002)	Matric	kPa	$\log\psi = 2.909 - 0.0229 w_f, w_f \geq 47$ $\log\psi = 4.945 - 0.0673 w_f, w_f < 47$
	Total	kPa	$\log\psi = 8.778 - 0.222 w_f, w_f \geq 26$ $\log\psi = 5.31 - 0.0879 w_f, w_f < 26$
ASTM D-5298-16	All	kPa	$\log\psi = 5.327 - 0.0779 w_f, w_f \leq 45.3$ $\log\psi = 2.412 - 0.0135 w_f, w_f > 45.3$
Schleicher and Schuell No. 589 filter paper			
McQueen and Miller (1968)	All	kPa	$\log\psi = 5.238 - 0.0723 w_f, w_f < 54$ $\log\psi = 1.8966 - 0.1025 w_f, w_f > 54$
Al-Khafaf and Hanks (1974)	All	kPa	$\log\psi = 5.117 - 0.0337 w_f, w_f < 83$ $\log\psi = 1.983 - 0.0090 w_f, w_f > 83$
Greacen et al. (1987)	All	kPa	$\log\psi = 5.058 - 0.0688 w_f, w_f < 54$ $\log\psi = 1.882 - 0.0102 w_f, w_f \geq 54$
McKeen (1988)	All	kPa	$\log\psi = 4.9 - 0.0624 w_f, w_f < 66$ $\log\psi = 1.25 - 0.0069 w_f, w_f \geq 66$
	Matric	kPa	$\log\psi = 2.659 - 0.018 w_f, w_f \geq 54$ $\log\psi = 5.438 - 0.069 w_f, w_f < 54$
Leong et al. (2002)	Total	kPa	$\log\psi = 8.778 - 0.191 w_f, w_f \geq 32$ $\log\psi = 5.26 - 0.0705 w_f, w_f < 32$
ASTM D-5298-16	All	kPa	$\log\psi = 5.056 - 0.0688 w_f, w_f \geq 54$ $\log\psi = 1.882 - 0.0102 w_f, w_f < 54$

* w_f , filter paper water content in %, All - one curve for both total and matric suction

(b) Chilled mirror dew-point device

The chilled mirror dew-point device (Figure 2-2) is one of the indirect methods for suction measurement. The suction is computed from the measured relative humidity of a soil sample. A study on compacted soil by Leong et al. (2003) reported an equilibration time of about 18 minutes. Agus and Schanz (2005) reported the chilled mirror method to have the greatest accuracy over the entire total suction range compared to three other methods: filter paper method, psychrometer method, and the relative humidity sensor. However, due to the high degree of error at low suction levels, the chilled mirror dew-point device is typically used only when suctions are above 1000 kPa (Leong et al., 2003; Bulut et al., 2008; ASTM D6836-16).

The relative humidity of the soil specimen is measured in the chilled-mirror dew-point device. The device consists of a mirror and a detector above the sample chamber where the soil is placed. The mirror temperature in the chamber is precisely controlled by a thermoelectric cooler to maintain a constant dew thickness. The sample temperature is set by the temperature controller measured by an infrared thermometer. Vapor equilibration is facilitated by a fan that circulates air within the sample chamber. The relative humidity measurement is taken to an accuracy of $\pm 0.1\%$.

Figure 2-2: Schematic of the chilled-mirror dew-point device (from Leong et al., 2003).

A plastic sample cup is half-filled with a soil sample and placed into the sample chamber, which is sealed, thus allowing the water in the soil to vaporize and condense on the mirror. Through the equilibration of the humidity of the soil sample and air within the chamber, the relative humidity is measured.

The total suction is computed from the thermodynamic relationship between total suction and the partial pressure of pore water vapour as in Equation 2.7.

$$\psi = - \frac{RT}{v_{w0}\omega_v} \ln \left(\frac{u_v}{u_{v0}} \right) \quad (2-7)$$

where: ψ = soil suction or total suction (kPa), R = universal (molar) gas constant (8.31432 J / (mol

K)), T = absolute temperature (i.e. $T = (273.16 + t^{\circ}\text{C}) \text{ K}$), t = temperature in $^{\circ}\text{C}$, v_{w0} = specific volume of water or the inverse of the density of water (i.e. $1/\rho_w \text{ m}^3 / \text{kg}$), ρ_w = density of water (i.e. 998 kg/m^3 at $t = 20^{\circ}\text{C}$), w_v = molecular mass of water vapour (i.e. 18.016 kg/kmol), u_v = partial pressure of pore vapour (kPa), and u_{v0} = saturation pressure of water vapour over a flat surface of pure water at the same temperature (kPa). The term u_v/u_{v0} = relative humidity (R_h) and is entered as a fraction in Equation (2-7).

Detailed procedure and explanation on the Chilled mirror device can be found in (ASTM D6836-16, Leong et al. 2003 and Fredlund et al., 2012).

(c) Squeezing technique (pore fluid squeezer)

The electrical conductivity can be indirectly used to compute the osmotic suction of soil (i.e. the inverse of electrical resistivity) using a small volume of extracted soil pore-water (Fredlund et al. 2012). The electrical conductivity of soil can be measured from a more dilute suspension of soil or saturated soil paste but it is better to use the pore-water extract for measuring conductivity to estimate the salt concentration (Leong et al. 2003).

Some methods such as gas extraction method, centrifuging, saturation extraction or dilution, immiscible liquid displacement, leaching, and pressurised squeezing can be used to extract pore-water from soil samples (Iyer 1990). Different methods are suitable for different soils. For example, the squeezing technique can be used to get free pore-water from a sample with dissolved salts from the soil (Fredlund et al., 2012).

Squeezing techniques using a pore fluid squeezer (Figure 2-3) are most widely used as they are relatively simple and inexpensive, and they can produce reliable pore-water samples. The pore fluid squeezer comprises a thick-walled cylinder and a T-shaped piston (Ram). The soil specimen is placed into the cylinder and pressures of about 30 MPa are applied on the piston using a compression machine to extract the pore water sample and collected using a syringe.

De Lange et al. (1992) reported several factors that influence the reliability of the pore-water data such as de-compressional effect, period, and condition of storage of the sediment (specimen) before pore-water extraction, squeezing pressure, temperature and oxygen contamination during pore-water extraction. From their study, it was concluded that:

- a) The de-compressional effects are specific to carbonate-rich sediments,
- b) Long storage time before pore-water extraction or storage under inadequate conditions, oxygen contamination, and temperature deviations may cause changes in pore-water composition, and

c) Large differences in the pressure of extraction did not change the pore-water composition.

Although the above conclusions were made, they did not give recommendations on the appropriate handling temperatures and adequate oxygen conditions.

Masuzawa et al. (1991) reported that in-situ extraction methods are considered the best possible way of collecting pore-water. However, in-situ methods require long sampling times, the depth of sampling is limited to 3 metres, few samples can be retrieved, associated solid phase is not recoverable and the equipment is unpredictable and expensive (De Lange et al., 1992, Pan et al., 2010). The osmotic suction is estimated from the electrical conductivity using a graph relating to osmotic pressure and electrical conductivity developed for pore water containing dissolved salts as shown in Figure 2-4.

Figure 2-3: Schematic of a Pore fluid squeezer (modified from Fredlund and Rahardjo, 1993).

Figure 2-4: Relationship of electrical conductivity to osmotic suction for various salt concentrations (modified from Leong et al., 2003).

(d) High capacity tensiometers

Tensiometers are one of the laboratory direct matric suction measurement methods. Tensiometers usually measure negative pore-water pressures whereas the air pressure is atmospheric. Before the work of Ridley and Burland (1993), most matric suction measurements were less than 100 kPa because of cavitation. Cavitation can occur once the water tension surpasses 100 kPa and tends to ranges of negative absolute pressures. This results in the formation of a vapour phase of a liquid (boiling) a process referred to as cavitation.

Ridley and Burland (1993) developed a tensiometer that could measure matric suctions up to 1500 kPa hence the name high capacity tensiometer (HCT). It was designed to delay cavitation by the use of high air entry value ceramic disks up to 1500 kPa and reducing the surface area and volume of the reservoir. This would allow the water in the reservoir to realise tensile strength as shown in Figure 5 (a). A modification was done by Ridley and Burland (1995) to eliminate some materials such as the O-rings that could facilitate cavitation as shown in Figure 5 (b).

Since then, other HCT have been developed by other researchers such as (Guan and Fredlund 1997, Tarantino and Mongiovi 2003, Lourenco et al. 2006, Mancuso and Papa 2014). Recently, an Ultra-High Capacity Tensiometer (UHCT) has been developed by Mendes et al. (2019) to measure suction up to 7.3 Mpa. In their study, a Nano-porous glass is used to replace the ceramic disk. However, to date, there is less or no experimental data to support suction measurements up to the reported capacity of the UHCT.

Figure 2-5: High capacity tensiometer modified from Ridley and Burland (1993, 1995).

Typical of all HCT are i) the high air entry value (HAEV), ceramic disk ii) water reservoir and iii) the strain gauge. The soil specimen is separated from the water reservoir by a saturated HAEV ceramic disk which allows water exchange between the soil and the water reservoir until equilibrium is attained. The water exchange (when soil draws water from the reservoir) will cause movements on the water reservoir which is captured by the strain gauge and the soil suction is recorded.

There are several challenges associated with the HCT among which includes the following:

- (i) Cavitation will disrupt all measurements once it happens, and this will call for re-saturation of the HAEV disk which takes a few hours depending on the air entry value of the disk being used in the HCT. Other errors associated with inadequate saturation of the ceramic disk can be found in Tarantino (2004).
- (ii) Contact, there needs to be good contact between the soil pore-water and the HAEV of the HCT. However, the effect due to contact has been reduced by the application of a slurry prepared at a water content close to the liquid limit at the tip of the tensiometer.
- (iii) Interpretation of results, discriminating bad measurements from good because it is not apparent to tell when to stop recording a true suction value (Take and Bolton 2003, Tarantino 2004).
- (iv) Long term measurement, it is hard to determine long term suction measurements because of air diffusion. The application of high positive water pressures may reduce the amount of entrapped air but cannot completely, eliminate it.

Figure 2-6 shows a typical equilibrium of the high-capacity tensiometer reported by Lourenço et al. (2009) which shows three different possible scenarios. Scenario (a) when the response time is equal

to the time of equilibrium. This is regarded as the normal equilibrium where the matric suction is reached gradually as the pore water pressure decreases from zero. Lourenço et al. (2009) suggest that with this type of equilibrium, the sample will have attained pore-water pressure equalisation earlier before measurement.

Scenario (b), there exists a sudden decrease in the pore-water pressure followed by an increase then the equilibrium is attained. Though Lourenço et al. (2009) attributed the behaviour to non-equilibration of the pore-water pressures within the specimen, however, the pore-water pressure equilibration in a soil specimen requires less than an hour. Wijaya (2017) attributed the behaviour to inadequate contact between the HCT and the soil specimen. The inadequate contact may cause the HCT to be exposed to evaporation which causes the HCT to register a higher negative pore-water pressure. However, with time, the amount of water between the soil specimen and the HCT increases. Once there is sufficient water to connect the HCT with the soil specimen, the HCT response is reversed and it will record the correct pore-water pressure of the soil specimen.

Scenario (c), there is almost no equilibrium obtained as the pore-water pressure keeps decreasing even after several hours. Lourenço et al. (2009) attributed this to specimens being surrounded by large air gaps during the measurement which aided evaporation from the soil specimen.

Notwithstanding the challenges, tensiometers can be used to measure suction directly and results are obtained in a short time. For this reason, the use of HCT has been incorporated in equipment such as direct shear (Caruso and Tarantino 2004, Jotisankasa and Mairaing 2010, Shwan 2016), oedometer cells (Monroy et al. 2007, Wijaya and Leong 2016a, Bagheri et al. 2018), and triaxial cells (Meilani et al. 2002, Muñoz-Castelblanco et al. 2012, Marinho et al. 2013, 2016).

In this study, matric suction measurements have been conducted using HCT developed by Wijaya and Leong (2016).

(a)

(b)

(c)

Figure 2-6: Equilibrium of the high-capacity tensiometer (Lourenço et al. 2009)

(e) Axis translation technique

The axis translation technique is commonly used in the laboratory to measure negative pore-water pressure to avoid the problem of cavitation associated with the water pressure measurement system. The axis translation technique is used for suction measurement and control in laboratory tests on either disturbed or compacted specimens over one atmosphere (100 kPa). The technique was established by Hilf (1956) using the apparatus illustrated in Figure 2-7, to measure matric suction of samples taken from the field.

An unsaturated soil specimen was placed in a closed pressure chamber and a pore-water pressure probe was inserted into the soil. The probe was connected by a tube filled with de-aired water to a null type pressure measuring system. The Bourdon gauge began registering a negative pressure the moment the probe was inserted in the specimen because the water in the tube tended to go into tension (Fredlund et al., 2012). The movement of the water in the measuring system towards the specimen was slowed and eventually reversed by increasing the air pressure in the pressure chamber until equilibrium was observed by the mercury plug. The matric suction was recorded as the difference between the pore-water pressure and the chamber air pressure.

Figure 2-7: Null-type, axis translation device used to measure negative pore-water pressures (from Hilf, 1956).

The axis translation technique has been used by other researchers (Fredlund 1989, Leong et al. 2003, Tripathy et al. 2012). The main modifications of the test equipment compared to that in Figure 2-8 includes a high air-entry disk and a water compartment below the high air-entry disk, but the working principle remains the same.

The upper limit of matric suction that can be measured by the apparatus in Figure 2-8 is governed by the air-entry value of the ceramic disk. High air-entry disks up to an air-entry value of 1500 kPa are commercially available. It has been observed by Bocking and Fredlund (1980) that the axis translation technique is best for soils with a continuous water phase and the presence of air bubbles in the soil specimen may result in an overestimation of matric suction. Air diffusing through the air-entry disk can cause underestimation of the matric suction. Bocking and Fredlund (1980) also suggest that some considerations should be done while interpreting the axis translation results for example.

1. The asymptotic nature of the transient flow of pore-water, flow equilibrium will be approached but, it can never be attained.
2. Air diffusion into the ceramic disc imposes a practical limit on the time for the test to run and should be measured for each axis translation apparatus.
3. The peak should not be taken as the actual suction value because it depends on the compressibility of the soil and the rate at which the chamber pressure is applied, and the suction value can temporarily overshoot.

Figure 2-8: Modified chamber for axis translation technique (from Fredlund et al., 2012).

Several limitations have been reported to be associated with the axis-translation technique among which they include: i) the pressures are not representative of the field conditions since the air pressure is greater than atmosphere and pore-water pressure is negative, ii) It is only valid for a certain range of degree of saturation since it only applies when the air-water interphase is continuous, iii) air

diffusion through the ceramic disks affects the test duration as this requires periodic flushing of the system (Vanapalli and Nicotera 2008).

2.2.2.3: Suction control techniques

For unsaturated soils, depending on the application, there is a need to apply or control suction. Different methods of suction control and application in laboratory testing of unsaturated soils. Some of the techniques are briefly described in this section.

(a) Axis translation technique

The details of the setup, operation, precautions, and limitations are similar to those while using the axis translation method for suction measurement and have been discussed in section 2.4.2.5.

In this technique, an air pressure u_a (pore-air pressure) is applied to the specimen while maintaining the pore-water pressure, u_w at atmospheric pressure or a small positive pressure via a high air-entry ceramic disk. The matric suction, S , is given as the difference between pore-air and pore-water pressures i.e., $u_a - u_w$. This method is usually used for unsaturated soil characterization for soil suctions between 0 and 1500 kPa (Fredlund and Rahardjo 1993).

The axis translation technique has been applied in geotechnical testing using triaxial apparatus (Ho and Fredlund 1982, Rahardjo et al. 1995, Thu et al. 2006, Goh et al. 2014), direct shear apparatus (Gan et al. 1988, Nam et al. 2011, Chen et al. 2013, Tang et al. 2018) and oedometer (Matyas and Radhakrishna 1968, Kassiff and Shalom 1971, Delage et al. 1992, Delage and Cui 2008, Nowamooz and Masrouri 2008, Ajdari et al. 2016, Derfouf et al. 2020).

(b) Vapour equilibration technique

In this technique, the principle is to create a controlled relative humidity condition and hence control soil suction. Constant suction environments can be generated in closed chambers through the application of chemical solutions with osmotic potential such as a saturated salt solution or an unsaturated acid solution in sealed containers. Figure 2.9 shows an illustration of a glass desiccator containing a salt solution at the bottom and a porous disk for suspending the soil specimens.

There will be a water exchange within the desiccator headspace between the vapour (from the salt solution) and liquid phases (from the soil specimens) until equilibrium is reached. The vapour pressure within the desiccator at equilibrium is related to the solution concentration.

Two commonly used solutions include acid and saturated salt solutions. Saturated salts solutions are advantageous because the osmotic salt concentration of the solution remains constant during the water exchange between the vapour environment and the specimens. However, the suction control is limited because of the low ranges of suction values (0 – 10 MPa) (Blatz et al. 2008). Unsaturated acid

solutions can be used to impose higher suctions, but the target suction value may not be attained because during water exchange, the concentration of the osmotic solution changes. When higher suctions are applied, small changes in the concentration can greatly vary the value of the target suction leading to errors.

Figure 2-9: Soil specimens in a constant environment (from Leong and Rahardjo 2002).

In practice, a correction is applied to establish the equilibrium suction by computing the change in the concentration of the osmotic solution or assuming that the water loss/intake by the specimen is extra/reduction from the aqueous solution.

The vapour equilibrium technique is easy to use, and higher suction values can be applied however it has some hindrances such as i) long equilibration times required, ii) accuracy, in low suction ranges, iii) effects of temperature variations and, iv) errors due to incomplete dissolution of the solute, water evaporation and water spillage (Blatz et al. 2008).

It is worth noting that the vapour equilibration technique controls total suction, not matric suction. This method has been used by several researchers such as (Tang and Cui 2005, 2006, Pintado et al. 2009, Sun et al. 2014). A detailed assessment of the vapour equilibration technique can be found in (Leong et al. 2009).

(c) Osmotic technique

This technique was developed by biologists (Lagerwerff et al. 1961) and introduced to soil science by (Zur 1966) and geotechnical engineering by (Kassiff and Shalom 1971). The technique has been applied to oedometer tests (e.g. Esteban 1990, Oteo Mazo et al. 1996). The osmotic technique should not be confused with osmotically-induced or osmotic consolidation (Barbour and Fredlund 1989, Bulolo and Leong 2019) where salt solution replaces the pore water in a saturated soil specimen causing volume change. Osmotically-induced or osmotic consolidation test is usually conducted in an oedometer without the use of a semi-permeable membrane (e.g., Di Maio 1996). A semi-permeable

membrane is used to separate the soil specimen and an aqueous solution of large-sized polyethylene glycol (PEG) molecules. The water flows through the membrane by a process of osmosis and the water transfer occurs in the liquid phase. Figure 2.10 shows the application of osmotic suction to control matric suction (osmotic technique) under no total stress.

Figure 2-10: Application of osmotic technique under no total stress (modified from Cui and Delage 1996).

The osmotic technique uses a simpler set up as there is no need for air pressure control as required in axis translation technique or circulation of air at a constant relative humidity as used in the vapour equilibration technique. Whereas in the vapour equilibration technique, a total suction is applied, in the osmotic technique, a matric suction is applied to the soil specimen. In the osmotic technique, the pore-water pressure is negative, and it reproduces the natural pore-water condition in soils, unlike the axis translation technique.

The axis translation technique and vapour equilibration technique require the soil specimen to be placed in a sealed chamber. Although the osmotic technique does not require a sealed chamber to conduct the test, it is not as popular as the axis translation technique and the vapor equilibration technique because of degradation of the salt solution and membrane used in the osmotic technique. The common salt solution used in the osmotic technique is an aqueous solution of polyethylene glycol (PEG) (a polymer-based solute) which has several molecular weights and may break down during long-duration tests (McGary 1960, Thill et al. 1979, Delage and Cui 2008a, Ulbricht et al. 2014). The

calibration curves given for a particular molecular weight PEG solution are also shown to be different for different researchers. This is illustrated for PEG 20000 solution in Figure 2-11. Possible causes of the degradation of the PEG solution includes oxidative degradation in the presence of air (Hans et al. 1995), degradation associated with interactions of molecules of PEG with the semi-permeable membrane used and with expelled ions from the pore fluid of soil-water systems (Tripathy et al. 2011), and degradation by soil microorganisms (Haines and Alexander, 1975) that could pass through the cellulosic membrane. Inorganic solutes (e.g., sodium chloride) have the advantages of being readily available and inexpensive as compared to polymer-based solutes (e.g., PEG). The polymer-based solutes are characterized by low osmotic driving force (Klaysom et al. 2013).

Unlike sodium chloride (NaCl) solution, the PEG solution consists of molecules with a range of molecular weights. For example, PEG 20000 solution consists of molecules of molecular weights ranging from 15000 to 20000 (Williams and Shaykewich 1969, Steuter et al. 1980, Delage and Cui 2008a). The hydrodynamic diameter of the PEG molecule also changes with the concentration (Michel and Kaufman 1973, Minagawa et al. 1994). In addition, regardless of the technique used to measure osmotic pressure (either vapour point deficit method or freezing point depression method), NaCl solution gives the same value but not PEG solution (Figure 2-12). The semi-permeable membrane commonly used in the osmotic technique together with the PEG solution is a cellulosic membrane that is prone to degradation during tests.

(Zur 1966) reported the difficulty encountered due to microbial decomposition of the cellulosic membrane when in contact with soil despite the use of bactericides and disinfectants. Tadza et al. (2016) reported that the addition of penicillin was not effective in removing the cellulose acetate degrading microbes and proposed using 70% ethanol to prevent cross-contamination. Sterilization of the soil eliminates the problem, but it destroys the structure of the soil. Similar degradation of the cellulosic membrane has been reported by Delage et al. (1992), Delage and Cui (2008) and Ng et al. (2007). Cellulosic membranes are susceptible to bacteria attacks in a long duration test (Slatter et al. 2000, Monroy et al. 2007). Tripathy et al. (2011) mentioned that cellulosic membranes are also known to be vulnerable to mechanical strain according to the membrane manufacturer. In addition, Tripathy et al. (2011) found that some alteration of the pore size of the cellulosic membrane occurs during tests using the osmotic technique leading to leakage of some PEG molecules through the membrane. The problem is more serious at higher applied suctions.

Figure 2-11: Calibration curves for PEG 20000 solution from different researchers

Figure 2-12: Comparison of osmotic suction for PEG and NaCl solution using two different measurement methods (from Kiyosawa 2003).

Suraj De Silva (1987) proposed that the cellulosic membrane be changed after six days but he used the semi-permeable membrane up to 10 days in his tests. Slatter et al. (2000) and Monroy et al. 2007) recommended the use of polyether sulphonated synthetic membrane as opposed to cellulosic dialysis

membranes for longer testing times. The PES membrane was only tested for suctions below 1000 kPa although there is no data to suggest that the membrane did not perform well at suctions greater than 1000 kPa. However, Delage and Cui (2008b) mentioned that cellulosic membranes are still being used because of the low permeability of PES membrane.

With recent advances in desalination technology, more robust and durable semi-permeable membranes are now readily available. For example, reverse osmosis (RO) membrane can operate within a wide range of pH (2 to 11), temperatures up to 45°C, and can perform over several years (DOW water and process solutions 2020). Monroy et al. (2007) further concluded that the osmotic technique can be used to perform long-duration tests with the correct choice of the membrane and salt solution.

It is generally assumed in the osmotic technique, the applied osmotic suction is equivalent to the matric suction of the soil specimen at equilibrium, the stress path history does not affect the osmotic suction hence osmotic pressure remains constant throughout the test and the head losses due to the membrane resistance are negligible (e.g., Delage and Cui 2008a). Tarantino and Mongiovi (2000) and Marcial (2003) found difficulty in applying suction greater than 1000 kPa in the osmotic technique using PEG solutions and a cellulosic membrane. The actual matric suction measured across the semi-permeable membrane by Dineen (1997), Dineen and Burland (1995) and Peck and Rabbidge (1969) showed differences from the osmotic suction of the PEG solution measured using a psychrometer by Williams and Shaykewich (1969) as shown in Figure 2-11.

Similar measurements conducted by Yuan et al. (2017) using PES membrane instead of cellulosic membrane showed that there is a difference in the osmotic suction depending on the molecular weight of the PEG solution especially at high suctions. To date, except for Dineen (1997), there is very little work in the literature that shows independent matric suction measurements conducted on soil specimens after the application of the osmotic technique. So far, the osmotic technique has only been applied using PEG solutions and no literature exists on the use of inorganic solutes and RO membranes in the osmotic technique. The advantages accrued to inorganic solutes and RO membranes are therefore worth investigating in the osmotic technique to address the challenges associated with the use of PEG solution and cellulosic membranes.

In this thesis, investigations on the osmotic technique with the use of sodium chloride (NaCl) solutions and reverse osmosis (RO) membrane in a conventional oedometer to induce matric suction in a soil specimen under a normal load were investigated. In addition, independent matric suction measurements were also conducted on the specimens after the test.

Other suction control techniques include the hanging water column and centrifuge used for controlling matric suction. These methods are fully explained in ASTM D6836-16 and referred to as methods A and E, respectively. The hanging water column and centrifuge control matric suction within 0 – 0.12 MPa and 0 - 0.08 MPa, respectively. The approximate equilibration time ranges between several hours to days for both the hanging water column and centrifuge. Both techniques are affected by the narrow suction ranges of the suctions that can be controlled. A summary of the suction control techniques is shown in Table 2-3.

Table 2-3: Summary of Suction control techniques

Technique	Suction component	Suction range (MPa)	Approx. Equilibration time
Axis translation	Matric	0 - 1.5	days-weeks
Vapour equilibrium	Total	3 – 1000	weeks - months
Osmotic	Matric	0 – 10	days-weeks
Centrifuge	Matric	0 - 0.08	hours - days
Hanging water column	Matric	0 - 0.12	Hours - days

2.2.3: Soil-water characteristic curve

The variation of suction with the water content of the soil following either the drying or the wetting process is referred to as the soil-water characteristic curve (SWCC) (Fredlund and Rahardjo 1993, Barbour 1998). Croney (1952) studied the mechanism of water movement in soils and outlined methods for estimating the moisture distribution with depth beneath impervious pavements under equilibrium conditions. The water content can be expressed in terms of gravimetric water content w (ratio of mass of water to mass of solids), volumetric water content θ (ratio of volume of water to total volume of soil), or degree of saturation S (ratio of volume of water to volume of voids).

In geotechnical engineering, the gravimetric water content is more commonly used while in soil science, the volumetric water content is more commonly used (Fredlund and Xing 1994). Fredlund (2006) explained how using volumetric water content to define the SWCC is more illustrative considering the soil volume changes with variation in matric suction.

SWCCs are often used to estimate other unsaturated soil properties like coefficient of permeability, shear strength, and compressibility, once the saturated soil properties are established (Fredlund and Rahardjo 1993).

Different laboratory methods such as hanging column, pressure extractor, chilled mirror hygrometer, and the centrifuge can be used to determine the SWCC and have been standardized in ASTM D6836-16. Other techniques such as the filter paper technique, osmotic technique, and vapour equilibrium technique have been described in sections 2.2.2.2.

Figure 2-13 shows a typical SWCC presented in terms of volumetric water content for both drying (desorption) and wetting (adsorption) paths. The drying and wetting paths are obtained using different sequences: in the drying path, matric suction is incrementally applied to initially saturated soil whereas, in the wetting path, matric suction is reduced gradually from initially dry soil.

Figure 2-13: Typical SWCC for a silt soil (modified from Fredlund et al, 2012)

One of the most common features on the SWCC is the air-entry value (AEV), which is the matric suction at which air pressure starts to enter the largest soil pores during the drying path. The opposite of AEV is the water-entry value (WEV), the matric suction at which water starts to permeate the smallest pores on the wetting path. On the drying path, the water content will decrease until a residual matric suction is reached while on the wetting path the water content will increase until saturation is reached.

For consistency in the estimation of the AEV and the WEV, the tangent method proposed by Brooks and Corey (1964) is generally used. To determine the AEV, as well as the WEV, the inflection point

is taken as the reference point. On the drying path, two tangent lines are drawn from low suction states and the reference point that intersect at a point referred to as the AEV. For the wetting path, tangent lines are drawn from high matric suction states to the reference point and their intersection point is referred to as the WEV (See Figure 2-13).

Soils follow different paths during drying (desorption) and wetting (adsorption) as illustrated in Figure 2-13 with the wetting path showing lower water content than the drying path for the same matric suction. The behaviour is referred to as hysteresis. Hysteresis is reported to depend on several factors such as:

- (i) contact angle difference between the soil particles and water for both drying and wetting paths (Likos and Lu 2004).
- (ii) irregularities in pore-geometry referred to as the ink-bottle effect (Bear and Verruijt 1987), and
- (iii) entrapped air in the pores during the wetting path (Fredlund and Rahardjo, 1993).

The first two are regarded as the main factors that contribute to hysteresis. A decision taken by the geotechnical engineer has to be based on which path is being followed (drying or wetting) for the appropriate estimation of unsaturated soil property function (Tami et al. 2004).

2.2.3.1: Factors affecting SWCC.

Several factors have been described to affect the shape of SWCC (Fredlund and Xing, 1994, Fredlund et al., 2012) and the most commonly reported factors include the soil type, soil structure, dry density, stress history, mineralogy and method of specimens' preparation. Typical drying SWCC for sandy, silty, and clayey soils are illustrated in Figure 2-14, in which volumetric water content is plotted in arithmetic scale, and matric suction is presented in logarithmic scale. Different soil types have different shapes of SWCC with sand having a steeper SWCC slope than silty and clay soils. Some of the factors that influence the shape of SWCC are briefly discussed in the next section.

(a) Soil type

Properties such as gradation characteristics, plasticity, and texture are all considered under soil type. Childs (1940) reported that the soil structure and in-situ pore geometry can be described by the SWCC when measured on undisturbed specimens. Vanapalli (1994) reported that the slope of the SWCC in the transition zone is steepest in sand reducing toward Regina clay and attributed the observation to differences in pore sizes across the different soil textures. He further noted that the shape of the SWCC for sand revealed a uniform pore size distribution which became gentler as clay content increases. Li and Zhang (2009) reported that the shape of SWCC is related to the shape of the pore-size distribution

(PSD) and reported that unsaturated compacted specimens exhibited a distinct dual-porosity structure. They further showed that for specimens having bimodal PSD, there were possible changes in the bimodal PSD during drying and saturation processes.

A similar study conducted by Yang et al. (2004) and Gallage and Uchimura (2010) reported that for sandy soils, the SWCCs and the grain size distribution curves have similar shapes. They further reported that a steeper drying SWCC and a smaller hysteresis are observed by soil with a uniform grain size distribution as compared to a less uniform grain size distribution.

A study on the relationship between plasticity and water retention behavior conducted by Lu et al. (2007) reported that water retention capacity increases with the increase in the plasticity index of the soil. Therefore, the soil type affects and influence the shape of the SWCC.

(b) Initial density and structure

Compaction and initial compaction water content can greatly affect the soil dry density and structure of a specimen. The dry density will increase when the compactive effort is increased and reduces the water content.

Figure 2-14: Soil-water characteristic curve for sandy soil, silty soil, and clayey soil (modified from Fredlund and Xing, 1994).

It has been reported that soil suction is independent of the density of soil at specific water contents and the most dominant parameter that influences suction is the water content (Marinho and Stuermer 2000, Agus and Schanz 2005, Birle et al. 2008). Birle et al (2008) reported that the fabric of the soil is influenced by the initial compaction water content. Recent studies conducted by Satyanaga et al.

(2013) reported that the initial dry density and water content can affect the shape of both unimodal and bimodal SWCC irrespective of the grain-size distribution (GSD).

Vanapalli (1994) and Vanapalli et al. (1999) showed that AEV and degree of saturation increased in order from dry of optimum to the optimum, and wet of optimum for compacted soils. Vanapalli et al. (1999) also concluded that the slope of the SWCC gets steeper as the initial water content gets lower, hence, a higher desaturation rate. Swanson and Barbour (1991) observed that the SWCC for fine-grained soils was more affected by the dry density of soil as compared to the coarse-grained soils. It is further reported by Gallage and Uchimura (2010) that soils with low dry densities show low AEV and residual suctions as compared to soils with high dry densities.

(c) Stress history

It has been reported that at low suctions, soils subjected to different stress histories will have different SWCC. Whereas some studies report that the AEV is independent of the pre-consolidation pressure hence, the AEV is the same (Vanapalli et al. 1999), Delage and Lefebvre (1984) observed that AEV increases with pre-consolidation pressure. It is further observed that an increase in the over consolidation ratio (OCR) decreases the suction capacity. Other studies have shown that the AEV and SWCC slope decreases with an increase in suction (Kawai et al. 2000, Lee et al. 2005). It is characteristic of the SWCC that within low matric suction ranges, the macropores and capillary forces in the soil are more predominant and therefore, the confining stress influences the variation of the SWCC (Gens and Alonso 1992).

2.2.3.2: Equations for SWCC

Many empirical equations are available in the literature to fit the SWCC data. The earliest SWCC equation is by Gardner (1958) where he developed a model for the permeability of unsaturated soils but has since been extended to model SWCC and regarded as the first SWCC equation. Leong and Rahardjo (1997) recommended that most of the SWCC equations can be derived from a generic equation given in Equation 2.8.

$$a_1 \Theta^{b_1} + a_2 \exp(a_3 \Theta^{b_1}) = a_4 \Psi^{b_2} + a_5 \exp(a_6 \Psi^{b_2}) + a_7 \quad (2-8)$$

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

Brooks and Corey (1964) proposed a best fitting equation assuming a constant water content for suctions below AEV and the material desaturates exponentially after the AEV as in Equation 2-9.

$$\begin{aligned}\theta_w &= \theta_s & \text{for } \psi < a \\ \theta_w &= \theta_s \left(\frac{\psi}{a} \right)^{-\lambda} & \text{for } \psi > a\end{aligned}\quad (2-9)$$

where: a is a fitting parameter related to the AEV, λ is the pore size distribution index which is related to the pore size distribution. However, Sillers et al. (2001) note that Equation 2-9 does not provide a continuous function for the SWCC and the discontinuity in the function at the suction value equal to a may cause instability during numerical modelling of unsaturated soil behaviour.

Van Genuchten (1980) proposed a closed-form, smooth, three-parameter, Equation (2-10). This allows for greater flexibility than the models proposed by Gardner (1958) and Brooks and Corey (1964) and fits SWCC data for a wide range of soils.

$$\theta_w = \theta_r + \frac{(\theta_s - \theta_r)}{\left[1 + (\alpha\psi)^n \right]^m} \quad (2-10)$$

where θ_w = volumetric water content, θ_s = saturated volumetric water content, θ_r = residual volumetric water contents, ψ = matric suction, α , n and m are fitting parameters.

α = parameter related to the inverse of air-entry value, n = the parameter related to the rate of water extraction from the soil when the air-entry value is exceeded, and m = the parameter related to the residual water content, commonly treated as $m = 1 - \frac{1}{n}$ or $m = 1 - \frac{2}{n}$ to reduce the number of variables.

Fredlund and Xing (1994) proposed a continuous, closed-form three-parameter model similar to van Genuchten's model as in Equation 2-11:

$$\theta(\psi) = C(\psi) \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{\psi}{a} \right)^n \right] \right\}^m} \quad (2-11)$$

where a = fitting parameter primarily related to the air-entry value; n = a function of the rate of water extraction from the soil when the air-entry value is exceeded; m = a function of the residual water content; $C(\psi)$ = correction factor which ensures $\theta(\psi)$ gets to 0 when suction, $\psi = 1 \text{ GPa}$; re-written as Equation 2.12

$$C(\psi) = \frac{\ln\left(1 + \frac{\psi}{\psi_r}\right)}{\ln\left(1 + \frac{1,000,000}{\psi_r}\right)} \quad (2-12)$$

ψ_r = matric suction corresponding to residual water content θ_r .

Leong and Rahardjo (1997) reviewed the most popular unimodal SWCC fitting equations and observed that Fredlund and Xing (1994) equation has few parameters and has the best fitting ability. However, they recommended that the correction factor $C(\psi)$ can be taken as 1 for practical applications (Leong and Rahardjo 1997).

van Genuchten (1980) and Fredlund and Xing (1994) equations are the most broadly used. Throughout this study, Fredlund and Xing (1994) and the van Genuchten (1980) equations are referred to as FX and VG equations, respectively.

Direct measurement of SWCCs is expensive, time-consuming and labour intensive. As a result, physical properties such as soil gradation properties, densities, organic content and soil texture have been regularly used to estimate SWCCs. Several SWCC estimation methods using soil gradation parameters and index properties can be found in the literature (e.g. Gupta and Larson 1979, Vereecken et al. 1989, Chiu et al. 2012, Zou 2018). However, the most accurate methods at present are those requiring one-point measurement of the SWCC (e.g., Houston et al. 2006, Chin et al. 2010).

The concept of unimodal and bimodal SWCC is important because they require different fitting parameters. Satyanaga et al. (2013), Li et al. (2014) and Wijaya and Leong (2016) have proposed equations for bimodal SWCC and more experimental data is required to fit the equation. These equations are shown in Equation 2-13 to 2-15 respectively.

$$\theta_w = \left(1 - \frac{\ln\left(1 + \frac{\psi}{\psi_r}\right)}{\ln\left(1 + \frac{10^6}{\psi_r}\right)}\right) \left[\theta_r + (\theta_{s1} - \theta_{s2}) \left(1 - \operatorname{erfc} \frac{\ln\left(\frac{\psi_{a1} - \psi}{\psi_{a1} - \psi_{m1}}\right)}{s_1}\right) + (\theta_{s2} - \theta_r) \left(1 - \operatorname{erfc} \frac{\ln\left(\frac{\psi_{a2} - \psi}{\psi_{a2} - \psi_{m2}}\right)}{s_2}\right) \right] \quad (2-13)$$

where,

ψ_a = the AEV of soil, ψ_m = the matric suction at the inflection point of SWCC

S = the geometric standard deviation of SWCC.

$$s = (\theta_{s2}) \exp \sqrt{\frac{\sum_{i=1}^n \left(\ln \frac{\psi_i}{\mu} \right)^2}{n}},$$

μ = the geometric mean of matric suction, $\mu = \sqrt[n]{\psi_1 \psi_2 \dots \psi_n}$, Subscripts 1 and 2 indicate sub curves 1 (associated with macropores) and 2 (associated with micropores), respectively.

$$\text{erfc} = \text{the complementary error function, } \text{erfc} = \int_{-\infty}^x \frac{1}{\sqrt{2\pi}} \exp\left(-\frac{x^2}{2}\right) dx$$

Li et al. (2014) proposed the following Equation 2-14

$$\begin{aligned} \theta(\psi) = & \frac{(0.75\theta_s - 3\theta_r) \sqrt{(\psi_a \psi_t)^{2/\log(\frac{\psi_t}{\psi_a})}}}{\psi^{2/\log(\frac{\psi_t}{\psi_a})} + \sqrt{(\psi_a \psi_t)^{2/\log(\frac{\psi_t}{\psi_a})}}} + \frac{(0.25\theta_s - \theta_r)(4\psi_t)^{0.8}}{\psi^{0.8} + (4\psi_t)^{0.8}} \\ & + \frac{3\theta_r \sqrt{(\psi_{a2} \psi_t)^{2/\log(\frac{\psi_t}{\psi_{a2}})}}}{\psi^{2/\log(\frac{\psi_t}{\psi_{a2}})} + \sqrt{(\psi_{a2} \psi_t)^{2/\log(\frac{\psi_t}{\psi_{a2}})}}} + \frac{\theta_r (4\psi_r)^{0.8}}{\psi^{0.8} + (4\psi_r)^{0.8}} \end{aligned} \quad (2-14)$$

where ψ_t is the residual suction for water stored in macro-pores, and ψ_{a2} is the air entry value for water stored in micro-pores.

Wijaya and Leong (2016) proposed Equation 2-15 as shown below.

$$\begin{aligned} w &= w_{sat} + m_1 (x - x_1) + \sum_{i=2}^n R_i(x) (m_i - m_{i-1}) \\ \text{where} \\ R_i(x) &= \frac{1}{2} \left[(x - x_1) + \frac{1}{c_i} \ln \left\{ \frac{\cosh[c_i (x - x_i)]}{\cosh[c_i (x_i - x_1)]} \right\} \right] \end{aligned} \quad (2-15)$$

x_1 = the value of $\log s_i$ when $w = w_{sat}$, w is the gravimetric water content at full saturation, s in matric suction, $c_i = \frac{2}{\log\left(\frac{s_{i+}}{s_{i-}}\right)}$ where s_{i+} is the matric suction at the point of convergence and s_{i-} is the matric

suction at the point of departure.

Generally, there are more parameters associated with a bimodal SWCC equation as compared to unimodal SWCC equations. Different approaches have been applied to reduce the number of

parameters so that the equations become easier to optimise. Equation (2-15) has an advantage over Equations (2-13) and (2-14) in that it can be used for both unimodal and bimodal SWCC, the parameters are simple to obtain and can be obtained graphically.

2.3: Compaction

Compaction is one of the ground improvement methods which densifies the soils by the application of mechanical energy. Soil compaction is the simplest way of improving the mechanical properties of soil such as strength, compressibility, and permeability. In compaction, there is a rearrangement of soil particles without the outflow of water. This means soil compaction is obtained by removing air from its pore space. There exists a certain moisture content that will produce a maximum dry density for a specific compaction effort (optimum water content). Both the maximum dry density and optimum water content are of great importance in engineering practice. The theory of compaction for coarse-grained soils is relatively simple and well-understood, but it is much more complicated for fine-grained soils due to the colloidal behaviour of clay. Soil compaction in the laboratory can be achieved by either static or dynamic loading. For dynamic compaction, the procedure is standardized whereas the procedure for static compaction has not yet been standardized and different procedures are reported in the literature. The two methods are briefly explained in the subsequent sections.

2.3.1: Dynamic compaction

The procedure for dynamic compaction has been standardized by ASTM as standard Proctor test (ASTM D698-12) and modified Proctor test (ASTM D1557-12) and British Standard (BS 1377- 4). A comparison between standard and modified Proctor tests is shown in Table 2-4. Both standard and modified Proctor tests use the same mould, the number of rammer-blows per layer and the same mode of energy transfer (from rammer to the soil directly). Though the procedure has been standardized, studies (e.g. Sridharan and Sivapullaiah, 2005) have been done to reduce the amount of effort involved, time, sample disturbance and reduction in the amount of soil used for the test.

Table 2-4: Comparison between standard and modified Proctor compaction tests.

	Standard Proctor test	Modified Proctor test
Application	Generally used for fills that require minimal compaction such as small parking lots, building structures	Generally used for fills expected to support large loads such as Roadways, airport runways and concrete parking aprons.
Test condition	Weight of hammer = 24.5 N	Weight of hammer = 44.5 N
	Drop distance = 305 mm	Drop distance = 457 mm
	Number of layers = 3	Number of layers = 5
	Specific energy = 600 kN-m/m ³	Specific energy = 2700 kN-m/m ³
	Number of blows per layer = 25	Number of blows per layer = 25

2.3.2: Static compaction

The procedure of static compaction has not yet been standardized and different procedures are reported in the literature. In static compaction, the loose soil is enclosed in a container and compaction is attained by compression via a piston. A set-up for static compaction is shown in Figure 2-15. Static compaction can be conducted by controlling either displacement or stress (Venkatarama Reddy and Jagadish, 1993). The compaction process can be completed by compressing the soil at a constant rate of stress (e.g., Tarantino and De Col, 2008), a constant rate of displacement (e.g., Sivakumar and Wheeler, 2000) or without controlling the rate purposely (e.g., Cuisinier and Laloui, 2004) until the target maximum stress is reached.

In displacement-controlled static compaction, the dry density or height of the specimen is known. The dry density for displacement-controlled static compaction is commonly determined from the standard Proctor curve (e.g., Delage et al. 1996, Vanapalli et al. 1996). A constant rate of displacement is applied to the soil until the target specimen height is reached. At the end of compaction, the maximum static stress applied on the soil sample will be variable and depends on the moulding water content and the target dry density.

In stress-controlled static compaction, the maximum normal stress applied to the specimen is known. The stress-controlled static compaction is more widely used in the literature (Gens 1995; Cuisinier and Laloui 2004; Tarantino and De Col 2008; Monroy et al. 2010) than the displacement-controlled static compaction. A major advantage of stress-controlled static compaction is that the stress history

of the soil sample is well-defined, which is important if the soil is subsequently subjected to mechanical testing (Tarantino 2011).

These are all aimed at easing the method of obtaining compaction curve parameters necessary for geotechnical engineering applications.

Figure 2-15: Static compaction set-up (from Venkatarama Reddy and Jagadish, 1993)

2.3.3: Engineering behaviour of compacted soils

The water content of compacted soil is referenced to the optimum moisture content on the Proctor compaction curve called the optimum water content. Thus, soils are referred to as compacted dry of optimum (on the dry side), or wet of optimum (on the wet side) when the moisture content is less or more than the optimum water content, respectively. The structure of a compacted soil may not be similar on both sides even at comparable dry densities and this difference has a strong impact on the engineering characteristics of soils. A brief description of some behaviour is briefly described.

2.3.3.1: Structure

According to Gupta et al. (1989), soil structure can be referred to as the arrangement of primary soil particles into secondary particles. At a macroscopic level, it comprises a measure of the relative percentage of soil solids to its pores always defined as void ratio, porosity, or bulk density. At a microscopic level, it includes the interactions of solids with other solids and is inferred from soil behavioural properties such as strength, or water and airflow behaviours (Gupta et al. 1989).

In the 1950s, extensive research was carried out by Lambe (1958a, 1958b) and Seed and Chan (1959a, 1959b) to understand the structure and engineering behaviour of compacted clay. For a given compactive effort, soil particles are randomly oriented and said to have a flocculated structure on the dry side whereas, on the wet side, particles are more oriented in a parallel arrangement and said to have a dispersed structure (Cetin et al. 2007).

Lambe (1958a) conceptually explained the structure of clay compacted dry and wet of optimum, as shown in Figure 2-16. This is attributed to the adsorbed water layer (water film) surrounding each particle on the wet side (Lambe 1958a). On the dry side of optimum, the compacted clay has a strong deficiency of water and the development of the diffuse double layer is prevented by the high electrolyte concentration because of a lesser amount of water.

The diffuse double layer depression leads to low inter-platelet repulsion and a tendency towards flocculation of the colloids (Lambe 1958a): the edge of a platelet that has a positive charge gets attracted by the surface of an adjacent platelet which has a negative charge, as shown in Figure 2-17(a). On the wet side of optimum, the electrolyte concentration is reduced due to the increasing amount of water. The diffuse double layer expands and the repulsion between platelets increases. As a result, the dynamically compacted clay tends to form a dispersed structure, as shown in Figure 2-17(b).

Figure 2-16: Effects of compaction on structure (from Lambe 1958a)

Figure 2-17: Flocculated and dispersed structure (modified from Lambe 1958a)

The experiments conducted by Seed and Chan (1959a, 1959b) clearly show that compaction methods (static and dynamic) would not affect the soil structure and engineering behaviour if the soil is compacted dry of optimum, but a considerable difference may exist if the soil is compacted wet of optimum. Diamond (1970) presented the difference in structure of soils prepared on dry and wet of optimum. He reported the existence of aggregates on the dry side of the optimum stack to each other of dimensions about 5 mm whereas, on the wet side, no large pores were observed indicating a more compact structure. Similar observations have been made by (Delage et al. 1996, Romero and Simms 2008, Li and Zhang 2009, Monroy et al. 2010).

2.3.3.2: Strength

For both drained and undrained conditions, soils compacted on the dry side of optimum exhibit higher strength but after saturation, the undrained strength on the dry side is higher if swelling is prevented. However, if swelling is allowed, the strength of the compacted soil prepared on the wet side can be higher. For the drained test, the strength of the samples compacted on the dry side of optimum is slightly higher or about the same as that compacted on the wet side.

The stress-strain modulus is higher on the dry side of optimum as compared to the wet side of optimum. Typical stress-strain relationships for clay compacted at dry and wet of optimum by static and dynamic compaction are schematically shown in Figure 2-18. The soils compacted at dry of optimum are quite brittle and show a similar stress-strain curve for static and dynamic compaction. At the wet of optimum, the stress-strain curve obtained by static compaction is steeper than that for dynamic compaction. The reason is the clay structure obtained by static compaction may be flocculated, while the clay structure obtained by dynamic compaction is dispersed due to the high shear strain induced by the compaction method. It was observed that at large shear strain (e.g., 20%), statically and dynamically compacted soil samples would give similar shear strength.

Figure 2-18: Stress-strain relationship for compacted clay prepared dry and wet of optimum by static and dynamic compaction (after Seed and Chan 1959a)

Lambe (1958a, b) and Seed and Chan (1959) reported that the shear strain caused by static compaction is negligible regardless of the moulding water content and concluded that statically compacted clay is also likely to have a flocculated structure on the wet side of optimum. The microstructure for a soil prepared by both static and dynamic compaction is rarely reported in the literature. Little research has since been conducted to investigate the influence of compaction methods on soil structure and engineering behaviour. As a result, it is commonly believed or assumed that static and dynamic compaction in the laboratory would lead to the same soil structure regardless of the water content (e.g. Crispin et al. 2011, Tarantino 2011). Hence displacement-controlled static compaction is widely used as a replacement for dynamic compaction in preparing the soil sample for laboratory testing.

Ahamed et al. (1974) compared the pore size distributions (PSD) of an illite clay, Grundite, compacted at the dry and wet of optimum and optimum water content using dynamic, kneading, and static compaction. The results show that all the compaction methods give similar PSD regardless of the water content. Although PSD is an indication rather than an equivalent to microstructure, the test results obtained in Ahmed et al. (1974) seem to justify that static and dynamic compaction would produce similar structures at the wet of optimum.

Sivakumar and Wheeler (2000) observed that the kaolin samples which were statically and dynamically compacted to similar dry density and water content at the dry of optimum have similar stress-strain relationships.

2.3.3.3: Swelling and compressibility

Soils compacted on the dry side of optimum have a high-water affinity. when they get in contact with water, the soil will absorb much more water and more swelling is recorded. Compressibility of the

soil compacted past the optimum water content (wet-side of optimum) is high and low on the dry side of optimum (Lambe 1958a). The rate of consolidation on the dry side of optimum is more rapid while the rebound per compression is greater on the wet side of optimum (Gupta et al. 1989). A study on several clay soils by Mishra et al. (2008) concluded that the swell behaviour is controlled by the clay mineralogy irrespective of the compaction condition.

2.3.3.4: Shrinkage

Soils compacted on the wet side of optimum tend to show more shrinkage during drying, than soils compacted in the dry side of optimum (Mishra et al. 2008, Habib 2013). This can be explained based on the more orderly orientation of particles on the wet side that allows them to pack more efficiently.

2.3.3.5: Permeability

Soil compacted on the dry side exhibits the same saturated permeability in all directions, therefore, less permeable, whereas on the wet side, the soil is more permeable along with particle orientation than across particle orientation (Wright et al. 1996)

2.4: Strength of unsaturated soils

2.4.1: Shear strength

The shear strength of unsaturated soils is one of the key properties of unsaturated soils. Most engineering projects involve unsaturated soils either in an in-situ or compacted condition. The shear strength of unsaturated soils is greatly affected by changes in the matric suction. Modifications are made to conventional apparatuses for saturated soils to obtain shear strength parameters for unsaturated soils (e.g., Escario and Saez 1986, Melinda et al. 2004, Schnellmann et al. 2013, Gu et al. 2019).

The modification done to conventional triaxial, or direct shear apparatuses are either for suction control or suction measurement during shearing. However, triaxial test compared to direct shear test presents additional problems such as difficulty in preparing uniform specimens, long drainage path and requires longer testing time to perform (Ho and Fredlund 1982). Regardless of the experimental technique used, obtaining unsaturated shear strength parameters requires more experiments, longer testing time, and the test set-ups are generally more complicated and more expensive as compared to testing saturated soils. As a result, researchers such as Nam et al. (2011) proposed the use of multistage direct shear tests for both saturated and unsaturated soils while Tarantino and El Mountassir (2013) proposed the use of constant water content tests where suction control or suction monitoring is not used to obtain a conservative estimate of the shear strength of unsaturated soil. Several techniques have been used to control suction during the determination of shear strength of unsaturated soils such as osmotic and vapour equilibration techniques (e.g., Blatz and Graham 2000, Blatz et al. 2008, Delage and Cui 2008, Hamidi et al. 2013, Bulolo and Leong 2020) and the axis-translation technique (e.g., Fredlund and Morgenstern 1977, Ng et al. 2007, Tripathy et al. 2012). In the axis-translation technique, equilibration time is always dependent on the high air-entry disk used where a disk with a higher air-entry value will require a longer equilibration time.

In constant suction tests (pore-air pressure and pore-water pressures are maintained during the test), it is assumed that matric suction is constant during shearing using a slow shearing rate. In a constant suction test, the matric suction of the soil specimen is always trying to equilibrate with the applied matric suction (Zhang 2016). The selected shearing rate should allow equilibration or dissipation of the induced pore-water pressures throughout the specimen. Different shearing rates ranging between 0.1 to 0.6 mm/h have been used for constant-suction direct shear tests on compacted unsaturated soils (Escario and Saez 1986, Gan 1986, Oloo and Fredlund 1996, Caruso and Tarantino 2004, Jotisankasa and Mairaing 2010). Gan (1986) observed that shear strength remained constant below a specific shearing rate from a plot of shear strength versus shearing rate. Satija (1978) observed that the water

content was more sensitive to the change in shearing rate than the deviator stress during the consolidated drained triaxial tests. Therefore, the variation of both strength and water content can be used as a criterion to select an appropriate shearing rate. Han (1997) proposed the use of pore-water pressure equalization during the mobilization of peak shear stress as a suitable method of selecting a shearing rate. Shearing rate has also been selected after running trial tests and choosing the rate that gives the maximum shear strength (Han 1997, Nishimura et al. 1999, Melinda et al. 2004, He et al. 2017). The maximum shear stress is affected by the net normal stress applied. Therefore, a range of net normal stresses (low to high) should be used in the selection of the shearing rate. A slower shearing rate than required can be used but the test will require a longer testing time. Additionally, the permeability of both the soil specimen and ceramic disk also play a role in the choice of the shearing rate because the rate should be slow enough to equalize the water pressures between the soil specimen and the ceramic disk.

Contradictory observations on the effect of matric suction on friction angle in constant suction direct shear tests were reported. Cokca et al. (2004), Merchan et al. (2008), and Hossain and Yin (2010) found that friction angle increases with increasing matric suction and Ye et al. (2010) and Schnellmann et al. (2013) found friction angle unaffected by matric suction. These observations on change in friction angle in constant suction direct shear tests may indicate either that void ratio changes with net normal stress and hence the friction angle or matric suction is not constant during the constant suction direct shear test.

Direct shear test results for unsaturated soils where the shear stress does not plateau but continues to increase with shear displacement (strain hardening behavior) have also been reported (e.g., Shimada 1998, Hossain and Yin 2010a, Zhou et al. 2016, Gu et al. 2019). In such cases, different criteria have been used to define the “failure stress” such as at an arbitrary shear strain (Zhou et al. 2016, He et al. 2017) at a particular shear displacement based on standards (Gu et al. 2019) or the shear stress at a maximum shear displacement (Ooi and Carter 1987).

2.4.1.1: Constant suction direct shear tests

The constant suction direct shear test is carried out similarly to a conventional direct shear test. The test is carried out in two stages. In the first stage, the unsaturated soil specimen is consolidated to the target net normal stress and matric suction. Matric suction is usually applied using Hilf (1956) axis translation principle entailing the application of both air pressure and water pressure. The unsaturated soil specimen is then sheared in the second stage.

When consolidating an unsaturated soil specimen to the net normal stress and matric suction, there are no clear guidelines and usually, both net normal stress and matric suction are applied simultaneously during the consolidation stage. It is better to increase the net normal stress at a constant rate to the target value and applying matric suction at the same time (Zhang et al. 2016). Intuitively, if net normal stress is applied together with matric suction, the voids will compress almost instantaneously resulting in a long time to reach matric suction equilibrium. By applying a continuous loading or small step loading to the target value while applying matric suction will certainly reduce the “consolidation” time to achieve the target net normal stress and matric suction.

During a direct shear test, the normal load is kept constant while the shearing area of the specimen reduces. As a result, there is a gradual increase in normal and shear stresses. Because of the existence of three phases in unsaturated soils, incorporation of area correction becomes difficult in the interpretation of shear strength in the direct shear tests. Different area correction equations have been proposed for direct shear tests with a circular shear box (Lai 2004, Dong et al. 2013, Skuodis and Tamosiūnas 2014) as summarised in Table 2-5.

However, direct shear stress results are usually reported with either (i) no area correction (ASTM D3080/D3080M-11, BS EN ISO 17892-10), (ii) area correction applied to both the shear and the normal stresses (Shwan and Smith 2015, Gallage and Uchimura 2016), or (iii) area correction applied to only the normal stress (Murthy 2002). If condition (i) is used, the stress path is a vertical straight line (OA) while conditions (ii) and (iii) give a stress path that curves to the right as illustrated by OA' in Figure 2-19. Skuodis and Tamosiūnas (2014) concluded that the use of area correction in shear and normal stresses increase the shear strength evaluation accuracy after observing a 10% difference in stresses between the use of uncorrected and corrected areas. The area correction is almost negligible for small horizontal displacements less than 2 mm for both square and circular shear boxes and therefore, the shape of the shear box does not matter at such small horizontal displacement.

For direct shear test results that show strain-hardening or plastic behaviour, plotting the stress path using corrected normal and shear stresses will show that the stress path reaches the failure line at A' and continues on the failure line to B whereas direct shear test result that shows strain-softening behaviour, the stress path reaches the failure line at A' and then moves below the failure line to C as shown in Figure 2-19.

Table 2-5: Summary of area correction equations for circular direct shear samples

Reference	Corrected area, A_c	Correction factor
Lai (2004)	$A_c = A_o F$ $A_o = \frac{\pi}{4} D^2$	$F = \frac{2}{\pi} \left\{ \cos^{-1} \left(\frac{\delta}{D} \right) - \left(\frac{\delta}{D} \right) \sqrt{1 - \left(\frac{\delta}{D} \right)^2} \right\}$ <p>where; δ = horizontal displacement D = sample diameter</p>
Dong et al. (2013)	$A_c = 2S$	$S = \frac{\pi R^2}{90} \tan^{-1} \left[\frac{2\sqrt{R^2 - \left(\frac{\delta}{2} \right)^2}}{\delta} \right] - \delta \sqrt{R^2 - \left(\frac{\delta}{2} \right)^2}$ <p>where, R = radius of ring dynamometer, mm</p>
Skuodis and Tamosiūnas (2014)	$A_c = A$	$A = 2 \cdot \left[\frac{D^2 \pi}{360} \cdot \cos^{-1} \left(\frac{0.5\delta}{0.5D} \right) \right] - \left[\left\{ 0.5\delta \sin \left[\cos^{-1} \left(\frac{0.5\delta}{0.5D} \right) \right] D \right\} \right]$

Direct shear test results from Gallage and Uchimura (2016) and He et al. (2017) are used to illustrate the above cases. Figures 2-20 and 2-21 show the direct shear test results where uncorrected and corrected shear stresses are plotted against horizontal displacement. The uncorrected shear stress attains a maximum value and remains constant whereas the corrected shear stress shows a strain hardening behaviour.

Figures 2-22 and 2-23 illustrate the stress paths for the uncorrected and corrected stresses showing that the stress path plotted with corrected stresses continues along the failure line. The c' and ϕ' obtained from the stress paths using corrected stresses in Figures 2-22 and 2-23 are 0 kPa and 38.8° , and 3 kPa and 18° , respectively, as compared to the reported values of 0 kPa and 38.8° in Gallage and Uchimura (2016) and, 5 kPa and 18° in He et al. (2017), respectively.

Figure 2-19: Stress path from a saturated direct shear test.

Figure 2-20: Shear stress versus horizontal displacement for constant suction direct shear tests from Gallage and Uchimura (2016).

Figure 2-21: Shear stress versus horizontal displacement for constant suction direct shear tests from He et al. (2017)

Figure 2-22: Stress paths for constant suction direct shear tests from Gallage and Uchimura (2016) with no peak shear stress

Figure 2-23: Stress paths for constant suction direct shear tests from He et al. (2017) with no peak shear stress

Figure 2-24(a) shows an example of direct shear test results from Han (1997) showing strain-softening behavior for both uncorrected and corrected shear stresses. Figure 2-24(b) shows the stress paths for both uncorrected and corrected stresses.

Figure 2-24: Shear stress versus horizontal displacement and stress paths for constant suction direct shear tests from Han (1997) with peak shear stress.

The c' and ϕ' obtained from the stress paths using corrected stresses in Figure 2-24(b) are 16 kPa and 29.1° , respectively, as compared with the reported values of 13 kPa and 29.4° , respectively. Hence, the use of corrected stresses in the stress path does not affect the determination of c' and ϕ' for a soil showing strain-softening behavior in the direct shear test.

2.4.1.2: Constant water content tests

In constant water content (CW) tests, pore-air pressure u_a is drained while pore-water pressure u_w is undrained (Fredlund et al. 2012). Therefore, the use of CW tests can simulate the undrained condition of unsaturated soils. Rahardjo et al. (2004) concluded that for suctions less than the air-entry value, the consolidated drained test shear strength parameters can be reproduced by constant water content test using triaxial apparatus.

Studies have been conducted where high capacity tensiometers (HCT) were used to measure suction during constant water content test (Caruso and Tarantino 2004, Tarantino and Tombolato 2005, Jotisankasa and Mairaing 2010, Marinho et al. 2016). However, fine-grained soils have suction range spanning several thousands of kPa making it difficult to use HCT which is limited to a suction of about 1500 kPa. Oloo and Fredlund (1996), Vanapalli and Lane (2002), Cokca et al. (2004) used a conventional direct shear apparatus to test compacted soils and assumed that any change in suction is negligible. The main challenge to the constant water direct shear (CWDS) test is the lack of suction measurement during the shearing process.

While the use of constant water content incurs the need to measure matric suction during the test, research has shown that there are self-equilibration and suction is most likely to remain constant if the displacement rate adopted is small though the water content on average remains the same as the initial water content (Heitor et al. 2013). Furthermore, the pore-air and pore-water pressures can be approximated as discussed in the following sections.

(a) Pore-air pressure evaluation

The vertical stress applied by a dead load that acts on the horizontal shear plane is kept constant during shearing. This implies that the volume change (compression or dilation) is represented by the vertical displacement (Figure 2-25). In other words, the volume change is a result of either compression or dilation of the voids and dissolution of air assuming that water and soils solids are incompressible, and the influence of vapour pressure and temperature fluctuations are negligible.

Figure 2-25: Initial and final volume condition for idealised CWDS shear band for compression and dilation cases.

Henry's law is used to calculate the amount of air dissolution in the water phase. Therefore, the total volume of air = free air + dissolved air.

Using Hilf (1948) analysis,

$$V_{Ti} = V_v - V_w + hV_w \quad \text{and} \quad V_{Tf} = V_v - V_w + hV_w - \Delta V \quad (2-16)$$

where V_{Ti} and V_{Tf} are the initial and final total volumes, V_v is the volume of voids, V_w is the volume of water in voids, h , is the solubility constant, ΔV is the change in volume from initial to final.

The solubility constant depends on the pressure in the environment and considering a constant temperature of 25°C as by Henry's law. The volume change ΔV may be positive or negative for compression and dilation respectively.

For undrained conditions, Boyle's law of pressure and volume equation can be written as

$$u_{Ti} V_{Ti} = u_{Tf} V_{Tf} \quad (2-17)$$

where u_{Ti} and u_{Tf} are initial and absolute pressures respectively $u_{Ti} = 100 \text{ kPa} = P_a$ and

$$u_{Tf} = u_{Ti} + \Delta u_a$$

Substituting equations 2-16 into equation 2-17 and dividing with the initial volume gives

$$\Delta u_a = \frac{u_{Ti} \frac{\Delta v}{V_T}}{n_0 + S_0 n_0 (h-1) - \frac{\Delta v}{V_T}} \quad (2-18)$$

where S_0 and n_0 is the initial degree of saturation and porosity respectively

Hilf (1948) suggested that changes in the air pressure could be related to the changes in the water pressure, therefore, $\Delta u_a = \Delta u_w$. Equation 2-18 is similar to that proposed by Hilf (1948) but with a difference in the adopted geometry and the volume interpretation considering both compression and dilation. It should also be noted that air solubility is considered as a function of air pressure.

(b) Evaluation of matric suction during shear

The matric suction during shearing can be estimated by considering the volumetric strain of the direct shear sample. As mentioned in section 2.5, one of the drawbacks of direct shear is defining the full strain state because the deformation is not uniform throughout the sample. However, Discrete Element Modelling (DEM) simulations by (Thornton and Zhang 2001, Cui and O'Sullivan 2006, Liu 2006) reported that the shear deformation is relatively uniform in the central region of the sample showing almost a simple shear. This central region is referred to as a shear band and Liu (2006) reported that the shear band is about 10 times the average grain size of the DEM sample. Jewell and Wroth (1987) reported that the shear zone in the direct shear test can be localised and thin compared to the mass of the specimen. Therefore, the shear strain (γ_{yx}) and normal strains (ϵ_y) on the horizontal plane can be estimated using an idealised simple shear zone from Figure 2-26 as in Equation 2-19.

$$\gamma_{yx} = \frac{dx}{L} \quad \text{and} \quad \epsilon_y = \frac{dy}{L} \quad (2-19)$$

Liu (2006) compared the strains computed using the above relationships and results from the simple shear test and reported that the results were in good agreement. He later concluded that within the shear zone, elements of the sample in the direct shear box undergoes a simple shear deformation.

Figure 2-26: Direct shear illustration of direct shear test and shear zone of specimens under volume changes (compression and dilation) (modified from Jewell and Wroth, 1987).

It is assumed that during shear in the idealised simple shear deformation mode, the sample length remains constant and as a result, the horizontal strain increment, $d\epsilon_x = 0$. Therefore, from Mohr's circle of incremental strain for direct shear tests with an idealised shear zone simple shear deformation mode (Figure 2-27), the principal strain increments in the direct shear box can be got (Bolton 1986, Shibuya et al. 1997). Therefore,

$$d\epsilon_1 = d\epsilon_y + \left(\frac{\sin \delta + 1}{\cos \delta} \right) \frac{d\gamma_{yx}}{2} \quad (2-20a)$$

$$d\epsilon_3 = d\epsilon_y + \left(\frac{\sin \delta - 1}{\cos \delta} \right) \frac{d\gamma_{yx}}{2} \quad (2-20b)$$

Figure 2-27: Mohr's circle of incremental strain for direct shear tests and dilatancy, δ , in simple shear deformation mode (modified from Shibuya et al. 1997).

If the shear deformation is approximated to the simple shear, then the incremental shear strain $d\gamma_{yx}$ and normal strain $d\epsilon_y$ on the horizontal plane are estimated from Equation 2-19. Supposing all dilation occurs within the shear band of thickness L , the dilation angle (δ) can be expressed using the rate of change of vertical displacement (Δy) and horizontal displacement (Δx) to have Equation 2-21.

$$\delta = \tan^{-1} \left(\frac{d\epsilon_y}{d\gamma_{yx}} \right) \approx \tan^{-1} \left(\frac{dy}{dx} \right) \quad (2-21)$$

The volumetric strain is expressed as Equation 2-22.

$$d\epsilon_v = d\epsilon_1 + d\epsilon_2 \approx 2d\epsilon_y + d\gamma_{yx} \tan(\delta) \quad (2-22)$$

Therefore, rewriting equation 2-22 in terms of vertical and horizontal shear displacement measured during direct shear can be as in Equation 2-23.

$$d\epsilon_v = 2 \frac{\Delta y}{L} + \frac{\Delta x}{L} \left(-\frac{\Delta y}{\Delta x} \right) \quad (2-23)$$

It is assumed that the ratio of change in initial volume is equivalent to the change in volumetric strain in the shear band since the shear deformation occurs in the central region of the sample. Therefore,

$$d\varepsilon_v = \frac{\Delta v}{V_T} \quad (2-24)$$

By substituting Equation 2-24 into Equation 2-18, the change in pore-air pressure or pore-water pressure is given by Equation 2-25.

$$\Delta u_a = \Delta u_w = \frac{u_{Ti} d\varepsilon_v}{n_0 + S_0 n_0 (h-1) - d\varepsilon_v} \quad (2-25)$$

The pore-air or pore-water pressure is calculated reflecting the initial values of porosity and degree of saturation as a starting reference since both the porosity and degree of saturation vary during shearing. Hilf (1948) noted that the change in suction is zero in a completely undrained system since the air phase transmits the pressure to the water phase. However, the air pressure in the direct shear test is likely to dissipate and drop to zero hence the suction change can be approximated to the change in pore-water pressure.

This analysis has been adopted by Oloo and Fredlund (1996) for estimating suction change during compression and Ribeiro Heitor (2013) for estimating the change in suction during shearing. Oloo and Fredlund (1996) based on Hilf (1948) analysis and proposed Equation 2-26.

$$\Delta u_w = \Delta u_a = \frac{u_{Ti} \Delta n}{(1-S_0)n_0 + hS_0 n_0 - \Delta n} \quad (2-26)$$

Oloo and Fredlund (1996) assumed that the change in air pressure drops to zero hence the variation in pore-water pressure = change in suction and the net normal stress = total normal stress.

It should be noted that in contraction, there is a reduction in suction whereas a dilation results in an increase in the suction. The water pressure can be expressed by considering the incremental volume change since it is related to the suction changes during shearing. Therefore, the final suction can be estimated from Equation 2-27.

$$ds \approx \Delta u_w \text{ hence } S_f = \Delta u_w + S_i \quad (2-27)$$

where S_f = final suction and S_i = initial suction measured at the compression stage.

Therefore, the final suction after shearing can be estimated using Equation 2-25 to 2-27. Lourenco et al. (2012) reported the importance of contact angles during drying or wetting cycles. However, this approach does not consider the difference in contact angles during wetting or drying during either compression or dilation. Nonetheless, Equation 2-27 gives an approximation and not the actual change in suction based on the specimen's volume change. Until now, there is little knowledge on

the changes that occur between the contact angles on the water menisci and the macrostructure of soil during shearing.

Equation 2-27 was validated by Ribeiro Heitor (2013) using clayey silt from Caruso and Tarantino (2004) and silt from Jotisankasa and Mairaing (2010). It was reported that the predictions agreed with the experimental results within ranges of small horizontal displacement with an overestimation in suction at large displacements for the clayey silt. Nonetheless, the prediction was capable of estimating the variations in suctions of specimens compacted at different water contents of the silt soil.

Among the limitations of the approach is that it has been modelled for sand or sandy materials in the direct shear box (Jewell and Wroth 1987, Shibuya et al. 1997, Liu 2006). The existence of a distinguished limited shear zone of deformation of a kaolin specimen subjected to direct shearing has been reported by researchers such as Tchalenko (1970) however, the governing mode of deformation is not solely that of simple shear. Whereas this approach can be applied to sand and sand-silt mixtures, caution should be taken while extending the approach to clayey material because it applies to soils in the transition zone of desaturation where both air and water phases are continuous. Marinho et al. (2003) also pointed out that the use of Hilf's axis-translation method may result in an overprediction of pore-water pressure whereas the applied correction to account for suction may under-predict or over-predict the pore water pressure at low and higher total stresses, respectively.

The use of conventional direct shear tests under constant water content tests for unsaturated soils is still scarce. Furthermore, the interpretation of direct shear tests on unsaturated soils under constant water content tests is still formative and further study is required.

Therefore, in this study, the approach for estimating the final suction after shearing has been used to predict the final suctions of soils specimens under constant water suction direct shearing. The procedure followed is similar to that proposed by Ribeiro Heitor (2013).

A summary of the procedures to estimate suction change when using a conventional direct shear apparatus is as follows:

- 1) Ensure there is minimal evaporation from the direct shear set up using grease between the halves of the direct shear box and covering the entire setup during the test.
- 2) Measure the as-compacted suction (using either HCT or filter paper or use the as compacted soil suctions if already measured).
- 3) Compute s_i , (the suction at the end of the compression stage) using the approach proposed by Oloo and Fredlund (1996) from the time-settlement curve.

- 4) Use a fast-shearing rate but ensuring that the chosen rate does not allow positive pore water pressure development during shearing for specimens with a high initial degree of saturation.
- 5) Record the horizontal and vertical displacements during shearing.
- 6) Compute the volumetric strains using Equation 8.
- 7) Compute the cumulative change in suction for the respective horizontal displacements using Equation 11.
- 8) Finally, estimate the suction during shearing using Equation 12

2.4.1.3: Shear strength equations

(a) Equations under single stress state variable approach

The most widely used approach in the single stress state variable is the effective stress approach introduced by Bishop (1959).

Mohr-Coulomb's shear strength in terms of Terzaghi's effective stress gives Equation 2-26 to estimate the shear strength of saturated soils. Therefore, Equation 2-2 proposed by Bishop (1959) is substituted in Equation 2-28 to yield 2-29, the shear strength equation proposed by Bishop et al. (1960).for unsaturated soils.

$$\tau_f = c' + (\sigma_n - u_w)_f \tan \phi' \quad (2-28)$$

$$\tau_f = c' + [(\sigma_n - u_a) + \chi(u_a - u_w)] \tan \phi' \quad (2-29)$$

where τ_f = shear strength on the failure plane, c' = the effective cohesion and ϕ' = the friction angle.

Challenges associated with the single stress variable has been discussed in section 2.2.1 and several researchers have come up with different ways of estimating the χ parameter.

Lamborn (1986) equation

Lamborn (1986) proposed Equation 2-30 for the shear strength of unsaturated soils. Lamborn (1986) used the irreversible thermodynamics principle to the energy volume relationship in solids, liquids and voids. The equation assumes that under unsaturated conditions, changes in the volumetric water content is related to the shear strength.

Though no experimental evidence was provided to support the assumption, it can be true if the changes in the volumetric water content on the shear strength contribution is directly proportional to the desaturation effect in unsaturated soils.

$$\tau = c' + (\sigma - u_a) \tan \phi' + \theta_w (u_a - u_w) \tan \phi' \quad (2-30)$$

θ_w = volumetric water content

Oberg and Sallfors (1997) equation

Oberg and Sällfors (1997) proposed Equation 2-31 to describe shear strength by use of an instantaneous degree of saturation, S .

$$\tau = c' + (\sigma - u_a) \tan \phi' + S(u_a - u_w) \tan \phi' \quad (2-31)$$

Oberg and Sällfors (1997) assumed that χ factor reflects the fraction of the pore area occupied by water, $\frac{A_w}{A_{tot}}$ and the contribution of soil suction is equivalent to the product of the degree of saturation and $\tan \phi'$.

They postulate that even with no unique relationship between $\chi \left(= \frac{A_w}{A_{tot}} \right)$ and S , the assumption that

$\frac{A_w}{A_{tot}} = S$ is satisfactory for engineering purposes. However, Vanapalli and Fredlund (2000) tested the equation on a clay material and observed that the prediction was not so good when compared with the measured data. Similar results were reported by Khalili and Khabbaz (1998) where $\chi = S$ did not

agree with the measured shear strength and proposed $\chi = \begin{cases} 1 & S < S_e \\ \left(\frac{S_e}{S} \right)^r & S \geq S_e \end{cases}$ where S_e = the air entry

suction for the soil and r = a material parameter.

(b) Equations under the independent stress state variable approach

Fredlund et al. (1978) developed the concept of soil phases and equilibrium states under applied stresses. These two phases (air and water) flow when subjected to pressures. The linear shear strength equation, Equation 2-32, was then defined in terms of two stress-state variables. The most convenient stress-state variables from the three stress variables are the net normal stress and matric suction.

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \quad (2-32)$$

where τ = Shear strength, c' = effective cohesion, ϕ' = effective friction angle, and ϕ^b = angle indicating a change in shear strength related to matric suction.

A series of Mohr circles corresponding to failure conditions on a two-dimensional plane can define the failure envelope for saturated soils using a tangent to the circles (Fredlund and Rahardjo, 1993). Similarly, for unsaturated soils, Mohr circles corresponding to failure envelopes when plotted in three-dimensional space produces a surface referred to as the extended Mohr-Coulomb failure for unsaturated soils (Fredlund and Rahardjo, 1993). Figure 2-28 demonstrates Mohr circles relating to failure envelopes plotted in a three-dimensional way.

Figure 2-28: Extended Mohr-Coulomb failure envelope for unsaturated soils (from Fredlund and Rahardjo, 1993).

From Equation 2-33, the shear strength equation for unsaturated soils when projected on the shear strength τ versus $\sigma - u_a$ plane can be recast as Equation 2-33, where C , the total cohesion, is a sum of the effective cohesion and the shear strength contribution due to matric suction.

$$\tau = C + (\sigma - u_a)_f \tan \phi', \text{ where } C = c' + (u_a - u_w)_f \tan \phi^b \quad (2-33)$$

Hence by plotting τ versus $(\sigma - u_a)$, at zero net normal stress, the intercept is given by C and the gradient of the line is given by $\tan \phi'$.

In the formulation of Equation (2-32), ϕ^b was assumed to be constant. However, Gan et al (1988) and Escario and Juca (1989) found out that ϕ^b is not a constant when tested over a large range of matric suction.

According to Lu (2008), matric suction is not a stress variable at a typical air-water-solid representative elementary volume (REV) level, but it can be a stress-state variable using stress definition on a REV, mechanical equilibrium, and logical reasoning. However, when matric suction is a stress variable, there is a coupling between the matric suction and the net normal stress when both are used simultaneously to describe the state of stress in unsaturated soils.

There is consensus that two stress-state variables generally describe the engineering behaviour of unsaturated soils. Matric suction is due to pore water existence in a meniscus form between soil particles. In unsaturated soils, the pore-water arrangement and the associated menisci are dependent on the particle surface properties, shape, size, and pore-water chemistry. Therefore, the effect of matric suction on the constitutive behaviour of soil is non-uniform and non-homogeneous (Leong 2016).

Lu and Likos (2006) argued that ϕ^b have uncertainties over a wide range of saturation and that limits its practical application. Other researchers argue that compressive indices of soils decrease with increasing suction (Pham, 2005, Alonso et al., 2013), increase with increasing suction (Wheeler and Sivakumar, 1995, Silva et al., 2002) or not affected by matric suction (Imhoff et al., 2004, Cui et al. 2010). These observations on the compression index complicate the interpretation of unsaturated soil behaviour and more research is needed.

Other proposed shear strength equations based on the two stress-state variables are shown in Table 2-6. Equations in Table 2-6 are valid for the full range of saturation and revert to the shear strength equation for saturated soils at full saturation.

Equations in Table 2-6 and Equation 2-32 can be expressed in the form of Bishop's χ , and are illustrated in Table 2-7. From Table 2-7, it is clearly shown that χ is expressed in terms of volumetric water content, degree of saturation, matric suction, and shear strength parameters. No unique relationship is observed with χ for different soil types (Coleman, 1962). Nonetheless, Table 2-7 reveals that there is an attempt to unify the different shear strength equations to greatly improve the application of unsaturated soil mechanics in practice.

Table 2-6: Shear strength equations for unsaturated soils

Reference	Shear strength equation	Notation
Vanapalli et al. (1996)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left(\frac{\theta - \theta_r}{\theta_s - \theta_r} \right) (u_a - u_w) \tan \phi'$	θ_r = residual volumetric water content θ_s = saturated volumetric water content
Shen and Yu (1996)	$\tau_f = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \left[\frac{1}{1 + (u_a - u_w) d} \right] \tan \phi'$ $\tau_f = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \left[\frac{1}{\cot \alpha + \frac{(u_a - u_w)}{\beta_1}} \right] \tan \phi'$	d = fitting parameter, α = initial angle of shear strength with respect to shear strength, and β = matric suction contribution to shear strength at the peak value of shear strength.
Bao et al. (1998)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left[\frac{\log \left[\frac{(u_a - u_w)_r}{(u_a - u_w)_b} \right]}{\log \left[\frac{(u_a - u_w)_r}{(u_a - u_w)_b} \right]} \right] x (u_a - u_w) \tan \phi'$	$(u_a - u_w)_r$ = residual matric suction $(u_a - u_w)_b$ = air-entry value
Khalili and Khabbaz (1998)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left[\frac{(u_a - u_w)}{(u_a - u_w)_b} \right]^{-0.55} x (u_a - u_w) \tan \phi'$	$(u_a - u_w)_b$ = air-entry value
Aubeny and Lytton (2003)	$\tau = c' + (\sigma - u_a) \tan \phi' + f_1(\theta) (u_a - u_w) \tan \phi'$	$f_1(\theta) = \begin{cases} \frac{1}{\theta} & \text{for } S = 100\% \\ 1 + \frac{S-85}{15} \left(\frac{1}{\theta} - 1 \right) & \text{for } 85\% < S < 100\% \\ 1 & \text{for } S \leq 85\% \end{cases}$
Tekinsoy et al. (2004)	$\tau = c' + (\sigma - u_a) \tan \phi' + [(u_a - u_w) + P_a] x \ln \left[\frac{(u_a - u_w) + P_a}{P_a} \right] \tan \phi'$	P_a = atmospheric pressure
Vilar (2006)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left[\frac{(c_{ult} - c')}{(c_{ult} - c') + (u_a - u_w) \tan \phi'} \right] x (u_a - u_w) \tan \phi'$	c_{ult} = ultimate shear strength

Table 2-7: Suction stress equivalent from proposed shear strength equations

Reference	Shear strength equation	Equivalent, χ
Fredlund et al. (1978)	$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b$	$= \frac{\tan \phi^b}{\tan \phi'}$
Vanapalli et al. (1996)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left(\frac{\theta - \theta_r}{\theta_s - \theta_r} \right) (u_a - u_w) \tan \phi'$	$= \left(\frac{\theta - \theta_r}{\theta_s - \theta_r} \right)$
Shen and Yu (1996)	$\tau_f = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \left[\frac{1}{1 + (u_a - u_w)d} \right] \tan \phi'$ $\tau_f = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \left[\frac{1}{\cot \alpha + \frac{(u_a - u_w)}{\beta_1}} \right] \tan \phi'$	$\left[\frac{1}{1 + (u_a - u_w)d} \right]$ $\left[\frac{1}{\cot \alpha + \frac{(u_a - u_w)}{\beta_1}} \right]$
Bao et al. (1998)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left\{ \frac{\log \left[\frac{(u_a - u_w)_r}{(u_a - u_w)} \right]}{\log \left[\frac{(u_a - u_w)_r}{(u_a - u_w)_b} \right]} \right\} \times (u_a - u_w) \tan \phi'$	$= \left\{ \frac{\log \left[\frac{(u_a - u_w)_r}{(u_a - u_w)} \right]}{\log \left[\frac{(u_a - u_w)_r}{(u_a - u_w)_b} \right]} \right\}$
Khalili and Khabbaz (1998)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left[\frac{(u_a - u_w)}{(u_a - u_w)_b} \right]^{-0.55} \times (u_a - u_w) \tan \phi'$	$= \left[\frac{(u_a - u_w)}{(u_a - u_w)_b} \right]^{-0.55}$
Aubeny and lytton (2003)	$\tau = c' + (\sigma - u_a) \tan \phi' + f_1(\theta)(u_a - u_w) \tan \phi'$	$= f_1(\theta)$
Tekinsoy et al. (2004)	$\tau = c' + (\sigma - u_a) \tan \phi' + [(u_a - u_w) + P_a] \times \ln \left[\frac{(u_a - u_w) + P_a}{P_a} \right] \tan \phi'$	$= \frac{[(u_a - u_w) + P_a]}{(u_a - u_w)} \ln \left[\frac{(u_a - u_w) + P_a}{P_a} \right]$
Vilar (2006)	$\tau = c' + (\sigma - u_a) \tan \phi' + \left[\frac{(c_{ut} - c')}{(c_{ut} - c') + (u_a - u_w) \tan \phi'} \right] \times (u_a - u_w) \tan \phi'$	$= \left[\frac{(c_{ut} - c')}{(c_{ut} - c') + (u_a - u_w) \tan \phi'} \right]$

Researchers (e.g. Karube et al., 1996; Lu and Likos, 2006) have made attempts to consider the non-uniformity and non-homogeneity of matric suction through the use of suction stress instead of only using matric suction. Lu and Likos (2006), Kim et al. (2010, 2013), and Chen et al. (2013) showed that the concept of suction stress applies to shear strength behaviour of unsaturated soils. The concept of suction stress is reviewed in the following section.

2.4.1.4: Suction stress

Suction stress is based on a combination of microscopic forces to form a macroscopic expression defined as suction stress. Lu and Likos (2006) utilized the concept of particle-scale to distinguish and conceptualize three types of forces in soils:

1. Active “skeletal” forces that propagated through soil grains,
2. Active “local” forces that concentrate at or near the interparticle contacts, and
3. Passive particle-particle contact forces balance the skeletal and local forces.

Under the saturated state, Terzaghi’s classic effective stress equation considers the soil-water system as an equivalent continuum medium with macroscopic stress defined at the boundary covering the active “skeletal” and passive particle-particle forces.

Under the unsaturated condition, the three types of forces have to be distinguished because the pore-pressure (macroscopic stress) disintegrates into several microscopic interparticle forces acting within the proximity of the grain contacts. The “local” forces (Van der Waals, double layer, cementation, surface tension and negative pore-water pressure forces) can be conceptually grouped into macroscopic stress referred to as “suction stress”. Details on the physical origin of suction stress are covered in Lu and Likos (2006).

Shear strength data plotted on a plane of net-normal effective stress and shear strength or net mean effective stress p' and deviatoric stress q can be used to quantify suction stress as illustrated in Figure 2-29. All the angles ϕ_1' and ϕ_2' in Figure 2-29 are effective internal friction angles at different saturated states. The apparent cohesions c_1' and c_2' estimated from shear strength tests at specific values of matric suction correspond to angles ϕ_1' and ϕ_2' , respectively.

A projection of apparent cohesion to the normal stress axis (horizontal axis) gives apparent tensile stress (σ_{s1} and σ_{s2}) as illustrated in Figure 2-29. Therefore, the apparent cohesion determined from shear strength tests at specific matric suction can be directly used to compute suction stress as $\sigma_s = c'/\tan\phi'$ where c' is the apparent effective cohesion at a given suction with ϕ' as the effective friction

angle. Though the calculation seems simple, the challenge remains on how to experimentally determine the suction stress hence, the need for extensive laboratory experiments to quantify the suction stress.

Figure 2-29: Suction stress characteristic curve from the Mohr-Coulomb failure envelope (from Lu and Likos (2006)).

2.4.2: Tensile strength

The presence of a tension zone in cohesive soils was first reported by Terzaghi (1943). He also acknowledged the function of tension in soils and how tensile stresses trigger slope failures. Soils near the ground surface experience multiple drying and wetting cycles according to weather and climatic conditions. As the soil dries, it experiences shrinkage and tensile cracks. These cracks affect the integrity of soil structures such as slopes, dams, and embankments in terms of permeability and strength. Cracks in the soil provide an easy pathway for rainwater infiltration and such phenomenon is commonly encountered in slopes and earth structures. Tensile cracks are highly influenced by the tensile strength of soils (Morris et al. 1992, Trabelsi et al. 2011, Vaniceek 2013, Shi et al. 2014, Li et al. 2019, Tang et al. 2019). Research shows that the matric suction of the unsaturated soils mainly influences the tensile strength of unsaturated soils (De Souza Villar et al. 2009, Yin and Vanapalli 2018).

Although tensile strength testing for soil is complex, the effect of cracking in earth structures has attracted several researchers into the tensile strength testing of soils. In practice and research, tensile strength testing has been conducted on even brittle materials like concrete and rock. Conventional shear and compression test equipment cannot correctly measure the cracking occurrences in Geotechnical structures. This has led to new developments in the methodologies, configurations, and laboratory test equipment to evaluate the tensile strength of soils. Tensile testing methods for soils are generally grouped depending on the method of load application: direct and indirect tensile tests.

Generally, the assumption is that the tensile strength for cohesionless soils is zero because it is hard to measure directly. Direct (e.g., axial tensile test, triaxial tensile test, bending test, hollow cylinder test) and indirect tensile test methods are the commonly used laboratory methods to compute the tensile strength of the soil. These are briefly described in the following sections.

2.4.2.1: Direct tests

Direct tensile strength tests usually involve constraining one of the boundaries of the test specimen and applying uniaxial stress to the opposite boundary. The direct tensile test is regarded as the only method where a specimen is subjected to true uniaxial tension (Peters and Leavell 1988, Win 2006). In a direct tensile test, it is assumed that uniform stresses are applied to the specimen, and torsional and bending stresses are absent. It is further assumed that the end constraints should not cause disturbances to the intermediate stresses.

This test involves a direct application of a tensile force to the specimen until failure along its longitudinal axis. Different simple geometrical shapes of specimens such as a prism, cylinder have been used to prepare specimens for tensile tests. The tensile strength is computed as a ratio of the maximum load supported by the specimen to the cross-sectional area where the load acts.

The direct tensile test is advantageous in that:

1. All stresses and strains can be directly computed and are homogeneous.
2. True uniaxial tension is satisfied.
3. Useful results are obtained from test results with no assumptions.
4. The true stress-strain relationship can be measured under tension since the direct tensile test is a uniaxial total stress test.

Several researchers such as (Tschebotarioff et al. 1953, Hasegawa and Ikeuti 1966, Ajaz and Parry 1975, Peters and Leavell 1988, Tang and Graham 2000, Trabelsi et al. 2010, Li et al. 2019, Murray and Tarantino 2019) have used direct tensile strength testing on the soil.

Despite the advantages, the validity of the tensile strength has been questioned by several researchers. This is a consequence of the difficulties associated with the test equipment and loading conditions which result in stress concentration and misalignment (Kennedy and Hudson 1968). Creep and volume changes as a result of sustained loading occurring in the material during testing have also been reported (Win 2006). Other general disadvantages include:

1. It is difficult to effectively clamp or hold the specimen at the ends.
2. Misalignment, stress concentration and eccentric loading cannot be eliminated.
3. The design of the apparatus to apply a uniform stress distribution is complicated and expensive in real practice.

To address some of the difficulties earlier on mentioned, a new direct tensile testing approach, equipment, procedure and techniques were developed by Leavell and Peters (1987). A relationship between tensile strength and suction of the soil could be studied.

2.4.2.2: Indirect tests

In this method, the tensile strength of the sample is indirectly determined. There exist several indirect tensile test methods that have been developed to address the challenges associated with the direct tensile test methods. The indirect tensile tests suffer the following disadvantages:

1. More suitable for brittle materials,
2. Assumption based on the linear elastic theory used to estimate tensile strength may not apply to all soil types,
3. Difficulty in obtaining the stresses directly from the loading and boundary conditions.
4. Compression and tension are mixed hence causing a complicated mode of failure.

Notwithstanding the setback of the failure to duplicate field loading conditions, research has shown that the indirect tensile test methods have certain advantages over the other types of tensile test. It is relatively simple, failure is located in a region of uniform tensile strength, the specimens and equipment are the same as the compression tests, the surface conditions of the specimen does not affect the failure and less variation of the test results (Kennedy and Hudson 1968).

Several studies have been conducted using the indirect tensile strength tests (Uchida and Matsumoto 1961, Kennedy and Hudson 1968, Khrishnayya and Eisenstein 1974, Al-Hussaini 1981, Dexter and Kroesbergen 1985, Das et al. 1995, Li and Wong 2013, Akin and Likos 2017, Pittaro 2019). The Brazilian tensile test (BTS) is the most frequently used.

The Brazilian tensile strength (BTS) test is an indirect tensile test originally developed for rock (ASTM D3967-16) but has been used to obtain the tensile strength of soils (e.g., Khrishnayya and

Eisenstein 1974, Das et al. 1995, Vesga and Vallejo 2006, De Souza Villar et al. 2009, Beckett et al. 2015, Iravanian and Bilsel 2016, Akin and Likos 2017). A review of the development of the BTS test can be found in Li and Wong (2013).

In the BTS test, a cylindrical-shaped specimen is positioned horizontally between two loading platens and gradually compressed up to failure along a diametrical plane. The tensile stress field is therefore generated at the failure point and the tensile strength is generally computed from the imposed load and the cross-sectional area of the specimen (Akin and Likos 2017).

In the BTS test, the applied load is compressive, but the failure mode is tensile if specific constraints on specimen geometry (thickness to diameter, t/d , and ratio) are met. In ASTM D3967 – 16, the t/d ratio is recommended to be within the range of 0.2 to 0.75. Akin and Likos (2017) found that the tensile strength is essentially constant when t/d ratio is greater than 0.42 for kaolin. Fahimifar and Malekpour (2012) also concluded that the Brazilian test greatly depends on the relative width of the loading strip and the length of the specimen. In this study, the t/d ratio of the test specimens was maintained as close to 0.6 as possible.

Furthermore, Frydman (1964) examined the applicability of the BTS and reported that for materials that show brittle failure, the compressive deformation does not affect the stress condition. Basing on the fore mentioned assumption, together with other considerations: the loading rate, testing temperature, composition and dimensions of the loading strip, and size and dimensions of the specimen (Akin and Likos 2017), the BTS method was adopted for the indirect measurement of the tensile strength of the soils used and the description of the method is covered in chapter 7 of this study.

Other tensile test methods include the hollow cylinder or ring tensile test where soil behaviour is observed under triaxial conditions with all the three principal stresses being varied. The tensile stress at the specimen's mean radius is calculated from Lamé's solution of stresses in a thick-walled pressure cylinder and it is assumed to be the tensile strength at failure. However, the stress field is not uniform throughout the cylinder wall and as a result, the stress-strain behaviour of the material must be known to estimate the maximum tensile stress. The advantage of this method is that all stresses are tensile and there is a direct relationship between the average stress and both the inner and outer pressure applied on the specimen.

The double punch method, modified from the compression testing techniques for brittle materials, was proposed by Fang and Chen (1971) for determining the tensile strength of soil. A compacted soil specimen (cylindrical) is compressed between two vertically loading platens by two circular disks

concentrically located on the specimen's surfaces. The tensile strength from this method is computed using the plasticity theory as compared to other indirect methods that employ elastic theories in the evaluation of the tensile strength.

Despite all the investigations conducted on tensile strength, the theory of soil failure under tension is still unclear. There exists a difficulty in developing a comprehensive theory because most tests conducted measure total stresses as compared to effective stresses. This has a bearing on the general understanding of unsaturated soils. As a result, attempts have been made by several researchers to predict the tensile strength of soil using the Mohr-Coulomb, Griffth and modified Griffth failure criteria as summarised by Lee and Ingles (1968). Whereas Griffth criterion has been reported to forecast excessive curvature in the compression zone of the strength envelope, the Mohr-Coulomb failure criterion has been reported to overpredict the tensile strength.

Basing on the failure criteria, researchers have developed models to predict the tensile strength of the soil, and these are discussed in the next section.

2.4.2.3: Tensile strength models

Several theoretical models (Fisher 1926, Morris et al. 1992, Lu et al. 2009, Varsei et al. 2016) and empirical models (Kim and Hwang 2003, Munkholm and Kay 2014, Yin and Vanapalli 2018, Wang et al. 2020) have been developed to predict the tensile strength of soils. Tensile strength models based on the macromechanics approach compared to the micromechanical approach are more attractive for practical applications (Yin and Vanapalli, 2018).

A summary of the tensile strength models based on the macromechanics approach is presented in Table 2-8. In Table 2-8, the models of Morris et al. (1992), Trabelsi et al. (2012), Tang et al. (2015), and Varsei et al. (2016) were developed for clayey soils while the models of Lu et al. (1992), Yin and Vanapalli (2018), and Wang et al. (2020) were developed for cohesionless soils.

Table 2-8: Summary of tensile strength models with consideration of matric suction using the macroscopic approach.

Reference	Tensile strength model's equations
Snyder and Miller (1985)	$\sigma_t = \frac{1}{f(s)} \chi (u_a - u_w)$ <p>where σ_t = tensile stress, $f(s)$ = proportionality factor based on stress concentration of Griffith (1924) fracture criterion and is a function of the degree of saturation, χ = Bishop's chi factor, u_a = pore-air pressure, and u_w = pore-water pressure.</p>
Morris et al. (1992)	$\sigma_t = 0.5 [c' + (u_a - u_w) \tan \phi^b]$ based on Frydman (1967) and Baker (1981) or $\sigma_t = (0.5 \text{ to } 0.7) [c' + (u_a - u_w) \tan \phi^b] \cot \phi'$ based on Bagge (1985) where, c' and ϕ' = effective shear strength parameters, ϕ^b = angle indicating rate of shear strength increase with respect to matric suction.
Lu et al. (2009)	$\sigma_t = \left[2_t \tan \left(\frac{\pi}{4} - \frac{\phi_t}{2} \right) \right] \sigma^s \tan \phi$ <p>where $\sigma^s = \begin{cases} u_w & \text{for } (u_a - u_w) \leq 0 \\ S_e (u_a - u_w) & \text{for } (u_a - u_w) \geq 0 \end{cases}$ $S_e = \frac{S - S_r}{1 - S_r}$</p> $(u_a - u_w) = \frac{1}{\alpha} \left[S_e^{n/(1-n)} - 1 \right]^{1/n}$ – van Genuchten (1980) SWCC equation S = degree of saturation, S_e = effective degree of saturation, S_r = residual degree of saturation, α and n = van Genuchten SWCC equation parameters.
Trabelsi et al. (2012)	$\sigma_t = \frac{2 \cos \phi'}{1 + \sin \phi'} \cdot \{ \sigma_s \tan \phi' + C^* \}$, $\sigma_s = \frac{A}{\tan \phi'} \cdot \left(\frac{ f(n^*) + f(n^*)}{2} \right) (u_a - u_w)$ <p>where</p> $C^* = B \cdot \left(\frac{ f(n^*) + f(n^*)}{2} \right) \quad f(n^*) = 1 - \left(\frac{n^*}{n_0^*} \right)^p$ <p>A, B = parameters, n^* = porosity, n_0^* = reference porosity, and p = material parameter characterizing the shape of the cohesion-porosity function</p>

Reference	Tensile strength model's equations
Tang et al. (2015)	$\sigma_t = \begin{cases} \left[2 \tan \left(\frac{\pi}{4} - \frac{\phi_t}{2} \right) \right] \sigma^s \tan \phi_t & 0 \leq S \leq S^c \\ \left[2 \tan \left(\frac{\pi}{4} - \frac{\phi_t}{2} \right) \right] \sigma^s \tan \phi_t + \sigma_{tr} & S^c \leq S \leq 100\% \end{cases}$ <p>where, $\sigma^s = S_e (u_a - u_w)$; $(u_a - u_w) = \frac{1}{\alpha} \left[S_e^{n/(1-n)} - 1 \right]^{1/n}$</p> <p>$\sigma_{tr}$ = residual tensile strength, and S^c = critical degree of saturation</p>
Varsei et al. (2016)	$\sigma_t = \frac{2 \cos \phi'}{1 + \sin \phi'} [\sigma_s \tan \phi' + c']$ <p>where, $\sigma_s = S_e^m (u_a - u_w)$, $S_e^m = \frac{S - S_r^m}{1 - S_r^m} = (S)^{\alpha^*}$</p> <p>$S_r^m$ = microscopic degree of saturation and α^* = material parameter (always greater than 1).</p>
Yin and Vanapalli (2018)	$\sigma_t = \left[2_t \tan \left(\frac{\pi}{4} - \frac{\phi_t}{2} \right) \right] \sigma^s \tan \phi$ <p>where,</p> $\sigma^s = S_e^k (u_a - u_w) + T_s a_{aw}$ $k = \begin{cases} \frac{1}{(n-1)} \frac{1}{1 - [(0.85 \pm 0.15) S_c]^{n/n-1}} & \text{for } C_u \leq 6 \\ \frac{1}{(n-1)} \frac{1}{1 - [(0.45 \pm 0.15) S_c]^{n/n-1}} & \text{for } C_u > 6 \end{cases}$ $\frac{S_0}{S_c} = \begin{cases} (0.85 \pm 0.15) & \text{for } C_u \leq 6 \\ (0.45 \pm 0.15) & \text{for } C_u > 6 \end{cases}$ $a_{aw} = \eta_s \frac{\pi}{e \cdot d_{50}} S_e^{\lambda_s} (1 - S_e) \quad ; \quad \eta_s = 0.73 C_u \quad ; \quad \lambda_s = \frac{S_r}{1 - S_r}$ <p>C_u = coefficient of uniformity, S_c = capillary degree of saturation, S_0 = degree of saturation corresponding to the peak tensile strength induced by capillary suction, e = void ratio, d_{50} = diameter at 50% passing.</p>

Reference	Tensile strength model's equations
Wang et al. (2020)	$\sigma_t = \left[2_t \tan \left(\frac{\pi}{4} - \frac{\phi_t}{2} \right) \right] \sigma^s \tan \phi$ <p>where $\sigma^s = S_e (u_a - u_w) + T_s a_{aw}$; $(u_a - u_w) = \alpha \left(S_e^{\frac{n}{1-n}} - 1 \right)^{\frac{1}{n}}$</p> $\alpha = \frac{12.07 T_s}{d_{60}} ; \quad n = \frac{1.07}{\log_{10} C_u} + 1;$ $a_{aw} = \eta_s \frac{\pi}{e d_{50}} S_e^{\lambda_s} (1 - S_e) ; \quad \eta_s = 0.73 C_u ; \quad \lambda_s = 0.3$

2.5: Direct shear testing of saturated soils.

The conventional direct shear apparatus has been used widely for many years. The conventional apparatus was designed by A. Casagrande in 1932 and is available in most soil laboratories. There has been increased use of the simple shear apparatus in research laboratories but the state of the art report by Saada and Townsend (1981) reported that the simple shear did not provide a real improvement to the direct shear.

The main concern associated with the shear box is that the stress conditions in the sample are not clear and the stress distribution is non-uniform. In addition to that, the physical limitations of the shear box give rise to stress concentration causing unequal distribution of shear. The non-uniform distribution of shear results in progressive failure which in effect lowers the strength. It has been reported that before the start of shearing, the lateral pressure corresponds to the earth pressure at rest and the principal stresses continue to rotate with continued shearing (Hvorslev 1960). Furthermore, the magnitude and direction of the stresses at every point vary. Also, the shape and orientation of the plane of rupture are not known. Bishop (1957) stated that it is uncertain whether the horizontal plane is a plane of maximum stress or a plane of rupture. Studies by Morgenstern and Tchalenko (1967) found out that angles of shear discontinuities to the horizontal initiate once shearing starts as a result of progressive deformation of the soil. The discontinuities keep forming at lower angles up to the residual strength when a continuous plane of rupture is developed. Skempton (1966) reported similar observations in the slip surface development in shear. Regarding stress-strain assessment, the thickness of the shear zone is non-uniform and not well-known which makes it hard to evaluate the

strain from the stress-strain relationship. As a result, a true stress-strain relationship curve is not acquired but rather an ‘average’ stress-shear displacement relationship.

Notwithstanding the disadvantages, the attraction of the direct shear test lies largely in the fact that:

- i) It is cheap and simple to operate, as no technical skills are required.
- ii) A thin specimen is used and the double porous stones reduce the drainage path and testing time for soils of low permeability as compared to the triaxial test,
- iii) Large shear displacements (strain) can be obtained in two directions,
- iv) It provides a close reproduction of certain field problems,
- v) Many correlations and much experience have been developed for comparison.

Many field conditions correspond to plane strain conditions, for example, the stability of embankment, slopes and retaining walls where the failure mode is direct shear-type where the shear deformation is approximately plane strain (See Figures 2-26a and 2-26b). Another example of direct or simple shear mode is the soil deformation around a driven and cast-in-place pile (See Figure 2-26c). The condition of plane strain is approximately equivalent to the rigid confinement imposed by the shear box parallel to the direction of shear.

Wroth (1984) reported that the principal axes of stress and strain increment are free to rotate in most in-situ testing devices. In conventional triaxial and plane strain apparatuses, the principal stresses are fixed and results are frequently correlated with field tests where the principal stresses are fixed but the difference is not treasured.

Field direct shear types of equipment have been developed by several researchers to try and obtain actual field measurements and obtaining large shear displacements without appreciable area of shear plane changes (Matsuoka et al. 2001, Fakhimi et al. 2008, Liu 2009).

(a) Stress condition along a circular slip surface

(b) Slip failure of an Embankment (Wroth 1984)

(c) loading due to downward displacement of a pile (Randolph and Wroth 1981).

Figure 2-30: Examples of Direct shear / Simple shear field situations.

2.6: Research gaps/Summary

Compaction

Commonly used compaction methods in the laboratory are static and dynamic compaction methods. The procedure for dynamic compaction has been standardized in ASTM and BS. However, the procedure of static compaction has not yet been standardized and different procedures are reported in

the literature. There is still debate on whether statically compacted samples and dynamically compacted samples have a similar structure at the dry of optimum and wet of optimum. It is therefore prudent to investigate whether strength is different between statically and dynamically compacted soil samples.

Suction measurement

Several methods for suction measurements are in use (Table 2-1) but not all methods apply to the whole suction range. Literature shows that the filter paper method is economical, easily accessible, easy to use, relatively accurate, and can be used to measure a wide range of suction. The test method has been standardized in ASTM D5298 - 16 (2010) however, the experimental procedures are still controversial, different calibration curves have been developed and the precision and bias for reporting results have not been determined hence more research is needed to re-evaluate the contact and non-contact filter paper suction measurements. The filter paper also requires extra precautions because strict protocols must be followed to avoid accidental moisture transfer. In addition, the moisture content of the filter paper is usually only determined once after it has been presumed to be in equilibrium with the soil suction. Hence to overcome the limitations of measuring moisture content of the filter paper, it is necessary to examine the feasibility of using other measurements such as the electrical resistivity (ER) of the filter paper instead to infer the suction.

Suction control techniques

The most used technique is the axis-translation technique which has drawn several discussions in recent years as regards its appropriateness in representing natural conditions. The use of the osmotic technique to control suction has several advantages as it represents the natural conditions but the degradation of both the cellulosic membrane and PEG solution commonly used in this technique with time poses a challenge. This makes the experiment questionable when it is to be conducted for a long period. In addition, there is a general assumption that at equilibrium, the matric suction of the soil specimen is equal to the applied osmotic suction neglecting the membrane effect and stress history of the soil. Hence, there is a need to investigate other combinations of osmotic solutions and membranes that can resist degradation as well as a need to conduct independent suction measurements to establish the final matric suctions of the specimens to conclude on the efficacy of using the osmotic technique to achieve a target matric suction.

Shear strength for unsaturated soil

From the literature, many equations have been proposed for the shear strength of unsaturated soils (e.g., Lambom, 1986; Vanapalli, et al. 1996; Oberg and Salifors, 1997; Bao et al., 1998; Khalili and

Khabbaz, 1998; Aubeny and Lytton, 2003; Tekinsoy et al., 2004; Vilar, 2006). Several shear strength equations for unsaturated soils are extensions of Terzaghi's (1936) effective stress equation for saturated soils (Equation 2-1). The most widely used equations are Bishop's (1959) single stress-state variable equation (Equation 2-2) and Fredlund et al. (1978) two stress-state variables equation (Equation 2-18). Limited success has been registered with the single stress-state variable approach to unsaturated soil mechanics. There is consensus that net normal stress and matric suction are the required stress-state variables to explain the unsaturated soil behaviour. However, studies by Kohgo et al. (1993) suggest that the effect of matric suction on the constitutive behaviour of soils is neither uniform nor homogeneous. Several researchers have tried to explain the non-uniformity and non-homogeneity of matric suction and they suggest that matric suction be replaced with suction stress (e.g., Karube et al., 1996; Lu and Likos, 2006). However, there is no procedure provided on how suction stress can be determined. Therefore, more research and laboratory experiments are needed to investigate these suggestions.

Constant water content test

The shear strength parameters of unsaturated soils have been investigated using modified triaxial and direct shear apparatuses by researchers (Ho and Fredlund 1982, Gan and Fredlund 1988, Escario et al. 1989, Rahardjo et al. 1995, Likos et al. 2010, Kato et al. 2011). Suction-controlled tests conducted using triaxial equipment usually take a long time and the equipment is expensive. Furthermore, Zhang (2015) showed that the stress path of unsaturated soils under constant matric suction cannot be controlled precisely. The use of conventional direct shear apparatus under constant water content condition is attractive for testing unsaturated soil but proper interpretation of data and results from direct shear apparatus is still scarce.

Tensile strength of unsaturated soils

Soils at the ground surface experience multiple cycles of drying and wetting. On drying, the soils experience shrinkage and cracks may appear. Conventional shear and compression test equipment cannot correctly measure the cracking occurrences in geotechnical structures. This has led to new developments in the methodologies, configurations and laboratory test equipment to evaluate the tensile strength of soils. Several tensile strength models have been proposed for unsaturated soils considering the effect of matric suction. However, the tensile strength models proposed are for either cohesionless (coarse-grained) or clayey (fine-grained) soils. It is attractive to have a tensile strength model that applies to both coarse-grained and fine-grained soils.

Chapter 3: Materials and Methods

3.1: Introduction

This chapter describes the soils and experimental methods used in this study. A summary of the experiment programme is shown in Figure 3-1.

Figure 3-1: Experiment programme

Two soils were used for the study. Direct shear tests, suction measurements and soil-water characteristic curve tests were conducted on statically and dynamically compacted specimens. Osmotic consolidation and osmotic technique of suction control were also investigated.

3.1.1: Basic soil properties

One of the soils used in this project is residual soil from the Bukit Timah (BT) Granite of Singapore. Basic soil property tests were conducted following ASTM standards. The test results are summarised in Table 3-1. Figure 3-2 shows the grain-size distribution of the residual soil.

Table 3-1: Summary of measured index properties

Soil property	Soil material	Standard
Specific gravity, G _s	2.65	ASTM D854-14
Atterberg limits		ASTM D4318 – 17
Liquid limit, (LL) %	56	
Plastic limit, (PL) %	36	
Plasticity index, (PI) %	20	
Gradation characteristics		ASTM D6913/D6913M-17 , ASTM D7928-17
Sand content, %	45	
Silt content, %	31	
Clay content, %	23	
Unified Soil Classification System (USCS)	MH	ASTM D2487-17

The other soil used in this study was a remoulded soil mixture of kaolin - bentonite in proportions of 70% – 30% (KB30) and 90% - 10%, (KB10), by dry mass. The physical and basic properties of the kaolin, bentonite, and kaolin - bentonite mixtures are summarised in Tables 3.2 and 3.3, respectively. The grain size distributions are shown in Figure 3-3.

The residual soil was used to investigate shear strength of unsaturated soils whereas the kaolin-bentonite mixture was used to investigate osmotic consolidation and osmotic technique. The bentonite was a Na-montmorillonite and the purpose for using bentonite was to simulate expansive soil.

Figure 3-2: Grain size distribution of residual soil used for this study.

Table 3-2. Bentonite and kaolin physical properties from the manufacturer.

Refined kaolin (KM20 GRADE)		API bentonite	
Moisture content (%)	Below 5.0	Marsh funnel Viscosity (Sec)	44
Viscosity (30% solution) (cp)	40 - 150	Liquid limit (%)	530
pH (30% solution)	4.5 - 6.5	Moisture content (% by weight)	13.4
Brightness (GE) (%)	7500%	PH	10.1
325 mesh residues (%)	Below 2.0	Free Swelling (ml)	32
Average particle size (μ)	8.2	MBA value	422.62
		Filtrate loss (ml)	17.6

Table 3-3. Basic soil properties

Soil Properties	Soil Type			
	Kaolin	Bentonite	KB30	KB10
	Values			
Specific Gravity, G_s	2.66	2.60	2.63	2.65
Liquid Limit, (LL) (%)	49	210	135	78
Plastic Limit, (PL) (%)	34	44	40	35
Plasticity Index, (PI) (%)	15	166	95	43
USCS classification	ML	CH	MH	MH

Figure 3-3: Grain size distribution of Kaolin, Bentonite, KB30 and KB10 soils.

3.2: Soil specimen preparation

This section describes the preparation of soil specimens used in the tests.

3.2.1: Compacted soil preparation

Residual soil sample from Bukit Timah Granite (BT soil) was air-dried, broken down and sieved through sieve No. 8 (2.36 mm) and about 1% of the material was retained on this sieve. Only soil passing through this sieve was used to satisfy the maximum grain size requirement for direct shear tests as specified in ASTM D3080/D3080M-11.

The soil materials passing sieve No. 8 were carefully mixed with distilled water to achieve different water contents. After thorough mixing, the soil mixtures were placed in Ziplock bags and kept in a high humidity chamber for at least one week to allow for moisture equilibration.

To obtain the compaction curve (moisture-density relationships), the soil was compacted using standard and modified Proctor efforts following ASTM D698 – 12 and ASTM D1557-12, respectively. The optimum water content and the maximum dry density were determined for the BT soil. The compaction curve with the zero-air void line and various degrees of saturation lines are shown in Figure 3-4. The optimum water content and maximum dry density of the soil are 21.5% and 1.62 Mg/m³, and 13% and 1.81 Mg/m³, for the standard and modified Proctor efforts, respectively.

Figure 3-4: Compaction Curve for the BT soil.

For this study, statically and dynamically compacted cylindrical specimens of diameter 63.5 mm and height of 20 mm were used. Compared to a standard Proctor sample diameter to height (101.6 mm x 116.4 mm), it is quite hard to trim the sample to obtain the required specimen as it requires a lot of time and effort to minimise specimen disturbance. This affects the quality of the specimens; therefore, an acrylic mould was fabricated to compact the specimens.

The acrylic mould is shown in Figure 3-5. The acrylic mould, 0.4 x 90.4 x 20 mm sits in the recess (90.5 x 90.5 x 6 mm) of a 120 mm square base that is 15 mm thick. The mould consists of an inward recess at the top of 6 x 5 mm for receiving the collar. A square collar of 90.4 x 90.4 x 25 mm is attached to the top of the mould to ensure that soil is compacted to fill the mould without spillage. The mould and the collar have a central hole of diameter 63.45 mm approximately equivalent to the specimen diameter required for the tests. The base has four screws at the bottom that help to fasten the mould to the base.

To ensure easy extrusion of the specimen into the direct shear box, holes were provided on the mould that matches the positions of both the hold down and lifting screws of the direct shear box. During extrusion, the specimen is properly guided into the shear box opening and gently pushed into the shear box.

Figure 3-5: Compaction acrylic mould

For compacting the soil specimen into the acrylic mould, the equivalent Proctor effort was computed based on equivalent energy per unit volume of soil. The compactive effort is computed using Equation 3-1.

The compaction is done in one layer for this study as the overall height of the specimen was 20 mm. The number of blows were computed for an equivalent standard and modified Proctor effort. The impact energy was transferred to the soil through a solid anvil (Diameter = 63.4 mm and height = 100 mm) that rests on top of the soil in the mould.

$$\text{compactive effort} = \frac{\left(\text{mass of rammer} \right) \times \left(\text{drop - height of rammer} \right) \times \left(\text{number of blows per layer} \right) \times \left(\text{number of layers} \right) \times g}{\text{mould volume}} \quad (3-1)$$

The number of blows obtained were five and nine blows for standard and modified Proctor efforts, respectively, to prepare the specimens in the acrylic mould. A summary of the compaction efforts in ASTM D3080/D3080M-11 and this study are summarised in Table 3-4.

Table 3-4: Comparison between the ASTM and adopted procedure of compaction.

Item compared	ASTM Procedure		Thesis procedure	
	Standard	Modified	Standard	Modified
	Proctor	Proctor	Proctor	Proctor
Mould height (mm)	116.4	116.4	20	20
Rammer weight (kg)	2.5	4.5	2.5	4.5
Rammer drop height (mm)	305	457	305	457
Rammer diameter (mm)	50.8	50.8	50.8/63.4	50.8/63.4
Compaction layers	3	5	1	1
Volume of the mould (mm ³)	943.7	943.7	63.34	63.34
Blows per layer	25	25	5	9

Before each compaction, the mould was secured to the base plate and the collar was secured to the top of the mould. The internal walls of the mould were oiled with a thin layer of hydraulic oil to reduce friction. With a known volume and dry density at a particular moisture content, the mass of soil required was computed and placed into the mould. The required number of blows as shown in Table 3-4 was applied to obtain standard and modified Proctor compacted soil specimens.

For statically compacted specimens, the displacement-controlled method was adopted. The compaction rate used was 1.27 mm/minute the lowest possible rate on the CBR machine used for compaction. The required mass of soil was placed into the acrylic mould and compressed in the CBR compression machine until the required height of the specimen (20 mm) was achieved (refer to Figure 2-13).

Typical compaction specimens used in the study are shown in Figure 3-6, with the smaller specimen obtained using the acrylic mould and the larger specimen obtained using the standard ASTM mould.

Specimens from Acrylic mould

ASTM standard compaction mould

Figure 3-6: Typical compaction specimens

3.2.2: Kaolin – bentonite mixture preparation

Two series of soil specimens were prepared by mixing powdered kaolin and bentonite with water to form a slurry. Water corresponding to 1.5 times the liquid limit of KB30 and 0.63 times the liquid limit of bentonite of KB10 were used to form slurries of KB30 and KB10, respectively.

The slurry was transferred to a mixer (Figure 3-7a) to obtain a homogeneous slurry. After mixing, the slurry was then transferred to a consolidation tank, 300 mm in diameter (Figure 3-7b) and consolidated under a vertical pressure of 125 kPa. This vertical pressure was chosen so that firm specimens could be obtained from the consolidated sample.

The settlement was observed regularly until there was no more settlement which is about a week. After consolidation, the sample was extruded from the consolidation tank and cut into blocks of 70 x 70 x 50 mm. The blocks were wrapped with three to five layers of cling wrap followed by two layers of aluminium foil, waxed, and placed into the humidity cabinet for later use.

(a) Mixer

(b) Consolidation tank

Figure 3-7: Blender and consolidation tank.

3.3: Suction measurements

Three methods of suction measurements were used in this study: i) filter paper method ii) high capacity tensiometer and, iii) chilled mirror dew- point. Most of the general descriptions have been covered in section 2.4.2, the adopted procedures for this study are described below.

3.3.1: Filter Paper Method

3.3.1.1: Using existing calibration curves

Whatman No. 42, ashless type, diameter 42.5mm were used in this study because of their thickness, smaller pore sizes (Chandler and Gutierrez 1986) and ability to measure suction over a wide range (Sibley and Williams 1990). The filter paper method for the measurement of soil suction is standardized in ASTM D5298-16. When the filter paper is in intimate contact with the soil specimen, the filter paper and the soil specimen will come to suction equilibrium by capillarity flow. As mentioned earlier, when the soil specimen and the filter paper are not in contact, it is only total suction that is measured.

The contact and non-contact filter paper techniques were used to measure the matric and total suction of the soil specimens, respectively, in this study. Static and dynamically compacted specimens were prepared over a wide range of water contents. Dynamically compacted specimens were prepared

using both the standard and modified Proctor efforts whereas statically compacted specimens were only prepared according to the standard Proctor compaction curve.

Standard Proctor compacted specimens were prepared according to the flow chart shown in Figure 3-8. Firstly, two compacted samples of about 101.6mm x 116.4mm (diameter x height) were each divided into approximately three equal disks (specimens) of about 101.6mm x 38mm each. These specimens were marked Xa, Xb and Xc. The notation “X” is the sample number while a, b and c indicate top, middle and bottom with respect to the compacted sample.

For the bottom specimens, a thin layer of soil that was in contact with the base of the mould was scraped off to remove the soil that may contain some mould oil that was used to reduce friction between the soil and mould during compaction so as not to contaminate the filter paper measurement.

A schematic representation of the six soil specimens for the two compacted soil samples of each water content as used for the filter paper test is shown in Figure 3-8. Specimens 1a and 1b were combined to form one set of measurements (SC1A), specimens 1c and 2a were combined to form measurement SC1B then specimens 2b and 2c were combined to form measurement SC1C.

For the modified Proctor effort, specimen preparation followed the same procedure as that for standard Proctor effort specimens except on the dry side of optimum where the samples could only be split into two specimens because the sample was compacted in five layers instead of three and separation into three specimens is difficult.

The statically compacted specimens were prepared using a CBR machine as explained in Section 3.3.1. After compaction, the specimens were extruded from the mould and quickly wrapped with cling wrap to avoid any moisture loss.

For each compaction water content, three sets of specimens were prepared and referred to as Samples 1, 2 and 3 giving six contacts (3-top and 3-bottom) and three non-contact filter paper results. Each sample consists of two specimens. The arrangement of the filter papers in each sample is shown in Figure 3-9.

Although ASTM 5298-16 recommends the use of three filter papers when setting up the contact filter paper method (for matric suction measurement), only one piece of filter paper was placed in contact with the soil at both ends (top and bottom) in this study. Leong et al. (2007) found that more pieces of filter papers lead to overestimation of the matric suction. To ensure proper contact, an acrylic plate (of diameter equal to the specimen diameter) was used to cover the filter paper at the ends of the specimen (see Figures 3-9).

Figure 3-8: Schematic representation of standard Proctor dynamically compacted specimens.

For the non-contact filter paper (total suction measurement), the filter paper was placed in between two layers of Polyvinyl Chloride (PVC) mesh that are secured together by a cable tie and placed in between the two soil specimens for equilibration (Figure 3-9). This was to ensure that the filter paper would not be in contact with the soil sample as well as minimizing the air space between the filter paper and the soil specimens as illustrated in Figure 3-9.

For all the soil specimens (dynamically and statically compacted), the same arrangement (Figure 3-9) was adopted. Each sample of two specimens was wrapped with at least three layers of cling film and two layers of aluminium foil and each sample was marked.

Figure 3-9: Filter paper arrangement for statically compacted specimen

The wrapped samples were placed inside an air-tight container and placed in a foam box. The form box was stored in a temperature-controlled room ($23^{\circ}\text{C} \pm 1^{\circ}\text{C}$) to minimise temperature fluctuations and moisture loss by evaporation. Although ASTM D5298-16 recommends a minimum period of 7 days for filter paper equilibration, the samples were tested after a period of between 14 - 21 days to allow for full equilibration of the suction between the soil and the filter paper.

After the equilibration period, the water content of the filter papers was determined following ASTM 5298-16. For this study, the calibration curves for Whatman No. 42 filter paper given in Leong et al. (2007) shown in Figure 3-10 were used. In the tests, whenever it was required to handle the soil

specimen, non-powder gloves were worn and a pair of tweezers was used to handle the filter paper to avoid contamination of the filter paper.

Figure 3-10: Total and Matric suction calibration curves (Leong et al., 2007)

3.3.1.2: Using electrical resistivity method

The filter paper can be assumed to be a thin disk and its electrical resistivity (ER) can be determined using the principles for measuring the ER of a thin disk (e.g., Kouba and Nader 2010).

The set-up for measuring ER of filter paper follows that in Figure 3-11, a current is applied to contacts 1 and 4 and the resistance is measured at contacts 2 and 3. Various angles of θ_0 can be used where:

$$\tan \theta_0 = \frac{L_{23}}{L_{12}} \quad (3-2)$$

The resistance measured R_m is given by:

$$R_m = \frac{4\rho}{\pi d} G(\theta_0) \quad (3-3)$$

where ρ = resistivity, d = diameter of filter paper and $G(\theta_0)$ = unitless geometric factor. For a particular set-up (i.e., fixed d and θ_0), R_m can be used as the proxy for ER of the filter paper.

Figure 3-11: Contact configuration for the electrical resistivity determination of a thin disk (from Kouba and Nader (2010)).

Three different test set-ups were used. The first test set-up was to determine the relationship between ER and the water content of the filter paper. The second test set-up was to establish the relationship between suction and water content of the filter paper and equilibrium time. The third test set-up was to demonstrate suction measurement using ER of the filter paper without handling the filter paper.

The first test-up consists of two identical acrylic blocks (Figure 3-12). Each block consists of an acrylic disk of thickness 5 mm and diameter of 60 mm and an acrylic cylinder of diameter 30 mm and height 30 mm. Four copper plates (contacts) were attached to the acrylic disk.

To obtain the ER-moisture content relationship of the filter paper, a dry filter paper was initially wetted to water content between 65-75%. From the calibration curves of Whatman No.42 filter paper, a water content between 65 – 75% corresponds to suction of about 20 kPa. Hence, the filter paper will measure soil suction greater than 20 kPa by a drying process. The moist filter paper was then placed centrally on the acrylic disk of one acrylic block while the second block was placed on top of the filter paper so that the four copper plate contacts are aligned and in contact. Direct current at 2V was then applied from one side via contacts 1 and 4 for not more than three seconds to avoid electroosmosis and the ER was measured from the other two contacts (2 and 3) using a multimeter as shown in Figure 3-13. The filter paper was then transferred to an analytical balance to determine its mass. The filter paper was then allowed to dry at room temperature in the analytical balance. When the mass of the filter paper changed by a few per cent, it is again transferred to the acrylic block for resistance measurement, and the procedures were repeated until the filter paper was almost dry. The final water content and dry mass of the filter paper were then determined by drying in the oven. The other water contents of the filter paper were then back-calculated from the dry mass of the filter paper.

Figure 3-12: Acrylic blocks and copper plate contacts

Figure 3-13: Schematic drawing of set-up to measure electrical resistivity of filter paper.

In the second test set-up, sodium chloride solutions prepared to target suctions of 100, 300, 500, 1000, 3000 and 5000 kPa were placed into airtight containers. The suctions of the salt solutions prepared were checked by measuring with electrical conductivity meter (ES-12) and chilled mirror hygrometer (WP4) (Leong et al. 2003) as shown in Table 3-5. A dry filter paper initially wetted to a water content of between 65 – 75% was placed on a plastic mesh supported on plastic members attached to the insides of the airtight container leaving a gap of about 10 mm as shown schematically in Figure 3-14(a) and then left for suction equilibration (Figure 3-14(b)).

The mass of the filter paper was weighed regularly until the change in mass was negligible over a day. The procedures are like those used by Houston et al. 1994) and Leong et al. (2002) for calibrating

initially dry filter papers over salt solutions to obtain the total suction calibration curves. At low suctions, condensation may occur on the lid and extra caution was taken when opening the lid to ensure that no condensate falls onto the filter paper. Once the mass of the filter paper was constant, it was rewetted to 65-75% and placed into another airtight container with a higher concentration salt solution and the procedures were repeated until the final concentration salt solution. The final water content and dry mass of the filter paper were then determined, and the other water contents of the filter paper were then back-calculated.

Table 3-5: Suction of prepared salt solutions

Target suction (kPa)	Suction using ES-12 (kPa)	Suction using WP4C (kPa)*
100	99	-
300	299	-
500	500	-
1000	1017	1194
3000	2956	3051
5000	4886	5078

*Measures suction of > 1000 kPa (ASTM D6836-16)

Figure 3-14: Calibration under salt solution

In the third set-up, two identical L-shaped holders were 3-D printed (Figures 3-15(a)). The two holders were designed to sandwich two 4 mm thick acrylic rings with a moist filter paper in between. The outside diameter of the acrylic ring is 55 mm, and the central hole diameter is 45 mm. Similar to test set-up 1, four copper plate contacts were attached to the acrylic ring as shown in Figure 3-15(b) for ER measurements. On the other face of the holder, a grooved recess was provided to allow an air-tight container with its bottom removed to be epoxied to the holder.

A filter paper wetted to 65-75% water content was positioned between the two acrylic rings and the L-shaped holders were then fastened together using bolts and nuts (Figure 3-15(c)). As the filter paper is held between the two L-shaped moulds, the mould creates a physical distance between the specimens in the containers and the filter paper. The specimens cannot practically come into contact with the filter paper because of the set-up and hence, no contamination.

Figure 3-15: Experimental setup for ER measurement from filter paper.

The small gap between the two holders on the outside was further sealed using cling wrap and tape. The airtight container is to allow soil specimens to be inserted and capped. The air space between the specimen and the filter paper is minimized to reduce the equilibrium time of the filter paper. In this study, a sponge soaked with a salt solution was inserted into the container to act as the suction “source” (Figure 3-15(d)). The ER of the filter paper was measured at regular intervals until it was

almost constant ($< 3\%$) indicating suction equilibrium. The sponge was then replaced with another sponge soaked with a higher concentration salt solution and the test was repeated. Salt solutions of suctions 100, 300, 500, 1000, 3000 and 5000 kPa were used. After the last test at suction of 5000 kPa, the set-up was disassembled quickly, and the final water content and dry mass of the filter paper were determined.

3.3.2: High capacity tensiometer

As discussed in section 2.4.2.4, the high capacity tensiometer made by Wijaya (2017) was used to measure matric suction by placing the soil specimen in intimate contact with the high air entry (HAE) disk.

The process started with the saturation of the HAE disk by applying a high water pressure of 1800 kPa (for one week) as compared to the HAE disk value of 1500 kPa. A vacuum was then applied to ensure that all the dissolved air in the reservoir was removed. The applied water pressure would enable water flow through the HAE disk to the bottom of the reservoir.

A kaolin paste prepared at water content close to the liquid limit was applied on the tip of the HCT to provide hydraulic continuity between the soil specimen and the HCT. The paste also helps to reduce the evaporation from the HAE disk before the specimen is placed and to improve contact condition especially when the soil surface is irregular as suggested by Oliveira and Marinho (2008). To ensure there was actual contact, a one kg mass was placed on top of the soil specimen during the measurement process. A schematic drawing of the high capacity tensiometer used in this study is shown in Figure 3-16. For most of the observations made in this study, scenario one discussed in section 2.4.2.4 was observed and the equilibrium time depended on the suction value.

Section view of the high capacity tensiometer Perspective view of the high capacity tensiometer

Figure 3-16: Schematic of the high-capacity tensiometer used in this study (from Wijaya 2017).

3.3.3: Chilled mirror dew-point method

The total suctions for soil specimens were conducted using the chilled mirror dewpoint in addition to the non-contact filter paper method. The measurements were conducted following (ASTM D6836-16, Leong et al. 2003). Researchers have reported that when the proper procedure is followed, the chilled mirror dew-point technique is the most reliable method for total suction measurements and can be used as a reference for evaluating other methods (Agus and Schanz 2007).

The WP4C dew-point hygrometer from Decagon Devices Inc. which can measure relative humidity to an accuracy of $\pm 0.1\%$ was used in this study. Before conducting any test, the device was turned on for about 30 minutes to warm up the electronics. The sample cups were halfway filled with soil and instantly placed in the drawer of the WP4 until equilibrium was achieved.

To improve on the accuracy of the results, there was a time lag between different suction measurements and periodic calibration of the device using standard saturated salt solutions of sodium chloride, potassium chloride and potassium sulphate.

With a measured relative humidity and temperature, Equation 2-7 was used to compute the suctions of the specimens. Equilibrium was achieved within minutes for each soil specimen.

3.3.4: Pore –fluid squeezing technique.

The osmotic suction of the soil specimens was conducted by measuring the electrical conductivity of the pore water extracted from the soil specimen. A small amount of the soil specimen was placed into the pore water squeezer (Figure 3-17a) and a pressure of up to 10 MPa was exerted on the soil specimen using a compression machine (Figure 3-17b) and the pore water was collected in a syringe (Leong et al 2007). The electrical conductivity of the pore-water was measured using the Horiba B-173 and ES-12 electrical conductivity meter (Figures 3-18a and 3-18b). The Horiba 173 is a portable compact electrical conductivity meter with a measurement range of 0 – 19.9 mS/cm and accuracy of $\pm 2\%$ full scale. The Horiba ES-12 electrical conductivity meter has a measurement range of 0 -199.9 mS/cm and an accuracy of 0.5% full scale. More details of the pore water squeezer were discussed in section 2.4.2.3.

The Horiba B-173 has the advantage over the Horiba ES-12 conductivity meter as it can measure the electrical conductivity using a single drop of fluid (~ 1 ml). The osmotic pressure π , was estimated using Equation 3-4 (Leong et al. 2007). where p_a is the atmospheric pressure ($=101.325\text{kPa}$) and EC

$$\pi = p_a (0.31 \text{EC}^{1.15}) \quad (3-4)$$

is the electrical conductivity in mS/cm. The WP4 measures the relative humidity (RH) in the air space above the sample in a sealed environment under constant temperature condition.

(a) Pore-water squeezer

(b) Compression machine

Figure 3-17: Compression machine and pore-water squeezer

(a) Horiba ES-12

(b) Horiba B-173

Figure 3-18: Electrical conductivity meters used in this study.

3.4: Soil-water characteristic curves

In this study, the soil-water characteristic curves were obtained following methods C (pressure plate extractor) and D (chilled mirror hygrometer) specified in the ASTM D6836-16. A pressure plate extractor and chilled mirror hygrometer were used for suction up to 1000 kPa and above 1000 kPa respectively. The HCT was also used to supplement the pressure plate tests for suctions less than 1500 kPa.

3.4.1: Axis-translation method

The pressure plate setup shown in Figure 3-19 consists of a pressure chamber with a saturated HAEV ceramic disk where the specimens are placed. hydraulic continuity between the saturated ceramic disk and a water burette outside the air-pressure chamber is provided by a neoprene rubber sheet (water reservoir) underneath the ceramic disk (Leong and Rahardjo, 2002).

(a) Schematic

(b) Photo

Figure 3-19: Pressure plate setup

Pre-saturation of the SWCC specimens was conducted before the placement in the pressure plate. Specimens were allowed to absorb water through a filter paper and cotton wool that are placed at the bottom and top of the specimen in a saturation container (a plastic box with a container). The water does enter directly into the specimen but rather through a fully saturated cotton wool. This helps to prevent erosion of the specimens. Specimens were left in their compaction moulds throughout the pre-saturation period to limit the lateral swell of the specimens. Trial tests were conducted to establish the approximate duration for the specimens to reach full saturation and the results indicated that 24 hours were sufficient to attain full saturation.

The initial mass and dimensions of the saturated soil specimens were measured and placed onto the saturated ceramic disk in the pressure plate. The suctions in the pressure plate were varied in steps according to ASTM D6836-16 from 10 to 1000 kPa. For every applied suction, the weights of the soil specimens were monitored regularly to establish the equilibrium state. Equilibrium at a given suction was assumed when there was negligible change in the mass of the specimen (less than 0.05g). At equilibrium, the mass and dimensions of the specimen corresponding to that particular suction were measured and the next suction was applied by adjusting the air pressure. At the end of the final suction (1000 kPa), the water content of the soil specimen was determined and used to back-calculate the water content for each previous matric suction to produce its SWCC.

3.4.2: Drying method

The HCT was used on both compacted and pre-consolidated specimens. For the compacted specimens, a pre-saturation of the specimens was conducted then left to air dry in a temperature-controlled environment to ensure the moisture distribution was uniform. For the pre-consolidated specimens, the specimens were trimmed before being left to air dry under similar conditions as the compacted specimens. The masses were regularly monitored, and the suctions were measured at every mass change interval depending on the range of suction. At lower suctions, the mass change was bigger as compared to the mass change at a higher suction range because a small mass variation will result in a bigger variation in suction at the higher suction range than in the lower suction range. After every suction measurement, water pressure was applied to the HCT to keep it saturated until the next suction measurement.

Both the mass and suction of the soil specimens were monitored concurrently to ensure that the AEV of the HCT was not exceeded during a suction measurement. Therefore, close to the AEV of the HCT, the water content of the soil specimen was determined and used to back-calculate the water content for each matric suction level to produce its SWCC.

3.4.3: Chilled mirror dew-point test

The WP4 was used for measuring suctions above 1000 kPa. After saturation, a piece of the specimen was cored out from the sample using a specially designed corer fashioned out of a PVC pipe of external diameter 35 mm. The specimen was placed into the sample cups used for the WP4.

The drying process was conducted using air-tight desiccators with saturated salt solutions at the base. This was conducted in a temperature-controlled environment range of $24 \pm 4^\circ\text{C}$. The masses of the samples (soil + cup) were regularly monitored as the soil dries and using the WP4 to measure the suctions. The drying continued until the soil had reached equilibrium with the saturated salt solution

(no further change in the mass). The cups were then placed in an open environment within the laboratory for further drying and both the change in mass and suction monitored. The process continued until there was no further change in the mass. Figure 3-20 illustrates the drying process in (a) desiccator and (b) an open environment.

Figure 3-20: Sample drying process.

With the relative humidity measured for different masses, the suction was computed using Equation 2-7. Similar to the pressure plate and HCT, the water contents at different masses were back calculated from the final water content of the individual specimens. The suctions were combined with those from the pressure plate and HCT to form a complete SWCC for each specimen.

3.5: Osmotic technique and osmotic consolidation

As mentioned earlier in section 2.4.3, several methods exist to control matric suction. The osmotic technique is an alternative method of inducing matric suction in a soil specimen by the use of a semi-permeable membrane and a salt solution. Ideally, only water molecules can pass through the membrane. Water from the soil specimen diffuses through the membrane by osmosis due to the osmotic gradient created by the salt solution on the opposite side of the membrane. This induces matric suction in the soil. The increase in the salt solution concentration (increase in osmotic pressure) increases the matric suction of the soil specimen.

3.5.1: Osmotic oedometer

The osmotic oedometer used in this study shown in Figure 3-21 is modified from a conventional oedometer. The modification consists of the addition of a reverse osmosis (RO) membrane, a peristaltic pump and sodium chloride (NaCl) solution reservoir.

Figure 3-21: Schematic drawing of updated osmotic oedometer

3.5.1.1: The peristaltic pump calibration

The peristaltic pump is GOSO brand-Model AB11 with a DC power input of up to 12V. The pump flow range is 0.1 – 100 ml/minute with a speed range between 0.1 – 100 rpm equipped with a pump tubing of 2.5 x 4.7 mm inside to outside diameters. The pump flow rate depends on the power supply with a full flow rate at the full voltage supply of 12V. Figure 3-22 shows a calibration setup of the peristaltic pump rate for the setup. The voltage supplied to the peristaltic pump was adjusted such that the outflow rate from the reservoir is the same as the inflow rate into the reservoir. This is done by ensuring that the meniscus level in the calibration tube is at the same level as the top of the porous stone and the membrane is not lifted from the porous stone.

Figure 3-22: Calibration of the peristaltic pump to circulate the salt solution through the base of oedometer.

The NaCl solution must flow through the porous stone as the inlet and outlet are on diametrically opposite sides of the porous stone. The flow is ensured to be at a constant velocity by the peristaltic pump. The setup was further checked by positioning a steel dummy specimen of the same dimensions as the soil specimen on top of the membrane to ensure that the application of the specimen would not cause a variation in the flow rates.

This system allows a matric suction to be applied to the soil specimen under a given vertical stress. If the volume or mass measurement device is added to the setup, it can be used to determine the soil-water characteristic curve of a soil specimen under specified vertical stress by changing the concentration of the salt solution after each equilibrium point. Equilibrium was assumed to have occurred when the deformation of the soil specimen was less than the resolution of the LVDT (0.01 mm) over six hours.

3.5.1.2: The RO membrane

The RO membrane used is a DOW Filmtec SW30HR which allows water molecules to pass freely but not dissolved salts and inorganic molecules, as well as organic molecules with a molecular weight greater than 100 (Dow Water and Process Solution 2020). The RO membrane has a minimum salt rejection of 99.6% and is a thin film composite membrane consisting of three layers: a polyester support web, which provides the major structural support and is made to provide a hard and smooth surface with no loose fibre, a microporous polysulfone interlayer that is cast onto the surface of the polyester support web to provide a substrate for the salt barrier layer because the web is too porous and irregular and the salt barrier layer consisting of an ultra-thin layer of polyamide. The microporous polysulfone interlayer provides strong support for the ultra-thin polyamide barrier which is about 2000 angstroms (0.0002 mm) thick to withstand high pressures. For RO membranes, the ultra-thin polyamide barrier is relatively thick and enhances the membrane resistance to both mechanical stresses and chemical degradation. The membrane also has some short-term resistance to chlorine attack. The membrane can operate within a wide range of pH (2 to 11), temperatures up to 45°C, and can perform over several years (DOW water and process solutions 2020). The permeability of the RO membrane computed using the falling head method was 3.08E-09 cm/s.

3.5.1.3: Salt solutions

Laboratory grade sodium chloride was used to prepare the salt solutions for the study. Different concentrations of sodium chloride solutions were prepared to give different osmotic suctions. The concentration of sodium chloride solution was based on the osmotic suction defined in terms of the molarity of the salt solution through the use of the van't Hoff equation (USDA 1954, Campbell 1985)(Figure 2-4).

The osmotic suction of the NaCl solution was checked using electrical conductivity meters (described in section 3.4.4) and chilled mirror dew-point device (WP4) (Leong et al. 2003, 2007) to ensure that the target osmotic suction was achieved. When measuring the relative humidity due to a salt solution, the osmotic suction can be obtained using Equation 2-7:

Figure 3-23 shows a comparison of the target osmotic suctions versus the measured osmotic suctions using the Horiba B-173, Horiba ES-12 and WP4 devices. The reading given by Horiba ES-12 electrical conductivity meter is the most consistent and reliable, followed by the Horiba B-173 electrical conductivity meter. The WP4 is only reliable for suctions above 1000 kPa.

Figure 3-23: Comparison between target osmotic suction and measured osmotic suction from electrical conductivity meters and WP4 device.

3.5.1.4: Test procedures

In this study, the conventional oedometer with a load lever arm ratio of 1:10 was used to conduct the one-dimensional volume change tests using ASTM D2435/D2435M-11 for guidance. The soil specimen was obtained from the soil blocks prepared earlier using the oedometer ring of 63.5 mm internal diameter and 19 mm height which have a sharp cutting edge. A thin layer of silicone grease was smeared on the inner surfaces of the oedometer ring before cookie cutting the specimen from the soil block to reduce friction between the specimen and the oedometer ring (ASTM D2435/D2435M-11). The soil specimen contained in an oedometer ring was placed in contact with the RO membrane on its bottom. Below the RO membrane, is the porous stone. The soil specimen was then inundated in water and consolidated to the specific effective vertical stress of 64 kPa.

At equilibrium with the vertical stress of 64 kPa, the water in the oedometer cell was removed by siphoning and the NaCl solution contained in the reservoir was circulated through the porous stone under a low-pressure head and returned to the reservoir via a peristaltic pump as shown in Figure 3-21. The reservoir contains a large volume of NaCl solution (500 cm³) compared to the volume of pore

water in the soil specimen (about 0.43 cm^3) to ensure that the concentration of the NaCl solution remains almost constant during the test. In the current setup, the suction and volume or mass of the water exchange is not measured during the experiment, but matric suction and water content of the soil specimen were measured at the end of every test. The effect of the osmotic technique was observed by comparing the results with an identical (control) test but with a continuous circulation of distilled water through the base of the oedometer. The tests were performed in an environment where the ambient temperature range is between $24 \pm 4^\circ\text{C}$. To minimize water loss from the soil specimen by evaporation, the top cap of the setup was covered by moist cotton wool. The test procedures were repeated with identical specimens but with a different concentration of NaCl solution.

3.5.2: Osmotic consolidation

As mentioned in section 2.4.3.3, the osmotic technique should not be mixed up with osmotically induced or osmotic consolidation. In this thesis, the osmotic consolidation is studied by comparing volume changes of two specimens. Conventional oedometer equipment with a load lever arm ratio of 1:10 was used to conduct the one-dimensional volume change tests following ASTM D2435/D2435M – 11. Both KB30 and KB10 specimens were cut out from the blocks using the oedometer ring of 63.5 mm internal diameter and 20 mm height. Silicone grease was smeared on the inner surfaces of the oedometer ring to reduce friction between the specimen and the oedometer ring (Fang et al. 2004). For each kaolin-bentonite mixture (KB30 and KB10), two specimens were prepared. One specimen was consolidated from 32 kPa to 2048 kPa in the oedometer, using a load increment ratio of 1 for successive loads. This gives the usual e -log σ' curve. For the osmotic consolidation, the other specimen was first consolidated from 32 kPa to 64 kPa. After which, the water in the oedometer cell was replaced with the saturated salt solution and settlement was observed until no further settlement was observed. Mechanical consolidation then resumes, and the specimen was consolidated up till 2048 kPa. This was repeated for each saturated salt solution (Potassium sulphate (K_2SO_4), Potassium chloride (KCl) and NaCl) and each kaolin-bentonite mixture (KB10 and KB30).

3.6: Direct shear test

3.6.1: Apparatus

The direct shear test apparatus was used to determine the shear strength parameters of the compacted soil. The apparatus utilizes a lever loading piston for the application of the vertical load to the sample with a lever arm ratio of 10:1. The direct shear apparatus is fitted with mechanical dial gauges for the measurement of vertical and shear displacements. The maximum shear displacement of the apparatus is 20 mm and a strain rate ranging between 0.0001 to 10.0 mm/minute can be applied. The maximum

vertical force and horizontal shearing force of the apparatus are 10 kN and 5 kN, respectively. (Shear test user manual, Vj Technology, 2017).

3.6.2: Test procedures

The direct shear tests followed ASTM D3080/D3080M-11 as closely as possible. The compacted specimen in the direct shear box was weighed and then placed into the direct shear apparatus. A normal load was placed on the soil specimen and the vertical settlement was observed until strain equilibrium was achieved. For the saturated soil specimen, the rate of shearing was set at 0.00694 mm/min. Shearing was done at a slower rate for saturated soil specimens to ensure that the shearing was performed at fully drained condition. Shearing was conducted until a maximum shear force was obtained or until a horizontal displacement of 15 mm. For the saturated soil test, the shear box containing the compacted soil specimen was inundated inside the direct shear apparatus for at least 48 hours before application of the normal load.

The shearing rate for unsaturated soil has not been fully investigated and several rates exist in the literature. Fredlund et al. (1997) reported that the permeability of the soil plays a big role in the determination of the shearing rate for unsaturated soils. Therefore, the shearing rate should be appropriately determined depending on the soil permeability. For this thesis, the shearing rate for unsaturated soils was determined experimentally by running a series of single-stage direct shear tests for rates between 0.01 mm/min to 1.25 mm/min. An initial net normal stress of 320 kPa was maintained on a specimen compacted wet of optimum (Dry density = 1.515 Mg/m^3 , and water content = 26%) and five shear rates were used. This was chosen because it is assumed that the highest variation in suction occurs at higher net normal stress and close to full saturation. The test results are shown in Figure 3-24, and it can be observed that there is a small variation in the shear stress peaks for shearing rates between 0.01 – 1 mm/min.

Therefore, a faster shearing rate for the unsaturated soil specimen was set at 1 mm/min to minimise changes in water content during the shearing. The above procedures were repeated for different normal loads of 125N, 250N, 500N and 1000N.

3.6.3: Area Correction

Since the area of the shear box is decreasing gradually during the shearing, the normal stress and shear stress will be affected and hence the change in the area needs to be considered. For a circular specimen (illustrated in Figure 3-25), the original area A_0 is given by Equation 3-5.

$$A_0 = \frac{\pi D^2}{4} \quad (3-5)$$

where D is the diameter of the soil specimen, hence the correction factor F is given by:

$$F = \frac{2}{\pi} \left\{ \cos^{-1} \left(\frac{\delta}{D} \right) - \left(\frac{\delta}{D} \right) \sqrt{1 - \left(\frac{\delta}{D} \right)^2} \right\} \quad (3-6)$$

where; δ = horizontal displacement and the arccosine in radians.

Figure 3-24: Shear displacement and shear stress at different shearing rates

Figure 3-25: Contact Area variation illustration during shearing of soil specimens

3.7: Brazilian tensile test

As earlier mentioned, the tensile strength of the soil is commonly determined in the laboratory by direct and indirect test methods. Although the indirect tensile test has been criticized as being less

reliable, research has shown that the indirect tensile tests have advantages over the direct tensile test. In this study, the Brazilian tensile strength (BTS) test was adopted.

3.7.1: Apparatus

An unconfined compression machine was fitted with a pair of loading platens designed to have a contact to a disk-shaped specimen at diametrically opposite surfaces. The compression machine is fitted with a load cell of 10kN, (shown in Figure 3-26a), the test setup as in Figure 3-26b and the loading platens are held in place by use of magnets embedded into the base of the bearing block (Figure 3-26c).

3.7.2: Test procedures

In the first test series, the compacted specimens were tested immediately after compaction to obtain the as-compacted strength. The compacted cylindrical disk specimens of 20 mm thick and 63.5 mm diameter specimens were prepared using a specially designed acrylic mould as described in section 3.3.1. Different water contents of 14, 18, 22 and 26% were used on test series 1 while test series 2 specimens were prepared at the wet of optimum water content (26%) and dried to water contents of 22, 18, 14 and 1.2% (air-dried).

Test series 2 specimens were dried to various water contents in airtight desiccators with saturated sodium chloride (NaCl) solution at the base of the desiccators. The saturated NaCl solution provides a constant relative humidity of about 76% (Young 1967) to dry the test specimens. The mass of the disk specimens was measured regularly and the test specimens that attained the targeted weight (and hence moisture content) were wrapped in cling film and kept in a temperature-controlled humidity chamber to allow for moisture equalization for about 10 days as recommended by Mendes (2011) before testing.

The test specimens for the BTS test in this study were loaded using two specially fabricated curved bearing blocks as recommended in ASTM D3967 – 16 to reduce contact stress. The radius of the contact arc and width of contact with the test specimen were 10° and 30 mm, respectively. The BTS test was conducted at loading rates between 0.1 to 0.5 mm/min and all test specimens failed within 1 to 10 minutes as recommended in ASTM D3967 – 16. The slower rates were used for drier specimens for water contents of 14% and below. The tensile strength σ_t was computed from the BTS test results using Equation 3-7 (ASTM D3967 – 16).

$$\sigma_t = 1.272 \frac{P}{\pi t D} \quad (3-7)$$

where P is the maximum applied load in the BTS test, t is the thickness of the specimen and D is the diameter of the specimen.

(a) unconfined
compression machine

(b) test setup

(c) curved bearing block details

Figure 3-26: Testing machine, setup and loading blocks for the Brazilian tensile strength.

Chapter 4: Soil Suction Measurements and Soil-Water Characteristic Curves

4.1: Introduction

Suction measurement results on both compacted and remoulded soils and the SWCCs of the compacted soils are presented in this chapter. The filter paper (FP) is the main technique adopted for suction measurement, however, other suction measurements such as using the high capacity tensiometer (HCT) and chilled mirror dew-point technique (CMDP) were also conducted. A new approach of using the electrical conductivity with the filter paper is proposed to measure suction of unsaturated soils.

The influences of compaction method on suction and compaction water content on the SWCCs are examined.

4.2: Suction measurements using existing filter paper calibration curves.

The advantages and disadvantages of the FP method for suction measurement have been discussed in Section 2.4.2.1.

In this study, an effort is made to investigate the following factors:

- 1) Suction measurements- matric and total suction.
- 2) Different calibration curves.
- 3) Variability of filter papers.

As already highlighted in Section 3.4.1.1, suction measurements were performed on the as-compacted soils specimens compacted using the standard compaction mould (ASTM D698-12) and ASTM D1557-12) and acrylic mould. Figure 4.1 compares the compaction curves for the soils compacted in the acrylic mould and the standard mould.

Contact and non-contact FPs were used to measure the suctions of the standard and modified Proctor as-compacted soil specimens.

4.2.1: Statically compacted specimens

Table 4-1 summarizes the water contents of contact and non-contact FPs for statically compacted soil specimens. It can be observed from Table 4-1 that the moisture contents for both contact and non-contact filter papers are almost the same and a 1:1 plot of the corresponding filter paper water contents are shown in Figure 4-2. The FPs were tested after an equilibration period of between 14 - 21 days. Whereas the non-contact filter paper measures total suction, the ranges of the moisture contents observed are like those observed for the contact filter paper. Therefore, the non-contact FP was interpreted using both Equations 4.1 and 4.2 for matric and total suction, respectively.

Figure 4-1: Compaction curves for the soil used for suction measurements.

It can be observed that Equation 4.2 gives lower values as compared to Equation 4.1 for the non-contact FP. The big divergence of the suctions between Equations 4.1 and 4.2 for the non-contact FP is attributed to the divergence of Equation 4.2 from Equation 4.1 above water content of 26%. Since the suction inferred from the FP measurements rely on the suction - moisture content relationship, the results suggest the use of a single equation for both matric and total suction. However, the current results are not sufficient to make a conclusive observation and more tests must be conducted to justify the observation. Table 4-2 shows the suction results from the measured water contents reported in Table 4-1.

ASTM 5298-16 recommends the use of three filter papers when setting up the contact filter paper method. An attempt was made in this study to improve the contact and to have a duplicate measurement for the same soil sample. This was done by placing one piece of filter paper in contact with the soil at both ends (top and bottom). An acrylic plate (of diameter bigger than the filter paper diameter) was used to cover the filter paper at each end. A cling wrap was further used to constrain the acrylic plate to ensure that the filter paper was in good contact with the soil specimen (as shown in Figures 3-9). However, it is difficult to quantify the degree of contact. Therefore, non-contact filter paper is a more reliable measurement of total suction.

$$\begin{aligned} \text{Matric suction} \quad & \log \psi = 2.909 - 0.0229w_f; \quad w_f \geq 47\% \\ & \log \psi = 4.945 - 0.0673w_f; \quad w_f < 47\% \end{aligned} \quad (4-1)$$

$$\begin{aligned} \text{Total suction} \quad & \log \psi = 8.778 - 0.222w_f; \quad w_f \geq 26\% \\ & \log \psi = 5.31 - 0.0879w_f; \quad w_f < 26\% \end{aligned} \quad (4-2)$$

Table 4-1: Measured filter paper moisture contents for contact and non-contact filter paper method (static compaction).

Moisture content (%)	Contact condition	Filter paper water content (%)			
		Sample 1	Sample 2	Sample 3	Average
26	C	54.58	53.85	54.35	54.06
	C	53.27	54.33	53.95	
	NC	52.74	51.19	51.95	
24	C	45.67	45.61	45.70	45.66
	C	45.11	46.15	45.73	
	NC	45.15	45.07	44.92	
21	C	39.92	39.72	39.74	39.73
	C	39.14	40.11	39.74	
	NC	39.47	39.22	39.16	
20	C	39.41	38.44	38.25	38.61
	C	39.24	38.18	38.13	
	NC	36.82	37.58	37.31	
18	C	34.57	34.33	34.18	34.28
	C	33.93	34.19	34.46	
	NC	33.46	34.29	34.15	
16	C	31.43	32.15	32.11	31.85
	C	32.13	31.98	31.27	
	NC	31.52	31.47	31.59	
14	C	26.86	26.91	26.88	26.90
	C	26.96	26.88	26.89	
	NC	26.80	26.87	26.63	

C – Contact filter paper, NC – Non-contact filter paper.

Figure 4-2: Comparison of contact and non-contact filter paper water content for statically compacted soil specimens.

Table 4-2: Suction values for average contact and non-contact filter paper moisture contents (static compaction).

Preparation water content (%)	Position	Average filter paper water content (%)	Matric suction (kPa)	Total suction (kPa)
26	C	54.06	46.91	0.002
	NC	51.96	52.40	
24	C	45.66	74.55	0.06
	NC	45.05	81.14	
21	C	39.73	186.97	1.14
	NC	39.28	198.17	
20	C	38.61	222.83	3.25
	NC	37.24	276.90	
18	C	34.28	435.02	17.27
	NC	33.97	463.36	
16	C	31.85	634.52	60.11
	NC	31.53	668.89	
14	C	26.90	1364.32	685.50
	NC	26.77	1377.84	

Figure 4-3 shows a variation of suction with compaction water content for statically compacted specimens. It can be observed from Figure 4-3 that the non-contact FP using Equation 4.1 gives a slightly higher suction as compared to the contact FP. With the increase in suction, the total suction results start tending towards the matric suction indicating that both Equation 4.1 and 4.2 give similar results at higher suctions. Therefore, the results from this study suggest that the use of Equation 4.2 with the non-contact FP is suitable for use in the high suction ranges (i.e., suction >1000 kPa).

Figure 4-3: Variation of suction with compaction water content for statically compacted specimens.

4.2.2: Dynamically compacted specimens

Like for the statically compacted specimens, the contact and non-contact FP moisture content were close for the dynamically compacted specimens. The FP water content for the contact and non-contact FPs are illustrated in Figure 4-4. The suctions corresponding to the water contents are presented in Figure 4-5. A similar trend on the variation of the non-contact FP suctions is observed with the dynamically compacted soil specimens. It can be observed that Equations 4.1 and 4.2 give similar suctions above 1000 kPa supporting the hypothesis that the non-contact FP is suitable for suctions higher than 1000 kPa. Tables for the water contents and suctions for the dynamically compacted (standard and modified Proctor) specimens are presented in Appendix A.

According to Leong et al. (2002), for the non-contact FP method, there is an equilibrium between the soil and the environment followed by equilibration between the environment with the FP. As a result, it is expected that smaller air space will result in a faster equilibration time. In this study, efforts were

made to reduce the airspace between the FP and the soil by using a mesh to separate the FP and the specimens (Chapter 3, Figure 3-9).

Figure 4-4: Comparison of contact and non-contact filter paper water content for dynamically compacted soil specimens.

Figure 4-5: Variation of suction with compaction water content for dynamically compacted specimens.

The use of the PVC mesh to separate the FP and the soil specimens reduces the issue of contamination and microorganisms attack of the FP. As mentioned in Section 2.2.2.2, ASTM D5298-16 recommends the use of sacrificial FPs to measure matric suction and total suction of the soil.

However, this might necessitate longer equilibrium time and higher suction result for drier soil specimens might be reported (Leong et al., 2002). There was no contamination and microorganisms observed in this study and the use of a sacrificial filter paper may not be required. From this study, 14 – 21 days were sufficient for equilibration for the FPs irrespective of the contact condition. Similar results were observed by Kizza (2019) where he investigated the equilibration times between initially dry and wet FPs.

Figure 4-6 shows a variation of suction with compaction water content for the different compaction methods. It can be observed that for standard Proctor effort, both static and dynamic methods gave similar suction values. This is true because the static compaction was performed under controlled density and water content. The variation in the results for the modified proctor effort is a result of the difficulty in attaining relatively a uniform dry density of the soil specimens from static and dynamic compaction methods. For this reason, it can be noted that the method of compaction does not affect the suction of the soil at an equivalent density and water content.

Figure 4-6: Variation of suction with water content for different compaction methods and efforts.

4.2.3: Variability of filter papers

The variability of Whatman No.42 FPs was investigated to check the difference in characteristics that result in different calibration curves. Several researchers such as Deka et al. (1995) and Marinho and Oliveira (2006) have reported differences in the characteristics of filter paper from different batches. Bulut and Wray (2005) suggested that the absorption characteristics of the Whatman No. 42 filter paper can change when used under pressure.

The new (unused) filter papers from the box weigh between 0.132 to 0.142g with an average of 0.137g. The used filter papers' weights had a bigger variation with the non-contact filter papers ranging between 0.105 to 0.144g with an average of about 0.125g and the contact filter papers ranging between 0.121 to 0.152g with an average of about 0.137g. The measured weights are comparable to the manufacturer's expected dry weight of 0.133g. The possible cause of the variation in the mass of the used filter paper could be a result of the temperature difference as the used filter paper masses were obtained after oven drying.

4.3: Suction measurements using ER.

4.3.1: Water content-electrical resistance relationship

As described in Section 3.4.1.2, three different test set-ups were used. The first test set-up was to determine the relationship between electrical resistivity (ER) and the water content of the filter paper. The second test set-up was to establish the relationship between suction and moisture content of the filter paper as well as to determine the equilibration time. The third test set-up was a prototype set-up to measure suction of soil specimens using ER of the filter paper.

Each filter paper in the study was labelled uniquely. Hence, filter papers with the same label are the same. Figure 4-7 shows the ER and mass of filter paper for seven filter papers as the filter paper dried. The water content-electrical resistance (w-ER) relationship shows a similar trend for each filter paper but there is data scatter between filter papers. The scatter observed is due to a variation in the test protocol where some readings were recorded after the application of a voltage for more than 3 seconds, the accuracy of the resistance reading and a slight microstructural difference in the filter paper.

To re-check the data scatter, two filter papers FP_8 and FP_9 were subjected to three cycles of wetting and drying following a stricter test protocol. After the first drying cycle, the filter papers were rewetted to the water content of 65-75% and measurements were again conducted as the filter paper dried. At the end of the third cycle, the final water content of the filter paper and its dry mass was determined and the water contents of the filter paper during the three cycles were back-calculated. The test results of FP_8 and FP_9 are shown in Figure 4-8. Figure 4-8 shows a similar trend to Figure 4-7 except that the variation of ER with water content for the filter papers becomes smaller. Hence, the test protocol is important in reducing differences between filter papers.

Figure 4-7: ER and water content of filter papers

Figure 4-8: ER and water content of filter papers subjected to 3 cycles.

4.3.2: Calibration curve

To determine the water content-suction relationship of the filter paper, two filter papers (FP_10 and FP_11) were initially wetted to 65-75% water content and allowed to equilibrate with different concentration salt solutions in test set-up 2 starting with the lowest concentration salt solution. The

filter paper was weighed regularly. Figure 4-9 shows a typical variation of mass of the filter paper with time for applied suction of 100 and 5000 kPa, respectively.

Figure 4-9: Relationship between ER and suction

Table 4-3 summarises the equilibrium times of the filter papers with different applied suction. The equilibrium time of the filter paper is dependent on suction source, contact condition, and suction level (Leong et al., 2002). ASTM D5298 – 16 recommends a minimum of seven days but Leong et al. (2016) used 21 days for initially wet filter papers. Marinho and Oliveira (2006) suggested equilibrium times for different suction, as shown in Table 4-3. For suction greater than 1000 kPa, the experiments were conducted for 8 days to ensure that no further change in the mass of the filter paper occurred after 6 days.

Table 4-3: Equilibration time for initially wet filter paper

Applied suction (kPa)	Estimated equilibration time (Days)	Suggested equilibration time Marinho and Oliveira (2006) (Days)
100	15 - 21	More than 30
300	10 - 15	15
500	10 - 12	15
1000	4 - 6	7
3000	2 - 4	7
5000	2 - 4	7

The results of FP_10 and FP_11 are plotted with the calibration curves from Leong et al. (2002) and ASTM 5298-16 and total suction calibration data using salt solutions from the literature (Leong et al., 2002; Marinho and Oliveira, 2006; Agus et al., 2010; Haghighi et al., 2012; Zhu et al., 2016; Durukan and Akinci, 2017) in Figure 4-10.

Figure 4-10: Comparison of data from filter papers FP_10 and FP_11 with calibration curves and data from others.

The FP_10 and FP_11 data plots are slightly above the calibration data for wetting filter paper from the literature for suctions between 100 and 1000 kPa. All the data tends to converge as suction increases beyond 1000 kPa. The filter papers, FP_10 and FP_11, underwent drying from the initially wet condition (between 65 and 75%) in this study whereas the calibration curves in Leong et al. (2002) and ASTM D5298-16, and the calibration data from the literature were from filter papers that underwent wetting from an initially dry condition.

The filter papers, FP_10 and FP_11 reached an equilibrium water content unattainable for an initially dry filter paper in the non-contact mode for suctions less than about 1000 kPa within a practical equilibrium time (< 21 days) according to the total suction calibration curve in Leong et al. (2002). Likos and Lu (2002) and Marinho and Oliveira (2006) recommended that each batch of filter paper should be calibrated before use due to possible differences from batch to batch. Hence, a calibration curve using the results of FP_10 and FP_11 was obtained for suctions greater than 100 kPa (w less

than 47%) in this study represented by Equation 4-3 and plotted in Figure 4-10. As the suction calibration range was limited, Equation 4-3 is restricted to suctions from 100 to 10000 kPa.

$$\log s = 4.722 - 0.0578w \quad \text{for } w < 47\% \quad (4-3)$$

where w – water content.

Using Equation 4-3, suctions were estimated for the data shown in Figure 4-8 (FP_8 and FP_9) and plotted in Figure 4-11. Figure 4-11 shows that the sensitivity of ER can be divided into two ranges (breakpoint at about 1600 kPa) and each can be approximated using a power-law function as shown in Figure 4-11.

Figure 4-11: Electrical resistance – suction relationship for filter paper

4.3.3: Suction measurement using ER of filter paper.

Test set-up 3 is a prototype set-up to measure ER of a filter paper as it equilibrates with the suction source without handling the filter paper. The equilibrium times in Table 2 were used as a guide. Different suctions ranging from 100 kPa to 5000 kPa was applied. The test was conducted for two filter papers (FP_12 and FP_13). Table 3 compares the applied suction and the estimated suctions using the equations in Figure 10. Suction was estimated to be within an error of -17.3 to 15.6% for the low suction range (< 1600 kPa) and less than 9.6% for the high suction range (≥ 1600 kPa). The

errors are comparable with those reported by Leong et al. (2002) using water contents of the filter papers (errors of -12 to +18% for suctions of 200 to 1400 kPa).

Table 4-4: Comparison of estimated suction versus applied suction.

Filter paper	Applied suction (kPa)	Measured ER (M Ω)	Estimated suction (kPa)	Equilibrium time (Days)	Error (%)
FP_12	100	2.212	116	21	15.6
	300	2.933	343	12	14.2
	500	3.213	487	8	-2.7
	1000	3.687	827	6	-17.3
	3000	13.47	3288	3	9.6
	5000	27.85	5234	3	4.7
FP_13	100	2.132	100	21	0.3
	300	2.823	296	12	-1.4
	500	3.113	431	8	-13.8
	1000	3.697	835	6	-16.5
	3000	13.27	3257	3	8.6
	5000	28.95	5366	3	7.3

4.3.4: Concluding remarks

The filter paper method is simple to use but the measurement accuracy is dependent on the calibration curve of the filter paper, and thus the accurate determination of the water content of the filter paper is very important. The determination of the water content of the filter paper requires extra precautions. Hence to overcome the limitations of measuring the water content of the filter paper, this study examines the feasibility of using electrical resistivity (ER) of the filter paper to infer the suction. For certain electrode arrangements and filter paper, the ER can be represented by resistance. The suction-resistance relationship of the filter paper is affected by the calibration protocol and strict adherence is required to obtain a reliable suction-resistance relationship. The suction-resistance calibration equation can be used for the filter papers in the same batch. A prototype set-up was designed to demonstrate how soil suction measurement can be performed using the non-contact filter paper method with electrical resistivity. It was demonstrated that the electrical resistance of the filter paper can be used to provide a reasonable estimate of total suction greater than 100 kPa with an error between -18 and +10%. The error can be further reduced if each filter paper is calibrated before use. The proposed method can be used to measure suction of a soil specimen or a soil specimen undergoing

drying to determine the soil water retention curve. The study has shown that the equilibrium times of the filter papers are within the ranges reported by other researchers which provide a guide for the use of initially wet filter papers.

4.4: Soil-water characteristic curves

Drying SWCCs have been obtained in this study to investigate the influence of the initial compaction water content and dry density on the SWCC. The characteristics of soil specimens compacted in the acrylic mould are shown in Table 4-6. The last column in Table 4-6 is for comparison with the standard Proctor compaction curve (ASTM D698-12) but all the specimens were compacted using acrylic moulds. Two dynamically compacted specimens at water contents of 14% and 26% representing dry and wet of optimum, respectively, were used to compare their SWCCs with the statically compacted specimens.

In this study, w-SWCC and S-SWCC representing SWCC in terms of gravimetric water content (w) and degree of saturation (S), respectively, are used.

Table 4-5: Soil properties for soil specimens used for SWCC.

BT- soil			Dry densities, Mg/m ³	
Specimen	Compacted at	Water content (%)	Acrylic mould	Standard mould
STC14	dry of optimum	14.1	1.51	1.52
STC16	dry of optimum	16.0	1.56	1.57
STC18	dry of optimum	17.9	1.59	1.60
STC22	optimum	21.5	1.61	1.62
STC24	wet of optimum	23.9	1.57	1.58
STC26	wet of optimum	26.1	1.51	1.52
DNC14	dry of optimum	14.0	1.51	1.52
DNC26	wet of optimum	26.0	1.52	1.52

STC – Static compaction, DNC – Dynamic compaction

4.4.1: w-SWCC

The w-SWCC is obtained using mass properties which makes it simpler to estimate and more dependable compared to θ_w -SWCC and S-SWCC that depend on volumetric properties. This form of

SWCC is referred to as the primary SWCC where the θ_w -SWCC and S-SWCC can be obtained. Volumetric properties of soil for example void ratio are problematic to be precisely determined.

Figures 4.13 and 4.14 show the w-SWCCs of BT soil specimens statically and dynamically compacted using standard Proctor effort at different initial water contents, respectively.

The density can be modelled in terms of either void ratio e , saturated water content w_{sat} , or relative density (Wijaya and Leong 2017). Therefore, in Figures 4.13 and 4.14, the effect of density is represented by the saturated water content, w_{sat} . The difference in w_{sat} affects the initial portion of the w-SWCC. It can be observed that the SWCCs show a band of SWCCs and do not merge at high suctions indicating the slight differences in the soil structure. It can be observed that for specimens compacted near or below the optimum moisture content (dry of optimum), lower suctions are observed as compared to those compacted above the optimum moisture content. The SWCCs for two compaction water contents of 14 and 26% representing dry and wet of optimum, respectively, for statically and dynamically compacted specimens are compared in Figure 4.14.

Figure 4-12: w-SWCC for statically compacted specimens.

From Figure 4.14, it can be observed that specimens statically and dynamically compacted wet of optimum show a similar w-SWCC compared to specimens compacted dry of optimum. The possible explanation is that both statically and dynamically compacted specimens wet of optimum has the same soil structure. The soil structure of soil can be reflected by the slope of the SWCC-w. Similar observations have been made by Vanapalli et al. (1999) who reported that a dispersed uniform

structure is observed from samples compacted wet of optimum and their SWCCs are normally controlled by the microstructure. A dispersed soil structure is associated with a more uniform and disconnected pore network that usually facilitates a gradual desaturation process. This gradual desaturation process will result in a gentler SWCC slope.

Figure 4-13: w-SWCC for dynamically compacted specimens

Figure 4-14: Comparison between dynamic and static compaction method

Furthermore, it can be observed from Figure 4-14 that at the same dry density, specimens compacted dry of optimum shows a lower w-SWCC compared to those specimens compacted wet of optimum. This can be attributed to the difference in soil structure between wet and dry of optimum.

4.4.2: S-SWCC

Fredlund et al. (2012) recommended that S-SWCC should be used to determine the air-entry value (AEV) of a drying soil that experiences volume changes. As earlier mentioned, the S-SWCC requires volume change measurements to be conducted. Hence, S-SWCC can be obtained from w-SWCC and a shrinkage curve. The shrinkage curve is a relationship between void ratio and gravimetric water content that represents the volume change of the soil as it dries.

In this study, volume measurements of the soil specimens were obtained during the SWCC tests to determine their shrinkage curves. Figures 4-15 shows typical shrinkage curves for dynamically and statically compacted BT soils dry of optimum ($w = 14\%$). Figure 4-16 shows typical shrinkage curves for soils compacted wet of optimum ($w = 26\%$). The data were fitted using a universal shrinkage curve equation proposed by Leong and Wijaya (2015).

It is observed that statically compacted specimens show a smaller $e_{\min}$ compared to dynamically compacted specimens with the same initial dry density and moisture content.

Figure 4-15: Shrinkage curve for specimens on dry of optimum

Figures 4-17 and 4-18 show typical S-SWCC for statically and dynamically compacted soil specimens, respectively.

(a) Dynamic compaction

(b) Static compaction

Figure 4-16: Shrinkage curve for specimens on wet of optimum

Fredlund and Xing (1994) equation was used to curve-fit the S-SWCC data in Figures 4-17 and 4-18. From Figures 4-17 and 4-18, it can be observed that the AEVs increase with compaction water contents with lower AEVs being recorded from specimens compacted at the dry of optimum specimens, followed by the specimens compacted at optimum water content and lastly the wet of optimum.

Table 4-6: AEV values and other SWCC variables for different compaction water content

Compaction Water content (%)	a (kPa)	n	m	AEV (kPa)
14	1381.9	0.68	3.63	48
16	1390.9	0.66	3.47	56
18	1403.0	0.61	3.26	62
22	1409.4	0.71	3.02	70
24	1398.5	0.70	2.93	85
26	1403.5	0.86	3.24	100

This can be attributed to the large voids between the clods which represent the macrostructure. According to Marinho (2005), the initial de-saturation process for the specimens compacted dry of optimum is controlled by the macrostructure which aids de-saturation at low suctions. Furthermore, Vanapalli et al. (1999) reported that specimens compacted at the dry of optimum possess less de-saturation resistance. In contrast, greater resistance to de-saturation is characteristic of specimens compacted wet of optimum because of their dispersed structure in nature. This enhances their water storage capacity and as a result, the de-saturation process requires high suctions to be initiated

(Vanapalli et al., 1999).

The slope of the S-SWCC for specimens compacted wet of optimum is higher than the S-SWCC for specimens compacted at the optimum as well as the dry of optimum moisture contents. The S-SWCC slope reflects the pore size distribution of the specimen and a steeper S-SWCC slope implies that specimens compacted wet of optimum possess a uniform pore size distribution.

Figure 4-17: S-SWCC for statically compacted soil specimens corresponding to the standard Proctor curve.

Figure 4-18: S-SWCC for soil dynamically compacted specimens corresponding to the standard Proctor effort.

Pore size distribution curves are more relevant to interpret water retention phenomena, and it is probable that, with 23% clay content, the wet BT samples have a clay matrix microstructure, with matrix pores (and their changes) controlling the shape of the SWCC. Similarly, the difference in

microstructure between the wet and the dry side (mentioned in p.129) has been well documented in the literature, with an aggregate microstructure on the dry side, characterised by larger and less deformable inter-aggregate pores that facilitate desaturation compared to the wet side

4.5: Concluding remarks

This chapter investigated suction measurements using the existing filter paper calibration equations (contact and non-contact) for statically and dynamically compacted soil specimens. A new method of using the ER of filter paper to measure suction in the non-contact mode was proposed. The influences of the compaction method on suction and compaction water content on the SWCCs were also examined. The main findings in this chapter are summarized below:

1. Both contact and non-contact filter papers gave similar water contents. This may not be true if the sample shows any osmotic suction. Reduced air space in the non-contact filter paper method facilitates faster equilibration between the filter paper and the air space within a sealed environment. However, results from this study suggest that the non-contact filter paper is appropriate for suctions higher than 1000 kPa.
2. The compaction method does not affect the soil suction if the dry density and water content is similar. Therefore, static and dynamic compacted soils of the same dry density and water content yield similar soil suction.
3. The higher the compactive effort, the lower the void ratio and as a result, at similar water contents, the modified Proctor compacted soils result in lower matric suctions as compared to the standard Proctor compacted soils.
4. For all the contact filter papers used, no contamination was observed therefore, the recommendation by ASTM D5298-16 to use a sacrificial filter paper may not be needed.
5. Generally, suction increases with a reduction in the moisture content irrespective of the method of compaction or the applied compaction effort.
6. The AEVs increase with compaction water contents with lower AEVs being recorded for specimens compacted at the dry of optimum specimens, followed by the specimens compacted at optimum water content and lastly the wet of optimum.
7. The desaturation rate for specimens compacted wet of optimum is higher than the specimens compacted at the optimum as well as the dry of optimum moisture contents.
8. It has been demonstrated that the ER of the filter paper can be used for suction measurement. Additionally, a single filter paper can be used multiple times in the proposed ER method to measure soil suction.

Chapter 5: Osmotic technique and osmotic consolidation

5.1: Introduction

The experimental study of unsaturated and expansive soils is more difficult than saturated soils because it is hard to measure soil suction. Soil suction is required for describing the mechanical properties of unsaturated soils. Matric and osmotic suctions are the two main components of soil suction (Krahn and Fredlund 1972, Fredlund et al. 2012).

Several devices (discussed in section 2.4.2) have been employed to measure suction. However, these devices have a limited range of suction measurement, latency and stability issues during measurement (Leong et al. 2007). As a result, most researchers have adopted the use of suction control methods to characterize unsaturated soils. Suction control methods have been discussed in section 2.4.3 of this thesis.

In this chapter, results from the osmotic technique used to control matric suction is presented together with osmotic consolidation. The latter part presents the volume change of soil that occurs due to a difference in salt concentration in the pore water between different zones of the soil referred to as osmotic consolidation.

5.2: Suction Control Using Osmotic Technique

As discussed in section 2.4.3, suction control methods have different advantages and disadvantages. Among the methods, the osmotic technique has added advantages as described in section 2.4.3.3. Despite the advantages, the osmotic technique has some shortcomings such as the degradation of the membrane (cellulotic) and molecular weight break down of the commonly used PEG solution during long-duration tests.

This section investigates the application of the osmotic technique using sodium chloride solutions and reverse osmosis (RO) membrane in a conventional oedometer. As described in section 3.6.1.4, the soil specimen was obtained from the soil blocks prepared earlier using the oedometer ring and was placed in contact with the RO membrane on its bottom. The soil specimen was then inundated in water and consolidated to the specific effective vertical stress of 64 kPa. At equilibrium, the water in the oedometer cell was removed and the NaCl solution contained in the reservoir was circulated through the porous stone under a low-pressure head and returned to the reservoir via a peristaltic pump (Figure 3-21). The tests were performed in an environment where the ambient temperature range is between $24\pm 4^{\circ}\text{C}$. The test procedures were repeated with identical specimens but with a

different concentration of NaCl solution. The updated osmotic oedometer is attractive as it eliminates the drawbacks of the previous osmotic oedometer using PEG solution and a cellulosic membrane.

5.2.1: NaCl and RO membrane performance

To assess the performance of the NaCl solution and the RO membrane in the osmotic technique, a check was conducted using the electrical conductivity measurements of the pore water of the soil specimen and the salt solutions. The measurement procedure as described in section 3.4.4 remains the same throughout the thesis whenever osmotic suction is deduced from the electrical conductivity of soil specimens and salt solutions.

Table 5-1 summarizes the measured osmotic suctions computed using Equation 2-7 from electrical conductivity measurements of the NaCl solution in the reservoir before and at the end of the test (See Figure 3-21).

Table 5-1: Concentrations and measured osmotic suctions of NaCl solutions before and after the test.

NaCl solution concentration (g/litre of water)	Target osmotic suction (kPa)	Measured osmotic suction before test (kPa)	Measured osmotic suction after test (kPa)	Difference in measured osmotic suction (%)
6.17	500	496	502	+1.15
12.62	1000	991	996	+0.51
25.01	2000	1934	1938	+0.21
62.51	5000	5032	5034	+0.04
123.457	10000	9960	9963	+0.03

The electrical conductivity measurements were made using the Horiba ES-12 electrical conductivity meter. The variation in osmotic suctions ranged from +1.15% to +0.3 % for the lowest to highest suctions, respectively. This is attributed to experimental error due to the evaporation effects of the NaCl solution in the reservoir over the test duration. Some of the test durations were as long as three months. However, the changes in osmotic suction of the NaCl solutions at the end of the experiment were negligible.

Table 5-2 presents a summary of the electrical conductivity results of the pore water extracted from the soil specimens before and after the test using the Horiba B-173 electrical conductivity meter and the osmotic suction calculated using Equation 3-2. The osmotic suction of the soil specimen before the test is negligible less than 6.1 kPa. There is a negligible difference in the electrical conductivity values of the pore-water before and after the test. The change in osmotic suction of the soil specimens

before and after the test ranges from about 0.3 to 16 kPa or 0.03% to 0.16% of the applied osmotic suction, which is negligible, indicating that the RO membrane has a good salt rejection rate.

Table 5-2: Electrical conductivity of pore-water measured from soil specimens before and after the test.

Applied osmotic suction (kPa)	Electrical conductivity before the test (mS/cm)	Initial osmotic suction (kPa)	Electrical conductivity after the test (mS/cm)	Final osmotic suction (kPa)
500	0.21	5.2	0.23	5.8
1000	0.23	5.8	0.24	6.1
2000	0.21	5.2	0.31	7.9
5000	0.24	6.1	0.41	11.3
10000	0.23	5.8	0.72	21.5

5.2.2: Deformation

The vertical deformation was used to determine the suction equilibrium of the soil specimen in the osmotic oedometer. At equilibrium, there is negligible water flow from the soil specimen through the RO membrane and the experiment was stopped. For this study, the experiment was stopped when the registered deformation was less than 0.01 mm over six hours.

Figure 5-1(a) illustrates the vertical deformations of the soil specimens subjected to different osmotic suctions at the base. The figures represent deformations starting from the point of application of the osmotic suction (the deformation under the vertical stress before application of the salt solution is not presented on the graph). For all specimens tested, the deformation under the vertical stress of 64 kPa before application of the osmotic suction was between 0.31mm and 0.34 mm. Figure 5-1(a) shows that the equilibration time increases with an increase in applied osmotic suction.

It can be observed that the deformation shows two segments, the first segment starts on the application of the osmotic suction and the second segment starts after a few days of application of osmotic suction. The lag time to the start of the second segment varies with the applied osmotic suction and it can be observed from Figure 5-1(a) that the longest lag time is at the lowest applied osmotic suction of 500 kPa. The possible explanation for the small decrease before levelling-off in the first segment is attributed to the process of siphoning out of the water from the oedometer cell that causes a delayed response to the applied osmotic suction.

(a) Inundation of water at the beginning of consolidation test and under various applied osmotic suctions

(b) No inundation of water at the beginning of consolidation test and under 5000 kPa applied osmotic suction

Figure 5-1: Time-dependent volume change of the specimens with different applied osmotic suctions at the base.

To check the hypothesis, an identical experiment was conducted where the soil specimen was not inundated with water in the oedometer cell at the beginning of the consolidation test and the osmotic suction of 5000 kPa was applied after the initial consolidation to 64 kPa. Results illustrated in Figure 5-1(b) show that there is only one segment that corresponds to the second segment of Figure 5-1(a). Larger deformations were recorded as the applied osmotic suction increases. This is consistent with

the results reported by other researchers (e.g., Delage et al. 1992). The deformations due to the application of the various osmotic suction values shown in Figure 5-1(a) are summarized in Table 5-3 and Figure 5-2. Figure 5-2 shows that the deformation increases at a decreasing rate as the applied osmotic suction increases.

Table 5-3. Deformations due to applied osmotic suction.

Osmotic suction (kPa)	Observed deformation (mm)
500	1.45
1000	2.42
2000	3.77
5000	5.99
10000	7.40

Figure 5-2. Total deformation against applied osmotic suction.

5.2.3: Matric suction

The matric suctions of the soil specimen on the top and bottom faces were measured using a high capacity tensiometer (HCT) at the end of the test. Typical responses of the HCT are shown in Figures 6-3 and 6-4 for soil specimens subjected to applied osmotic suctions of 2000 and 5000 kPa, respectively. It can be observed that the matric suction recorded at the bottom faces of the specimens are lower than the matric suction recorded at the top faces showing that there is a pore-water pressure

gradient within the soil specimen and the pore water was subjected to one-dimensional flow through the bottom of the specimen. Hence, the osmotic technique imposes a matric suction on the bottom of the soil specimen causing it to consolidate further.

Figure 5-3. Responses of the HCT to soil specimens under applied osmotic suction of 2000 kPa.

Figure 5-4. Responses of the HCT to soil specimens under applied osmotic suction of 5000 kPa.

After measuring the matric suction immediately after the test, the soil specimens were wrapped with two layers of cling wrap and one layer of aluminium foil and kept in an air-tight zip lock bag in a humidity cupboard for 24 hours for moisture equilibration. Thereafter the matric suctions were again measured for both the top and bottom faces of the soil specimens. The matric suction readings are summarised in Table 5-4.

The matric suctions at the top and bottom faces are closer in value and the average matric suction is almost equal to the average matric suction of the soil specimen measured immediately after the test. The difference in average matric suctions immediately after the test and one day after the test becomes smaller as the applied osmotic suction increases. The matric suction induced by the osmotic technique is the measured matric suction less the measured matric suction of the control test specimen where water instead of NaCl solution was circulated at the base of the oedometer.

Table 5-4. Matric suctions measured on top and bottom faces of specimen immediately after the test and after one-day moisture equilibration.

Osmotic suction (kPa)	Measured matric suction (kPa)					
	Immediately after the test			1 day after the test		
	Top	Bottom	Average	Top	Bottom	Average
Control	51.6	33.9	42.8	44.6	40.1	42.4
500	185.3	102.3	143.8	107.0	104.2	105.6
1000	203.2	162.0	182.6	152.0	150.3	151.2
2000	337.7	231.3	284.5	291.0	231.0	261.0
5000	782.6	723.5	753.1	738.2	732.1	735.2
10000	1422.1	1201.3	1311.7	1318.2*	1305.2	1305.2

* - Final reading recorded before the HCT cavitated. Hence this value was ignored in averaging.

The corrected average matric suctions due solely to osmotic technique are summarized in Table 5-5. Nevertheless, the measured matric suction of the soil specimen at the end of the test is much less than the applied osmotic suction. Similar results have been reported by researchers using PEG solutions such as (Peck and Rabbidge 1969; Delage and Cui 2008; Slatter et al. 2000; Zur 1966). The difference in measured matric suction and applied osmotic suction has been reported to be about 20% (Dineen and Burland 1995).

Figure 5-5 shows the variation of corrected average matric suction measured after 1-day moisture equilibration with the applied osmotic suction. According to Figure 5-5, the measured matric suction is about 13% of the applied osmotic suction. The osmotic suction of a saturated NaCl solution was measured to be 38803 kPa using the WP4 compared to a theoretical value of 38790 to 38972 kPa based on Equation 2-7.

Table 5-5. Corrected average measured matric suctions immediately after the test and after one-day moisture equilibration.

Osmotic suction (kPa)	Corrected measured matric suction (kPa)			
	Immediately after the test		1 day after the test	
	Corrected average	Reduction from applied osmotic suction (%)	Corrected average	Reduction from applied osmotic suction (%)
500	101	79.8	63.2	87.4
1000	139.8	86.0	108.8	89.1
2000	241.7	87.9	218.6	89.1
5000	710.3	85.8	692.6	86.1
10000	1268.9	87.3	1262.8	87.4

According to the relationship in Figure 5-5, the expected matric suction using a saturated NaCl solution in the osmotic oedometer is about 13% of 38803 kPa, that is, 5000 kPa. The lower matric suction is attributed to the membrane resistance effect (Slatter et al. 2000, Tarantino and Mongiovi 2000, Monroy et al. 2007). The membrane resistance effect is a complicated phenomenon and has been a topic of intense research in desalination. It has been attributed to the concentration polarization phenomenon in membrane processes (Tang et al. 2010, Klaysom et al. 2013, Feher 2017).

Figure 5-5. Relationship of measured matric suction and applied osmotic suction.

The water flux Q_v through a membrane is driven by a pressure difference across the membrane and can be written as (Henkens et al. 1979, Kim and Hoek 2005, Feher 2017, Dominijanni et al. 2018).

$$Q_v = L_p (\Delta p - \Delta \pi) \quad (5-1)$$

where L_p is the hydraulic permeability, Δp is the hydrostatic pressure, and $\Delta \pi$ is the osmotic pressure. However, Equation 5-1 is modified due to the concentration polarization phenomenon to

$$Q_v = L_p (\Delta p - \Delta \pi_{\text{eff}}) \quad (5-2)$$

where $\Delta \pi_{\text{eff}}$ is the effective osmotic pressure. The ratio of $\Delta \pi_{\text{eff}}$ and $\Delta \pi$ is known as the reflection coefficient ω , i.e.,

$$\omega = \frac{\Delta \pi_{\text{eff}}}{\Delta \pi} \quad (5-3)$$

The symbol ω for reflection coefficient follows that used by Dominijanni et al. (2018). In the osmotic oedometer, Δp is given by the matric suction in the soil specimen and $\Delta \pi$ is given by the osmotic suction of the NaCl solution. At equilibrium, no flow condition prevails, and the matric suction of the specimen is given by $\Delta \pi_{\text{eff}}$.

Figure 5-6 shows the variation of the reflection coefficients from Peck and Rabbidge (1969) ranging from 0.54 to 0.73, Dineen and Burland (1995) ranging from 0.77 to 0.83, Slatter et al (2000) ranging from 0.6 to 1, and Tarantino and Mongiovi (2000) ranging from 0.9 to 0.95 and Yuan et al. (2017) ranging from 0.85 to 1.0 for different concentrations of PEG solutions and cellulosic membranes. This indicates that different osmotic technique setups have different reflection coefficients and as a result, there is a need for independent suction measurements of the soil specimen for verification at the end of the test. For the osmotic oedometer used in this study, ω ranges between 0.11 to 0.14, with an average value of 0.13. The effect is attributed to the possibility that the membrane resistance effect in the RO membrane is more than the cellulosic membrane or PES membrane due to its three-layered structure. Further experimental evidence of ω is obtained from the soil-water characteristic curve (SWCC) of the soil used in this study.

Figure 5-7 shows the SWCC of the soil obtained from drying a soil specimen. The matric suction of the soil specimen as it dries was measured using the HCT for suction below 1500 kPa. Above 1000 kPa, the suction of the soil specimen as it dries was measured using WP4. The gravimetric water content and the matric suction of the soil specimens from the osmotic oedometer tests are also plotted in Figure 5-7, showing good agreement with the SWCC.

Figure 5-6: Reflection coefficients from others.

Figure 5-7: Drying SWCC for KB30 from HCT and WP4 together with the water content and matric suction of the soil specimens from the osmotic oedometer test.

5.2.4: Summary

In this section, the application of the osmotic technique using sodium chloride solutions and a reverse osmosis (RO) membrane in a conventional oedometer has been investigated. The following conclusions can be deduced:

- 1) Independent matric suction measurement of the soil specimens conducted after the test using a high capacity tensiometer revealed that the matric suction is about 13% of the applied osmotic suction. The reduction is attributed to the membrane resistance effect and explained using the phenomenon of concentration polarization in membrane processes.
- 2) It is expected that the matric suction as a function of the applied osmotic suction will be different for different combinations of membrane and solution. Hence, a calibration should be performed to determine the achievable matric suction when applying the osmotic technique.
- 3) The use of sodium chloride solutions and a reverse osmosis (RO) membrane in a conventional oedometer (updated osmotic oedometer) is attractive as it eliminates the drawbacks of previous osmotic oedometer using PEG solution and a cellulosic membrane.
- 4) In this study, the setup has only been verified for an induced matric suction of about 1300 kPa using a sodium chloride solution with osmotic suction of 10000 kPa. For a saturated sodium chloride solution, the osmotic oedometer can theoretically induce a matric suction of up to 5000 kPa on the soil specimen.
- 5) Volume or mass measurement device can be added to the setup for the determination of the soil-water characteristic curve of the soil specimen under constant normal stress by changing the concentration of the salt solution after each equilibrium point.
- 6) The use of a microfiltration membrane can be investigated in further studies.

5.3: Osmotic Consolidation

5.3.1: Introduction.

In unsaturated soil mechanics, osmotic suction is attributed to the salt concentration of the pore water (Fredlund and Rahardjo, 1993) which affects the soil structure of clayey soils and does not act like matric suction (Leong and Abuel-Naga 2017). Research has shown that the volume change of soil can occur due to a difference in salt concentration in the pore water between different zones of the soil (Barbour 1987, Barbour and Fredlund 1989, Di Maio 1996b, Rao and Thyagaraj 2007, Thyagaraj and Rao 2013). The resulting osmotic gradient causes water to move between zones and this is termed osmotically induced consolidation (Barbour and Fredlund 1989).

The pore water in soil generally contains dissolved salts. Salt transportation into clay soils affects the double diffuse layer. Besides this, Saiyouri et al., (2004) reported another mechanism of discrete adsorption of the layer of water molecules along with the interlayer spaces of the smectite minerals, within the particles. This can result in reduced electrical repulsion forces between the clay platelets leading to a reduction in the void ratio and changes in effective stresses. Experimental results by Di Maio (1996) showed that exposure of specimens of Ponza bentonite to saturated salt solutions

produced ion diffusion into the pore fluid causing large volume decreases and large residual shear strength increases.

A volume reduction in soils translates into a void ratio reduction hence an increase in the effective stress. Rao and Thyagaraj (2007) showed that an additional stress component is created due to an increase in osmotic suction in pore water. Barbour et al. (1992) explained that two stress state variables exist in saturated soils: the generally accepted effective stress and a physico-chemical stress state variable representing the net electrostatic repulsive stresses between the soil particles controlled by the osmotic pressure of the pore fluid. Osmotic consolidation occurs as a result of a change in the electrostatic repulsive-minus-attractive (R-A) stresses between clay particles.

However, there are restrictions on the use of this stress state variable (R-A) because it cannot be experimentally quantified in the laboratory and occurs only under ideal conditions. For this reason, there is a need for a more easily measured quantity other than (R-A). In this study, both osmotic and osmotically induced consolidation is not differentiated and only one term osmotic consolidation is used.

Volume change Δv can be caused by a change in mechanical stress, given as

$$\frac{\Delta v}{V_0} = \frac{\Delta e}{(1 + e_0)} = \Delta(\sigma - u_f)m_v \quad (5-4)$$

or, it can be caused by a change in osmotic pressure (Barbour et al., 1992) given as

$$\frac{\Delta v}{V_0} = \frac{\Delta e}{(1 + e_0)} = m_\pi \Delta \pi \quad (5-5)$$

Equating equations (5-4) and (5-5) gives equation 5-6

$$\Delta(\sigma - u_f)m_v = \Delta \pi m_\pi \quad (5-6)$$

where; $\Delta(\sigma - u_f)$ = effective mechanical stress, $\Delta \pi$ = change in osmotic pressure, m_v = coefficient of volume change, m_π = osmotic coefficient of volume change.

Very little work has been done to quantify the mechanical equivalence of osmotic consolidation. In this section, an attempt is made to quantify the mechanical stress equivalence of osmotic consolidation of expansive soils submerged in a salt solution.

A detailed method is covered in section 3.6.2, but a summary is presented here. Conventional oedometer equipment was used to conduct the one-dimensional volume change tests. Both KB30 and KB10 specimens were cut out from the blocks prepared according to section 3.3.2 (See also Figure

3.7) using the oedometer ring. For each kaolin-bentonite mixture (KB30 and KB10), two specimens were prepared. One specimen was consolidated from 32 kPa to 2048 kPa in the oedometer, using a load increment ratio of 1 for successive loads. The other specimen was first consolidated from 32 kPa to 64 kPa. After which, the water in the oedometer cell was replaced with the saturated salt solution and settlement was observed until no further settlement was observed. Mechanical consolidation then resumes, and the specimen was consolidated up till 2048 kPa. This was repeated for each saturated salt solution (Potassium sulphate (K_2SO_4), Potassium chloride (KCl) and NaCl) and each kaolin-bentonite mixture (KB10 and KB30).

5.3.2: Osmotic consolidation

Figures 5-8 to 5-10 show the time-dependent settlement at vertical stress of 64 kPa for soil specimens inundated with Potassium sulphate (K_2SO_4), Potassium chloride (KCl) and Sodium chloride (NaCl) solutions, respectively.

The first portion of the curve is due to mechanical loading up to vertical stress of 64 kPa. At vertical stress of 64 kPa, water in the oedometer cell was drained/removed and replaced with a saturated salt solution. Further settlement was observed. It was observed that the settlement caused by NaCl solution is much more significant than that caused by either K_2SO_4 or KCl solutions. Similar trends were observed with both kaolin-bentonite mixtures but KB10 specimens showed much lesser settlement than KB30 specimens (Table 5-6). This is attributed to the low bentonite content in the KB10 specimens. Table 5-6 shows a summary of the settlements caused by different salt solutions.

Figure 5-8: Sample KB30 – 32– 64kPa effective stress consolidation and osmotic consolidation under K_2SO_4 solution.

Figure 5-9: Sample KB30 – 32–64 kPa effective stress consolidation and osmotic consolidation under KCl solution

Figure 5-10: Sample KB30 – 32–64 kPa effective stress consolidation and osmotic consolidation under NaCl solution

Table 5-6. Settlements due to different saturated salt solutions

Salt solution	Osmotic pressure (kPa)	KB30 settlement (mm)	KB10 settlement (mm)
K ₂ SO ₄	3753	0.38	0.10
KCl	23368	0.83	0.14
NaCl	38803	1.25	0.16

A salt solution with higher osmotic pressure will result in a larger consolidation settlement. Besides that, the sodium cation entering the clay particles reduces the thickness of the double diffuse layer and contributed to the consolidation settlement. Similar observations were made by Nara et al. (2014) who studied crack velocity with the stress intensity under variation of salt concentration. They concluded that electrolyte concentration decreases the thickness of the double diffuse layer as observed by the decrease in crack velocity and increase in stress intensity factor. Furthermore, in the interlayer spaces and regions where the individual montmorillonite stacks are nearby, double diffuse layer overlap will occur and anion exclusion effects will take place (Bradbury and Baeyens, 2002). However, the potassium cation will form a K-linkage between soil particles (Grim 1953), which will reduce the osmotic consolidation effect. Therefore, the osmotic effect caused by potassium cation is not as significant as that caused by sodium cation.

5.3.3: Equivalent mechanical stress

Figures 5-11 and 5-12 show the, $e - \log \sigma'$ plots, for the cases of water, NaCl, KCl and K₂SO₄ of KB30 and KB10, respectively. The dotted line represents mechanical consolidation under, water while the other curves represent the different salt solutions used in the experiments. Since the vertical stresses were kept constant during the exposure to the salt solution, the changes in the void ratio are entirely attributed to the salt effect (osmotic consolidation). From the plots, the equivalent mechanical pressure is computed by drawing a horizontal line at the end of osmotic consolidation on the, $e - \log \sigma'$ plot for each salt solution to the $e - \log \sigma'$ plot for water and determining the change in effective vertical stress σ' as summarized in Table 5-7.

Figures 5-11 and 5-12 show a general trend of reduction in the void ratio at the same vertical stress on the introduction of the salt solution. The behaviour of KB30 and KB10 specimens, when exposed to NaCl solution, is different compared to that of the specimens exposed to KCl and K₂SO₄ solutions. Specimens exposed to KCl and K₂SO₄ solutions under additional vertical stress tend towards the mechanical consolidation line (dotted lines, tested only under, water) whereas those exposed to NaCl solution remains almost parallel. It can be also observed that additional vertical stress after completion of osmotic consolidation resulted in a small decrease in the void ratio. This can be attributed to the

change in the interparticle concentration due to the increased loading since the concentration of the salt solution was not changed.

According to Barbour (1987), the difference in stress from the void ratio after osmotic consolidation, to the stress level along the virgin branch at the same void ratio is taken as the net (R-A). In this study, the difference is considered as a result of osmotic pressure. Specimens exposed to NaCl show the highest equivalent mechanical stress (Table 5-7) because of the highest deformation observed in Table 5-6. This is probably due to ion exchange and it is worth noting that the affinity of montmorillonite for K^+ is higher than Na^+ however ion-exchange reaction depends on ion concentration (Di Maio, 1996).

Figure 5-11: Mechanical and osmotic consolidation (KB30)

Fig. 5-12: Mechanical and osmotic consolidation (KB10)

Table 5-7: Equivalent mechanical stress of salt solutions.

Soil specimen	Saturated salt solution	Effective vertical stress (kPa)	Change in equivalent vertical stress (kPa)
KB30	NaCl	64	41
	KCl	64	25
	K ₂ SO ₄	64	16
KB10	NaCl	64	36
	KCl	64	23
	K ₂ SO ₄	64	15

5.3.4: Osmotic coefficient of volume change

Table 5-8 shows the osmotic coefficient of volume change, m_π computed using Equation 5-6 and the coefficient of volume change (m_v) computed from the mechanical consolidation curve. Figure 5-13 shows a three-dimensional constitutive surface from experimental and theoretical relating void ratio and swelling pressure for different pore fluid concentrations. From Figure 5-13, it can be observed that on the osmotic suction π plane, the slope of the constitutive surface is described by the coefficient of volume change, m_v and on an effective stress $(\sigma - u_f)$ plane, the slope is defined by the osmotic

coefficient of volume change, m_π . It can be observed that a higher coefficient of volume change was registered with specimens of KB30 as compared to specimens of KB10.

Table 5-8. Compressibility properties of the specimens used in this study.

Soil specimen	Saturated salt solution	Coefficient of volume change, m_v (1/kPa)	Osmotic coefficient of volume change, m_π (1/kPa)
KB30	NaCl	0.162E-04	1.707E-06
	KCl	0.162E-04	1.728E-06
	K ₂ SO ₄	0.162E-04	6.886E-06
KB10	NaCl	3.879E-04	0.360E-06
	KCl	3.879E-04	0.382E-06
	K ₂ SO ₄	3.879E-04	1.551E-06

Fig. 5-13: Experimental constitutive surface for Na-montmorillonite during compression (From Barbour 1987).

The coefficients of volume change have been computed for a total stress change of 64 kPa and a saturated salt solution (suction = 38803 kPa) since the matric suction was applied at total stress of 64 kPa as shown below.

Table 5-9: Comparison of coefficients of volume change due to total stress, osmotic suction and Matric suction.

Coefficient of volume change, Total stress, m_v (1/kPa)	1.35E-03
Coefficient of volume Osmotic suction, m_π (1/kPa)	1.71E-06
Coefficient of volume change, Matric suction, m_s (1/kPa)	7.60E-06

5.3.5: Summary

In this section, the effect of salt solution on the volume change behaviour of soil specimens of two kaolin - bentonite mixtures were observed using a conventional oedometer using saturated salt solutions of NaCl, KCl and K_2SO_4 . The following conclusions can be reported:

- 1) Exposing soil specimen to salt solution caused an additional settlement over the consolidation settlement under constant vertical stress but did not affect the soil compressibility.
- 2) The pore-water salt concentration affects the soil structure and induces additional settlement.
- 3) The osmotic coefficient of volume change, (m_π) is only a fraction of the coefficient of volume change, (m_v). Therefore, a large osmotic suction is needed to cause an equivalent settlement under vertical stress.

5.4: Concluding Remarks

In this chapter, two main aspects have been presented. The osmotic technique used to control matric suction together with osmotic consolidation where volume changes occur when salt solution replaces the pore water in a saturated soil specimen.

The main conclusions from this chapter are summarised below:

- 1) The osmotic technique is still valuable in controlling matric suction of soil if a correct combination of the salt solution and membrane is made. Currently, cellulosic membranes and PEG solutions are commonly used but with the recent membrane development technology, other degradation resistant membranes are available and can be used to solve the problem of membrane degradation during long-duration tests. Furthermore, the use of inorganic salts can

be a solution to the problem of solution molecular breakdown reported when organic salts are used.

- 2) Higher matric suctions can be easily achieved when the osmotic technique is used to control suction within reasonably a shorter time, the method is easy and cheap as compared to other methods of matric suction control.
- 3) Soil-water characteristic curve of a soil specimen under constant normal stress can be determined when a volume or mass measurement device is added to the osmotic oedometer setup.
- 4) While both the osmotic technique and osmotic consolidation involve the use of a salt solution, different types of suctions are applied to the specimen.
- 5) It can be observed that a more definite deformation or volume change is observed when a matric suction is applied to a specimen as compared to an osmotic suction. Therefore, pore-water salt concentration may affect the soil structure and induces additional settlement.
- 6) A large osmotic suction is needed to cause an equivalent settlement under vertical stress since the osmotic coefficient of volume change, (m_π) is only a fraction of the coefficient of volume change, (m_v).

Chapter 6: Direct shear tests

6.1: Introduction

As explained in section 2.5 of this study, many field conditions correspond to plane strain conditions such as the stability of embankment, slopes and retaining walls where the deformation is approximately plane strain. Several advantages and disadvantages of the direct shear box have been reviewed in section 2.5 of chapter 2 and some of the outstanding advantages are restated here.

The direct shear box uses a thin specimen sandwiched by two pieces of porous stones thus reducing the drainage path and testing time for soils of low permeability as compared to the triaxial test. Large shear displacements (strain) can be obtained in two directions in the direct shear test and it provides a close reproduction of certain field problems. In addition, correlations and much experience of the direct shear test have been developed for comparison.

The direct shear apparatus has been modified to perform suction-controlled tests referred to as constant suction (CS) tests. These tests are similar to consolidated-drained (CD) tests and have been commonly used to determine the shear strength parameters of unsaturated soils. Researchers have reported the need for expertise to conduct experiments using the modified direct shear apparatus as well as the high costs. Furthermore, the suction range is limited by the high air-entry value disk and the test duration is long.

As discussed in Chapter 2, the constant water content (CW) test addresses some of the concerns of the CD test and more representative of the field stress path than the CD test. Ideally, matric suction should be measured during constant water content test but there are other challenges associated with the procedure as stated in Chapter 2.

6.2: Constant suction direct shear test

The analysis and interpretation of CW direct shear tests can be understood better by examining the analysis and interpretation of CS direct shear tests. This section describes the analysis and interpretation of the CS direct shear tests from the literature. Contradictory observations on the effect of matric suction on friction angle in constant suction direct shear tests were reported. Cokca et al. (2004), Merchan et al. (2008), and Hossain and Yin (2010) found that friction angle increases with increasing matric suction while Ye et al. (2010) and Schnellmann et al. (2013) found friction angle unaffected by matric suction. These observations on change in friction angle in CS direct shear tests indicate either that void ratio changes with net normal stress and hence the friction angle or matric suction is not constant during the CS direct shear test.

Figure 6-1 shows two types of failure envelopes commonly obtained in CS direct shear tests: (i) the failure envelopes are represented by lines of different slopes for different matric suction values (Mohamed et al. 2006, Hossain and Yin 2010b, Likos et al. 2010, Shwan and Smith 2015); and (ii) the failure envelopes are represented by lines of the same slope for different matric suction values (Sun and Xu 2007, Zhou et al. 2016, Gu et al. 2019). It is therefore prudent to review the method of interpretation of the shear strength from CS direct shear tests.

Figure 6-1: Different relationships of net normal stress versus shear strength from constant suction direct shear tests.

Constant-suction direct shear test data from the literature have been collated for this study. The soils tested in (Gu et al. 2019) Zhou et al 2016, Gallage & Uchimura 2016, Schnellmann et al (2013), Hossain and Yin 2010, Mohamed et al (2006), Nishimura et al. 1999 and Han (1997) are summarised in Table 6-2.

The use of stress path in the assessment of the shear strength parameters was reviewed in Chapter 2 (2.4.1.1) and it is further illustrated here that the use of corrected stresses in the stress path improves the accuracy in obtaining the peak shear stress. The stress path approach was used to first determine the effective stress shear strength parameters, c' and ϕ' . A typical illustration of the use of the stress path approach for the interpretation of the saturated tests from Han (1997) is shown in Figure 6-2.

Table 6-1: Basic and other soil properties

Property \ Reference	Hossain and Yin (2010)	Gu et al. (2019)	Zhou et al. (2016)	Gallage and (2016)	Uchimura and (2013)	Schnellmann et al. (2013)	Mohamed et al. (2006),	Han (1997)	Nishimura et al. (1999)
Soil type	Silty sand	Silty clay	Silty sand	Edoaki silty sand	Chiba silty sand	Sand	Sandy silty clay	Clay	Silt
Liquid limit, LL (%)	32.8	28.3	38	NP	25.78	NP	95	47	NP
Plasticity index, PI (%)	10.1	11.1	10	NP	2.26	NP	50	18	NP
Gravel (%)	5.8	0	0	0	0	1.14	1.7	0	0
Sand (%)	44.1	13	100	83.5	64	88.38	47	49	0
Silt (%)	36.8	82	0	14.5	31	8.48	11.3	25	92
Clay (%)	13.3	5	0	2	5	2	40	26	8
D ₅₀ (mm)	0.06	0.02	0.32	0.22	0.14	0.35	0.04	0.04	0.016
D ₆₀ (mm)	0.14	0.028	0.40	0.28	0.18	0.47	0.13	0.20	0.018
Specific gravity, G _s	2.599	2.69	2.68	2.75	2.72	2.72	2.68	2.66	2.65
USCS Classification	SM	MH	SM	SM	SM	SW-SM	MH	CL	ML
Coefficient of curvature, C _c	0.47	1.19	0.90	3.03	1.95	1.34	0.004	0.06	1.12
Coefficient of uniformity, C _u	30.00	9.33	4.44	16.47	54.40	6.71	433.33	285.71	3.00
van Genuchten (1980)									
α (1/kPa)	0.0196	0.1387	0.0052	0.3788	0.1925	0.3316	0.0095	0.0848	0.1176
n	1.2565	1.5991	1.3468	1.4443	1.4800	2.7926	1.0787	1.0631	1.7023

The c' and ϕ' obtained from the stress path approach for all the soils listed in Table 6-1 are summarized in Table 6-2. Most of the soils showed strain-softening behaviour and hence similar values of c' and ϕ' were obtained. Differences in c' and ϕ' were observed with soils where strain-softening behaviour was not observed.

Table 6-2: Effective stress shear strength parameters

Reference	Reported values		Stress path approach	
	c' (kPa)	ϕ' (°)	c' (kPa)	ϕ' (°)
Gu et al. (2019)	0.49	26.1	0.5	26.1
Zhou et al. (2016)	7.74	34.3	7	34.3
Schnellmann et al. (2013)	0	33.6	0	33.5
Han (1997)	25	33.4	13	29.1
Mohamed et al. (2006)	3.5	24.4	3	25.1
Gallage and Uchimura (2016)	0	35.1	0	36.9
Gallage and Uchimura (2016)	0	38.0	0	38.1
Nishimura et al. (1999)	0	27.0	0	26.5
Hossain and Yin (2010)	0	29.9	15	27.1

Figure 6-2 shows a case where the stress path approach provides lower c' and ϕ' as compared to the reported values. This shows that the stress approach is useful in improving the accuracy of estimating the saturated shear strength parameters of c' and ϕ' .

Figure 6-3 shows constant suction direct shear tests from Hossain and Yin (2010) conducted at matric suction of 50 kPa at different initial net normal stresses for a completely decomposed granite (CDG). The thick full line represents the failure line with a slope of $\tan \phi'$. It can be seen from Figure 7 that total cohesion varies from 40 kPa (full line) to 62 kPa (dotted line) for initial net normal stresses of 50 and 300 kPa, respectively. Hence, it is common to use a line with a different slope than $\tan \phi'$ to interpret the test results. Another plausible explanation for such test results is that ϕ' is affected by the applied net normal stress. At high net normal stress, an unsaturated soil specimen may compress leading to an increase in ϕ' due to the reduction of void ratio. Such an effect cannot be separated and quantified easily. Therefore, for all soils in this study, where there is a variation in the total cohesion with the net normal stress, the lowest net normal stress which is least affected by compression of voids was used for deducing the total cohesion.

Figure 6-2: Mohr failure envelope from direct shear tests on saturated Bukit Timah Granite residual soil (data from Han 1997).

Figure 6-3: Stress paths for unsaturated direct shear tests for CDG at 50 kPa matric suction (data from Hossain and Yin (2010)).

6.2.1: Proposed shear strength equation

Interpretation of CS direct shear tests requires a shear strength equation. Equations 6-1 and 6-2 explicitly assume that the contribution of matric suction to shear strength is in the form of $\chi(u_a - u_w)$ and $(u_a - u_w) \tan \phi' / \tan \phi^b$, respectively. Other forms have been suggested by Karube et al. (1996), Lu et al. (2010), Kim et al. (2010), Cha et al. (2014), Patil et al. (2019). These forms are restrictive. Regardless of the form, the contribution of matric suction to shear strength can be collectively termed suction stress, a term initially coined by Karube et al. (1996) and widely promoted by Lu et al. (2010). Hence, Equations 6-1 and 6-2 can be generalised using suction stress, P_s , instead as shown in Equation 6-3.

$$\tau = c' + (\sigma - u_a) \tan \phi' + P_s \tan \phi' \quad (6-1)$$

Following Equation 6-1, P_s can be obtained from C as follows:

$$P_s = \frac{(C - c')}{\tan \phi'} \quad (6-2)$$

The variation of P_s with matric suction for the soils listed in Table 6-2 is shown in Figure 10. Considering Figure 6-4, one possible form for P_s is given by Equation 6-3.

$$P_s = \left[B \cdot P_a \left(\frac{u_a - u_w}{P_a} \right)^{k_2} \right] \quad (6-3)$$

where B and k_2 are constants, P_a is the atmospheric pressure. It is further suggested that B and k_2 are related to basic soil properties.

To determine the relationships of parameters, B and k_2 , with basic soil properties, the best-fit values of B and k_2 for each soil in Table 6-2 were first determined. A correlation matrix between B and k_2 with the basic soil properties and combinations (α , n , d_{50} , C_c , C_u , P_{200} , $\alpha \cdot C_u$, $\alpha \cdot C_c$, $n \cdot C_u$, $n \cdot C_c$, $\tan \phi'$) was then determined.

The correlation matrix is shown in Table 6-4. Parameter B has the highest correlation with $n \cdot C_c$ (0.94294) while parameter k_2 has the highest correlation with $\tan \phi'$ (-0.96225). Regression analyses were performed for parameter B with $n \cdot C_c$ to give Equation 6-7 and for k_2 with $\tan \phi'$ to give Equation 6-5.

Figure 6-4: Variation of suction stress with suction

The relationships for Equations 6-4 and 6-5 are shown in Figures 6-5 (a) and (b), respectively.

$$B = 1.9118 \cdot \exp^{-0.526 n \cdot C_c} \quad (6-4)$$

$$k_2 = 3.0728 \exp(-2.729 \tan \phi') \quad (6-5)$$

Figure 6-5: Relationships of parameters B and k_2 with basic soil properties.

6.2.2: Validation of proposed shear strength equation

To test the validity of the proposed shear strength equation, three soils from Nam et al. (2011) and one soil from Gan (1986) were used. These data were not used in the derivation of the relationships for parameters B and k_2 in the proposed shear strength equation. The parameters of the validation dataset are summarized in Table 6-5. The performance of the proposed shear strength equation is shown in Figure 6-6.

It can be observed that the proposed shear strength equation provides good estimates of the shear strength compared to the reported values.

Table 6-3: Correlation matrix for parameters B and k_2

Parameter	B	k_2
a	-0.83054	-0.59959
N	-0.65744	-0.31682
P_{200}	0.287019	0.687369
D_{60}	-0.30424	-0.71732
D_{50}	-0.54496	-0.74715
D_{30}	-0.64561	-0.79031
D_{10}	-0.33103	-0.46481
C_c	-0.88385	-0.66317
C_u	0.8091	0.4594
$\tan \phi'$	-0.5178	-0.96225
n.Cc	-0.94294	-0.67065

Table 6-4: Parameters of validation data used in the proposed shear strength model.

Reference		Nam et al. (2011)		Gan (1986)
USCS	CL	SM	MH	CL
c' (kPa)	5.00	4.30	15.80	10.49
ϕ' (°)	32.4	35.0	32.0	25.5
S_r	0.0	0.0	0.0	0.0
α (1/kPa)	0.0025	0.0244	0.0017	0.0036
n	1.450	1.468	1.368	1.343
C_c	1.00	1.06	1.76	1.33
C_u	100.0	40.9	46.0	208.3

(a) CL (data from Nam et al. 2011)

(b) SM (data from Nam et al. 2011)

(c) MH (data from Nam et al., 1996)

(d) CL (data from Gan, 1996)

Figure 6-6: Validation of the proposed shear strength model.

6.2.3: Summary

Constant-suction direct shear test for unsaturated soil was reviewed. There are two interpretation methods for constant suction direct shear tests. One method suggests soils tested at different matric suctions have different friction angles while the other method suggests that soils tested at different matric suctions have the same friction angle. Interpreting the constant-suction direct shear tests by plotting the stress path where both net normal and shear stresses are corrected for the shear area is useful to identify the failure line especially in cases where there is no peak or maximum shear stress with shear displacement.

The stress path plotted for such test shows that the shear stress moves along the failure line once the shear stress reaches the failure line removing the ambiguity in picking the correct failure stresses. Constant suction direct shear tests from the literature were collated and re-analysed to show that it is more correct to plot corrected shear stress versus corrected net

normal stress for constant-suction direct shear tests with a failure line at a gradient of $\tan \phi'$. For each matric suction, a higher net normal stress can cause the void ratio of soil to decrease and hence the effective stress friction angle to increase.

Based on the extended Mohr-Coulomb envelope, a new shear strength equation using suction stress to account for the contribution of matric suction to the shear strength of unsaturated soils is proposed. The suction stress was found to vary non-linearly with matric suction and is related to the n parameter in van Genuchten (1980) soil-water characteristic curve equation, coefficient of compressibility C_c and $\tan \phi'$. The proposed shear strength equation was validated for four soils and good agreement were obtained between the estimated and experimental shear strength. The form proposed for suction stress should be examined for a wider range of soils to examine its validity.

6.3: Constant water content tests

6.3.1: Introduction

This section describes the results of the constant water (CW) direct shear tests conducted in this study for specimens prepared by standard Proctor compaction. The main challenge in CW direct shear test is the lack of suction measurement during the shearing process. However, attempts have been made by several researchers to monitor suction as already described in Chapter 2.

For this study, suction was not directly measured but rather a simple interpretation based on the Hilf analysis of pore air/water pressure (Section 2.4.1.2) was adopted to evaluate variations in suction during shearing. The results of the CW direct shear tests on the as-compacted as well as specimens dried from the as-compacted condition were illustrated. In this study, only the drying path was examined because of the difficulty of wetting as-compacted specimens homogeneously.

This section covers the saturated direct shear tests, followed by unsaturated tests by CW direct shear tests together with its analysis, matric suction estimation procedure and the proposed interpretation.

6.3.2: Saturated shear strength parameters

As stated earlier, the interpretation of shear strength of unsaturated soils requires the saturated shear strength parameters and unsaturated soil tests. It is therefore vital to assess the saturated shear strength parameters of the soil i.e., ϕ' and c' . It has been already highlighted that different compaction water contents yield different soils and different ϕ' and c' apply to soils at each compaction water content. The direct shear test has been described in Chapter 3. Figure 6-11 (a and b) shows typical shear stress and vertical displacement versus shear displacement for (a) dynamically, and (b) statically compacted saturated soil. Other shear stress versus horizontal displacement plots are shown in Appendices B and C (for unsaturated soils). The specimens were compacted at the same water content and to the same dry density and so any difference is due solely to the method of compaction.

Generally, there is an increase in shear stress with an increase in the net normal stress. Dynamically compacted specimens showed contraction behaviour at higher net normal stresses (80, 160 and 320 kPa) but dilation behaviour at low net normal stress (40 kPa). This behaviour was opposite for statically compacted specimens where dilation was observed at higher net normal stresses (80, 160 and 320 kPa) and contraction at low net normal stress (40 kPa) as in

Figure 6-7. In addition, the shear stresses for soil specimens compacted on the dry of optimum are higher than those compacted on the wet of optimum at a comparable dry unit weight and under the same normal load.

Figure 6-7: Typical shear stress and vertical displacement versus shear displacement for dynamically and statically compacted soil specimens at a compaction water content of 26%.

For dynamically compacted specimens, there is a clear observable peak (Figure 6-7(a)) followed by strain-softening behaviour which is typical of an overconsolidated soil. It is easy to obtain the peak shear strength corresponding to each net normal stress for such tests. For this reason, the corrected (where an area correction is applied, and the stress path used) and uncorrected stresses (where the initial net normal stress remains constant) give almost the same values of ϕ' and c' .

For statically compacted specimens, there are no clear peaks (Figure 6-11 (b)) and strain-hardening behaviour is observed typical of a normally consolidated soil making it hard to

choose the peak that corresponds to the applied net normal stress. As a result, researchers either use an arbitrary horizontal displacement or choose a strain rate ranging between 5 to 12 % depending on the recommendations from different standard codes.

Typical Mohr-Coulomb failure envelopes for dynamically and statically compacted specimens are shown in Figure 6-8. For example, in Figure 6-7(b), for all the net normal stress, it was assumed that the peak occurred at a horizontal displacement of 5 mm (about 8 % strain). For this soil, the assumption might work as the difference between the use of stress path method and assumed horizontal displacement at failure show close values, but it might not work for other soils. To circumvent this problem, the stress path method to analyse the test results which has been illustrated in the previous section for CS direct shear tests was used. Therefore, throughout this study, the stress path method was used to analyse the direct shear test results.

Figure 6-8: Typical Mohr-Coulomb envelope for dynamically and statically compacted specimens at 26 % water content.

Table 6-5 summarizes ϕ' and c' for the compacted soils prepared at different water contents and using different compaction methods. Also shown in Table 6-5 is the average ϕ' for the same compaction method regardless of compaction water content. From Table 6-5, it can be observed that c' ranges from 3.5 to 6 kPa and from 4.5 to 8.6 kPa, and ϕ' ranges between 29.6° to 32.9° and 30.2° to 32.9° for dynamically and statically compacted specimens respectively. Figure 6-9 shows the interpretation of the effective cohesion, c' using the minimum, average and maximum friction angles for all the compaction methods.

Though the variation in the friction angle is small, as earlier mentioned, different densities and compaction water contents yield “different soils”. In this study for interpretation of the unsaturated tests, the ϕ' and c' obtained for saturated tests prepared at those water contents were used.

Table 6-5: Variation of friction angles and cohesion values for the different compaction methods and water contents.

Method of compaction	Compaction Water content (%)	Effective friction angle, ϕ' (°)	Effective cohesion, c' (kPa)	Average friction angle ϕ' (°)	Average cohesion c' (kPa)
Dynamic compaction	14.0	32.89	3.5	30.89	4.7
	18.0	30.46	4.5		
	22.0	30.60	4.8		
	26.0	29.63	6.0		
Static compaction	14.0	32.00	4.5	31.77	7.1
	19.0	32.90	7.1		
	21.0	32.20	7.5		
	22.7	32.80	8.5		
	24.2	30.50	8.6		
	26.0	30.20	6.6

Figure 6-9: Errors in determining shear strength parameters of compacted soils.

6.3.3: Analysis and interpretation

For each compaction water content, the total cohesion was obtained from the CW direct shear test results using the corresponding ϕ' . The peak shear stresses for the CW tests were obtained from the stress path method. For tests where the stress path continues along the failure line, the first point on the failure line was used as the peak shear stress with the corresponding net normal stress. Typical peak shear stresses for the unsaturated specimens at different net normal stresses are plotted in Figures 6-10 and 6-11 for compaction water contents of 14.0 and 26.0%, respectively. The full line represents the saturated test while the dashed lines represent the unsaturated peak shear stresses of unsaturated tests at the same gradient as for the saturated tests, i.e., $\tan \phi'$.

It can be observed from Figure 6-10 that a single line at gradient $\tan \phi'$ goes through all the peak shear stresses at various net normal stresses but Figure 6-11 shows that a single line at gradient $\tan \phi'$ does not pass through all the peak shear stresses at various net normal stresses. A plausible explanation for the difference is that a soil specimen compacted on the dry of optimum does not experience significant compression under the applied net normal stress to cause a change in the matric suction. The situation is reversed for a specimen compacted on the wet of optimum where significant compression occurs as the net normal stress caused the void ratio to reduce and degree of saturation to increase leading to a decrease in matric suction causing the line to shift downwards with a reduced total cohesion C . The total cohesion, C , is a combination of c' (from saturated direct shear tests) and the contribution from suction stress.

Figure 6-10: Maximum shear and effective/net normal stresses for specimens compacted at $w = 14\%$.

Figure 6-11: Maximum shear and effective/net normal stresses for specimens compacted at $w = 26\%$.

At the highest net normal stress in Figure 6-11, the peak shear stresses for both dynamically and statically compacted soil specimens lie below the saturated line. It is thus hypothesized that the degree of saturation of the unsaturated soil specimen at the highest net normal stress of 320 kPa is close to saturation such that positive u_w develops at the shearing rate of 1mm/min.

To test the hypothesis, unsaturated soil specimens compacted at 26% water content were sheared under the two highest net normal stresses of 160 kPa and 320 kPa at a reduced shearing rate of 50% of 1 mm/min (i.e., 0.5 mm/min). This is illustrated in Figure 6-12 for a statically compacted specimen. The shear stress–displacement curves at a normal stress of 160 kPa are almost the same but at normal stress of 320 kPa, the shear stress-displacement curve at the slower rate is higher (Figure 6-12a). In Figure 6-12b, the stress paths at the slower shear rate agree better with the failure line.

It can also be observed that for a lower net normal stress (160kPa), there is an insignificant difference between the peak shear strengths as a result of a reduction in the shearing speed. Therefore, a shearing rate of 0.5 mm/min was only used for the highest net normal stress for specimens compacted at a water content of 26 %.

The total cohesion values observed at different water contents and net normal stresses are summarised in Table 6-6. The variation of total cohesion with net normal stress for soil specimens is shown in Figure 6-13.

Typical shear stress versus horizontal displacement curves

Stress path at net normal stresses of 160 and 320 kPa

Figure 6-12: Effect of shearing rate on the peak shear strength

It is observed from Figure 6-13 that the total cohesion remains constant at low compaction water contents irrespective of the net normal stress. However, at higher water contents and net normal stresses (Table 6-6), there is a decrease in the total cohesion. This suggests that the matric suction may remain the same as the as-compacted specimen suction at the dry of optimum (low water contents) as well as low net normal stress at wet of optimum (high water contents) but decreases with an increase in the net normal stress at wet of optimum. A similar observation has been made by researchers such as (Oloo and Fredlund 1996, Vanapalli and Lane 2002, Cokca et al. 2004). Further to these observations, the use of an approximation method proposed by Oloo and Fredlund (1996) has been adopted to illustrate that the change

in suction is minimal during and after shear. This observation becomes interesting specifically for soils with low water contents since there is a difficulty to use either the ATT or HCT test methods under constant suction tests. The estimation of matric suction from CW direct shear test is illustrated in the next section.

Table 6-6: Total cohesion values

Method of compaction	Net normal stress (kPa)	40	80	160	320
	Water content (%)	Total cohesion values (kPa)			
Dynamic compaction	14.0	78			
	18.0	61			
	22.0	46	38	25	16
	26.0	37	25	18	12
Static compaction	14.0	70			
	19.0	55	54	50	48
	22.7	44	38	20	11
	24.0	43	38	28	10
	26.0	33	25	15	6.6

Figure 6-13: Variation of total cohesion with net normal stress at different water contents.

6.3.4: Estimation of matric suction

As mentioned in section 2.4.1.2, the lack of suction control or measurement during the shearing process poses the main challenge to the interpretation of the CW direct shear test. As a result, the matric suction has to be estimated. Details have been covered in Chapter Two of this thesis

but a summary of the approach to estimate suction change when using a conventional direct shear apparatus is as follows:

- 1) Ensure there is minimal evaporation during the test using grease at the halves of the shear box and covering the entire direct shear box during the test.
- 2) Measure the as-compacted suction (using either HCT or filter paper method).
- 3) Compute s_i , (the suction at the end of the compression stage) using the approach proposed by Oloo and Fredlund (1996) from the time-settlement curve.
- 4) Use a fast-shearing rate as this will reduce testing time as well as reducing the possibility of the pore-water pressure to equilibrate but ensure that the chosen shearing rate does not allow excess pore-water pressure development during shearing for specimens with a high degree of saturation, especially under high net normal stress.
- 5) Record the horizontal and vertical displacements during shearing.
- 6) Compute the volumetric strains using Equation 2-21. (Some software automatically compute the volumetric strains and these can be used).
- 7) Compute the cumulative change in suction for the respective horizontal increments using Equation 2-26.
- 8) Finally, estimate the suction during shearing using Equation 2-27.

Based on the above approach, the suctions for specimens compacted at 26% water content were estimated during and at the end of shearing. The parameters used are shown in Table 6-7. The estimated suctions at peak shear are extrapolated using horizontal displacement at which the peak shear stress is taken.

Figure 6-14 shows a typical suction variation with the horizontal displacement for a dynamically compacted specimen at 26 % water content. It can be observed that the change in suction during shear is small even with the highest net normal stress of 320 kPa used in this study. From this observation, it can be concluded that the choice of an appropriate shearing rate can ensure no excess pore-water pressure development in the specimen during shearing. A similar trend was observed for the other specimens prepared at different water contents. Therefore, it was concluded that there was negligible change in suction and the initially measured suctions for the as-compacted specimens could be taken as the suctions at failure.

Table 6-7: Suction estimation during direct shearing

Void ratio	e	0.749				
Compaction water content	w (%)	26				
Initial degree of saturation	S ₀ (%)	92				
Initial porosity	n ₀	0.428				
Shear zone	L (mm)	0.2				
Absolute pressures	u _{Ti} (kPa)	100				
Coefficient of solubility	H (cm ³ (STP))/(cm ³ (pol) atm))	0.1293				
Initial net normal stress	($\sigma - u_a$) kPa	40	80	160	320	
Initial measured suction	(u _w - u _a) kPa	47	47	47	47	
Suction at the end of consolidation	s _i (kPa)	46.94	46.89	46.83	46.76	
Suction at peak shear stress	s _f (kPa)	46.81	46.99	44.22	42.48

Figure 6-14: Suction variation during shearing under different initial net normal stresses.

6.3.5: Proposed interpretation

From Figures 6-10 and 6-11, the total cohesion, C is obtained for each net normal stress and water content. From Equation 2-32 the parameter $\tan \phi^b$ can be computed using Equation 6-6.

$$\tan \phi^b = \left(\frac{C - c'}{(u_a - u_w)_f} \right) \quad (6-6)$$

The observation in Figures 6-10 and 6-11 can be explained using Figure 6-15. Once a specimen is subjected to loading, the void ratio reduces from e_0 , and its SWCC becomes a new SWCC corresponding to the SWCC with the lower void ratio (shown as e_1 , e_2 and e_3 in Figure 6-15). On the wet of optimum, there is a reduction in the void ratio with the normal load. Hence for the constant water content direct shear test as indicated by line A-A, there is a reduction in the matric suction. Thus, the intercept of the constant gradient line shifts downwards towards the saturated shear strength line. On the dry side of optimum, the reduction in the void ratio is minimal as indicated by line B-B and there is almost no significant change in the suction.

Figure 6-15: SWCC of clayey silty sand with different void ratios (Modified from Wijaya & Leong, 2015).

Hence, CW direct shear tests at the lower net normal stresses will experience a negligible change in void ratio and the intercept will reflect a closer suction as the as-compacted specimen. Hence, the intercept can be used to evaluate $(u_a - u_w)$ of the specimen. The same approach was applied to specimens compacted at wet of optimum and dried to specific water contents. Equation 2-29 can be further expressed using Bishop's χ as shown in Equation 6-7. With ϕ' and the computed values of ϕ^b , the χ value can be computed from Equation 6-7.

$$\chi = \frac{\tan \phi^b}{\tan \phi'} \quad (6-7)$$

For illustration, results from dynamically compacted specimens are shown in Table 6-8 for the respective compaction water contents. From Table 6-8, the compaction water contents of 20, 22, 24 and 26.0% only the $\chi (= \tan \phi^b / \tan \phi')$ indicated with * is used as it is assumed that under the low net normal stress, the matric suction is more accurate or unchanged at the initial matric suction measured.

Table 6-8: Summary of CW direct shear test results of dynamically compacted specimens

Compaction water content (%)	Net normal stress (kPa)	Total cohesion, C (kPa)	Suction (kPa)	$\tan \phi^b$	$\phi^b (^\circ)$	$\chi = \frac{\tan \phi^b}{\tan \phi'}$	Degree of saturation (%)
14	40	78	1421.63	0.0516	2.95	0.0879	49.52
	80						
	160						
	320						
18	40	61	506.08	0.1112	6.35	0.1896	72.12
	80						
	160						
	320						
22	40	55	168.50	0.2985	16.62	0.5088*	91.26
	80	38		0.1976	11.18	0.3368	
	160	25		0.1205	6.87	0.2053	
	320	16		0.0671	3.84	0.1143	
26	40	37	58.47	0.5525	28.92	0.9416*	94.41
	80	25		0.3472	19.15	0.5918	
	160	18		0.2275	12.82	0.3877	
	320	12		0.1249	7.12	0.2128	

* Indicates the used equivalent χ value at the lowest net normal stress.

A plot of $\tan \phi^b / \tan \phi'$ versus degree of saturation (S) is given in Figure 6-16. The $\tan \phi^b / \tan \phi'$ ratios fall within the assumed ranges of $\tan \phi^b / \tan \phi'$ (0.1 to 1.0) for Bukit Timah Granite residual soils observed by Rahardjo et al. (2004). It can be also observed from Figure 6-16 that there is a sharp drop in $\tan \phi^b / \tan \phi'$ within the transition zone where the unsaturated soil transits from one of the continuous water phase to one of the continuous air phase.

The as-compacted soils give lower values for $\tan\phi^b$ compared to soils dried from a wet of optimum compacted condition and therefore, might provide conservative $\tan\phi^b$ for engineering design. The ratio of $\tan\phi^b / \tan\phi'$ is 0.85 at the $S = 94\%$ (continuous air phase) and is about 0.1 at $S = 50\%$ (continuous air phase). An attempt has been used to compare the observations with other data from the literature.

Figure 6-16: Plots of $\tan\phi^b / \tan\phi'$ versus degree of saturation for the as-compacted and dried soil specimens

The equivalent χ values are also supplemented with data from the literature (Table 6-9) and plotted in Figure 6-17. The χ values are bounded by two curves.

Table 6-9: Properties of soils tested in direct shear from previous studies.

Reference	USCS Soil description	Condition	Test procedure*
Heitor et al. (2013)	SP-SC,	Compacted	CW
Kato et al. (2011)	SP-SM,	Compacted	CS
Jotisankasa and Mairaing (2010)	ML	Undisturbed	CW
Gan (1986)	CL	Compacted	CS

* CW = constant water content, CS = constant suction

It can be observed that the range of S for a particular χ bounded by the upper and lower curves is small. The curves can be represented by Equation 6-8.

$$\chi(S) = \frac{\tan \phi^b}{\tan \phi'} = \begin{cases} 0.005e^{6.143S} & \text{for upper bound} \\ 0.0018e^{6.143S} & \text{for lower bound} \end{cases} \quad (6-8)$$

where S_r is the degree of saturation.

Equation 2-29 can be combined with Equation 6-8 to give a lower and upper bound shear strength model for unsaturated soils shown as Equation 6-9.

$$\tau = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \chi(S_r) \tan \phi' \quad (6-9)$$

Figure 6-17: Equivalent χ value versus degree of saturation

Best-fits curve parallel to the upper and lower bound curves can be drawn for each set of soil data as shown in Figure 6-18 and described using an exponential function as shown in Equation 6-10.

$$\chi(S) = C.e^{6.143S} \quad (6-10)$$

Using Equation 6-10, different values of C for each soil listed in Table 6-9 were determined as summarised in Table 6-10. To determine the relationships of the constant C , with basic soil properties, a correlation matrix between C with the basic soil properties and combinations (d_{60} , d_{50} , d_{30} , d_{10} , C_c , C_u , P_{200} , LL , PL , PI and $\tan \phi'$) was first determined. The correlation matrix is

shown in Tables 6-11. The constant C has the highest correlation with C_c (0.9887). A regression analysis was then performed for the constant C with C_c to give Equation 6-11.

$$C = 0.0003 \cdot C_c + 0.0026 \quad (6-11)$$

The relationship for Equation 6-11 is shown in Figures 6-19. For practical application, Equation 6-10 can be written as Equation 6-12 to restrict the value of $\chi(S)$ to be at most equal to S at the high end of S .

$$\chi(S) = \min(C \cdot e^{6.143S}, S) \quad (6-12)$$

Figure 6-18: Regression lines for χ value for different soils

Figure 6-19: Relationship of constant C with C_c

Table 6-10: Calibration data basic soil properties

Reference	C	d ₆₀	d ₅₀	d ₃₀	d ₁₀	c _c	c _u	P ₂₀₀	tan ϕ'	LL	PL	PI
Kato et al. (2011)	0.00375	2	1.2	0.3	0.01	4.50	200	25	0.7659	0	0	0
Heitor et al. (2013)	0.00277	0.19	0.17	0.13	0.07	1.27	2.71	63	0.5672	25.5	15.5	10
Jotisankasa and Mairaing (2010)	0.00264	0.034	0.015	0.0027	0.0013	0.17	26.15	77	0.5475	41.6	15	26.6
Gan (1986)	0.00298	0.025	0.013	0.002	0.00012	1.33	208.33	87	0.4754	35.5	16.8	18.7
This study (Dynamic compaction)	0.00254	0.25	0.025	0.0028	0.0014	0.02	178.57	58	0.5985	56	36	20
This study (Static compaction)	0.00260	0.25	0.025	0.0028	0.0014	0.02	178.57	58	0.6200	56	36	20

Table 6-11: Correlation matrix

	C	d ₆₀	d ₅₀	d ₃₀	d ₁₀	c _c	c _u	P ₂₀₀	tan ϕ'	LL	PL	PI
C	1											
d ₆₀	0.902207	1										
d ₅₀	0.93821	0.987838	1									
d ₃₀	0.882302	0.912053	0.954434	1								
d ₁₀	-0.00359	-0.04309	0.055994	0.351157	1							
c _c	0.988783	0.907624	0.951976	0.935608	0.133562	1						
c _u	0.370865	0.374745	0.277817	0.062862	-0.65744	0.283779	1					
P ₂₀₀	-0.64418	-0.90137	-0.85459	-0.81885	-0.0793	-0.68462	-0.27504	1				
tan ϕ'	0.65022	0.910568	0.859772	0.792862	-0.02507	0.673166	0.277198	-0.98675	1			
LL	-0.91498	-0.78381	-0.86992	-0.92377	-0.34604	-0.94926	0.014526	0.54919	-0.53497	1		
PL	-0.82065	-0.6186	-0.72611	-0.76081	-0.24295	-0.83062	0.15516	0.306435	-0.32811	0.938438	1	
PI	-0.84308	-0.84687	-0.88149	-0.95119	-0.41839	-0.90534	-0.19652	0.780919	-0.71696	0.860078	0.630896	1

Using Equations 6-9 and 6-12, the suction of the CW direct shear tests can be estimated. The estimated suctions for the soils listed in Table 6-12 are shown in Figure 6-20. It can be observed from Figure 6-20 that the relationship between $\log(\text{suction})$ and compaction water content is a straight line with a high R^2 value. Similar observations were made by (Oloo and Fredlund 1996, Leong et al. 2003, Taibi et al. 2011).

For three of the datasets from Nam et al. (2011), the data points indicated by fill symbols in Figure 6-20 at the high end of compaction water content do not fall on the straight line. Comparing the water content of these data points with the SWCCs from Nam et al. (2011) indicate that these water contents were before the air entry value and therefore are not on the straight line.

Figure 6-20: Estimated Suction variation and compaction water content.

To substantiate the above discussion, the data in Figure 6-20 were plotted together with the SWCCs for the soils in Figure 6-21. In Figure 6-21, the SWCCs plotted were the experimental SWCC and the SWCC estimated using Chin et al. (2010). The parameters used for estimating the SWCC using Chin et al. (2010) are summarised in Table 6-12.

Table 6-12: Parameter used in Chin et al. (2010) SWCC estimation.

Reference	Oloo and Fredlund (1996)	Nam et al. (2011)			Rahardjo et al. (2004)
Soil type	CH	MH	SM	CL	CL
Parameter					
a	194.0	410.0	9.2	506.0	458
n	0.605	0.491	0.683	0.42	0.46
m	0.654	0.453	0.807	0.35	0.40
ψ_r (kPa)	588.6	704.7	504.6	763.4	733.5
Variable, x	220	130	297	90	110
Void ratio, e	0.53	0.96	0.82	1.04	0.545
G_s	2.68	2.73	2.69	2.72	2.72
Porosity, n	0.35	0.49	0.45	0.51	0.35

It can be observed that the estimated suctions are close to the best fit for the SWCC experimental data. As mentioned earlier, soils compacted at different water content have different soil structures and are expected to have a slightly different SWCC than those that undergo drying. A small difference is observed, and the points are close to the SWCC data for clay soils. However, the model overestimated the SWCC for the sand material, the reason is attributed to the difference in the degrees of saturation corresponding to the different preparation moisture contents.

(a)

(b)

(c)

(d)

(e)

Figure 6-21: Comparison of SWCCs with the estimated suctions from Figure 6-20 for soils used in the validation datasets.

6.3.6: Summary

This section investigated the use of conventional direct shear tests to obtain unsaturated shear strength parameters from CW tests. The important findings in this section are summarised below.

1. The use of stress path can improve the accuracy of estimating the peak shear strength for shear stress plots that do not show a clear peak during shear.
2. Stress history, suction, void ratio, strain rate and structure have a significant influence on the shear strength behaviour of unsaturated soils.
3. The as-compacted soils give lower values for $\tan\phi^b$ compared to soils dried from a wet of optimum compacted condition and therefore, might provide conservative $\tan\phi^b$ for engineering design.
4. The shear strength of compacted residual soils can be determined using constant water content direct shear tests and interpreted using the extended Mohr-Coulomb failure criterion for unsaturated soils.
5. The contribution of matric suction to shear strength was expressed as $(u_a - u_w)\chi(S) \tan\phi'$ where $\chi(S) = \tan\phi^b / \tan\phi'$. The $\chi(S)$ relationship for each soil can be described using an exponential function of S (Equation 6-10).
6. The constant C in the exponential function (Equation 6-10), is related to the coefficient of curvature as given in Equation 6-11.
7. Using Equations 6-10 and 6-11, the matric suction of a compacted soil can be estimated from CW direct shear tests. The estimated matric suctions were compared with the SWCCs of the soils and good agreement was obtained.
8. Using Equations 6-11 and 6-12, the shear strength of a compacted soils can be estimated from CW direct shear tests if the initial matric suction of the compacted soil is known.

Chapter 7: Tensile strength of compacted soils

7.1: Introduction

Soils at the ground surface experience multiple cycles of drying and wetting. On drying, the soils experience shrinkage and cracks may appear. Conventional shear and compression test apparatus cannot determine the tensile strength of soil directly. This has led to the development of indirect methods to determine the tensile strength of soils. The advantages and disadvantages of the different tensile test methods have been discussed in Chapter 2. In this chapter, the Brazilian tensile strength (BTS) test was used to determine the tensile strength of compacted soil. Factors affecting the tensile strength of soils and the change in tensile strength as a soil dry were investigated. A new model is also proposed to estimate the tensile strength of coarse and fine-grained soils.

7.2: Brazilian tensile strength

Two test series were conducted in this study. The first test series examines the tensile strength of the as-compacted soil specimens whereas the second test series investigates the tensile strength development of soil specimens compacted wet of optimum as it dries to the dry of optimum water content. The test specimens used for Test series 1 and 2 were statically compacted. The preparation of the soil specimens and the test procedures have been detailed in Chapter 3. Observations during the BTS tests show that crack initiated at the centre of the disk and the stress-displacement curves exhibited brittle failure in almost all the tests indicating the general applicability of the BTS test for the compacted soils in this study (Frdyman, 1964; 1967).

7.2.1: As-compacted soil specimens.

In this subsection, the BTS test results of as-compacted specimens are presented and discussed. Table 7.1 shows the average water contents, target and achieved dry densities of the specimens used. Figure 7.1 shows test series 1 specimens on the standard Proctor compaction curve.

Table 7-1: Properties of test specimens

Compaction water content (%)	Compacted at	Target dry density (Mg/m ³)	Achieved dry density (Mg/m ³)
14	Dry of optimum	1.515	1.511
18	Dry of optimum	1.595	1.595
22	Wet of optimum	1.617	1.612
26	Wet of optimum	1.515	1.505

Figure 7-1: Average moisture content and dry density of specimens for Test series 1 shown with the standard Proctor compaction curve.

Figure 7.2 shows the variation of Brazilian tensile strength σ_t and dry density with compaction water content for Test series 1. From Figure 7.2, it can be observed that σ_t shows a similar trend as the compaction curve, increasing with water content on the dry side of optimum to a peak value and reducing with increasing water content. The peak σ_t is observed to occur close to the optimum water content. This suggests that both void ratio and compaction water content influence σ_t . There is more scatter in σ_t at the dry of optimum and this is a result of the sensitivity of σ_t to the small differences in soil structure as it is harder to compact samples at the dry of optimum.

A comparison of the σ_t at the same dry densities shows that the dry of optimum specimens show a higher tensile strength as compared to the wet of optimum specimens. The difference can be attributed to the difference in soil structure. It has been mentioned in Chapter 2 that different soil structures are produced when soil specimens are compacted at the wet and dry of optimum even at the same dry density. Similar observations have been made by studies where specimens were dynamically compacted (Kizza 2019).

A comparison between the statically and dynamically compacted specimens of the same soil was made with data from previous studies as shown in Figure 7.3. It can be observed from

Figure 7.3 that both statically and dynamically compacted specimens show a similar trend for σ_t with more scatter in the dynamically compacted specimens. Also, dynamically compacted specimens at dry of optimum show slightly higher σ_t than the statically compacted specimens. This is because of the difference in the dry densities and greater variability of the specimens between the previous studies and the current study.

Figure 7-2: Variation of dry density and tensile strength with compaction water content

Figure 7-3: Tensile strength from statically and dynamically compacted soils based on the standard Proctor compaction curve.

7.2.2: Dried specimens compacted wet of optimum.

To examine further the influence of soil structure and matric suction on soil strength, Test series 2 was conducted. Test series 2, comprises specimens compacted wet of optimum using the

acrylic mould. As described in Chapter 3, the specimens were dried in desiccators with saturated NaCl solutions to selected water contents before testing. The σ_t of the specimens were determined when the desired water contents were achieved. Table 7-2 shows that the dry densities of the specimens that were dried increases and became higher than the dry densities of the as-compacted specimens at low water contents (17.7, 14.0 and 8.0%).

Table 7-2: Dry densities for as-compacted specimens and specimens dried from as-compacted water content of 26.0%.

Compaction water content (%)	As compacted dry density (Mg/m ³)	Average water content after drying from as compacted water content of 26.0% (%)	Average dry density after drying from as compacted water content of 26.0% (Mg/m ³)
8.0	-	8.0	1.643
14.0	1.513	13.8	1.617
17.7	1.598	17.8	1.600
21.8	1.600	21.9	1.565
26.0	1.514	26.0	1.514

Figure 7.4 shows the variation of tensile strength as the specimens dried. The results from Test series 2 are also plotted to compare with the tensile strength of the as-compacted specimens from Test series 1.

It can be observed that drying of the soil specimens from the wet of optimum to dry of optimum exhibited an increase in σ_t . The σ_t is observed to increase up to a maximum and then a slight drop in σ_t at extremely low water contents. This can be attributed to the development of internal micro-fissures at low water contents in clayey and silty materials (Sun and Cui, 2017). As the specimens dry, their stiffness and strength increase and become more brittle. A point is reached when the specimens display no more increase in stiffness and the strength increment is negligible. Therefore, the higher tensile strength observed for specimens that are dried is due to the difference in dry density and soil fabric. Similar observations have been reported by other researchers such as Zeh and Witt (2007).

Figure 7-4: Variation of tensile strength for as compacted and dried specimens with water content.

To establish the variation over which the tensile strength increment occurs, the shrinkage curve is used. Volume measurements were performed on the BTS specimens, and the shrinkage curve data is fitted using universal shrinkage equation proposed by Leong and Wijaya (2015) as shown in Figure 7.5.

Figure 7-5: Shrinkage curve for specimens compacted at wet of optimum.

Figure 7.6 shows the BTS σ_t plotted together with the SWCC data of the soil specimens.

The SWCC data was fitted with Fredlund and Xing (1994) SWCC equation using a unity correction factor as recommended by Leong and Rahardjo (1997). The degrees of saturation at which σ_t stops to increase varies between 83 and 95% (Figure 7.6) corresponding approximately to the transition zone of the shrinkage curve from the saturated line and the zero-shrinkage condition (Figure 7.5). It is therefore worth noting that during drying, there is an increase in suction and a reduction in the volume which collectively influence the σ_t of the soil. A similar trend of σ_t has been reported for dynamically compacted specimens with different Proctor compaction efforts (Kizza 2019).

The results from the current study are used as part of the validation data set for the proposed tensile strength model.

Figure 7-6: Variation of BTS σ_t with the degree of saturation

7.3: Proposed tensile strength model for coarse and fine soils.

Several tensile strength models for unsaturated soils considering the effects of matric suction have been proposed. However, the tensile strength models proposed are for either cohesionless (coarse-grained) or clayey (fine-grained) soils. Generally, tensile strength models considering the effects of matric suction have been developed for coarse-grained soils rather than for fine-grained soils and hence tensile strength data together with soil-water characteristic curve data for fine-grained soils are scarce in the literature. In some literature, unsaturated soils were prepared by compacting soil at various water contents to the same dry density, but the soils

prepared in this manner have a different soil structure. Such data are not used in this section. As tensile strength data for fine-grained soils with the same soil structure and soil-water characteristic curve data are lacking in the literature, this study has included Brazilian tensile strength tests on fine-grained soils initially compacted wet of optimum and allowed to dry to various water contents (Section 7.2.2). The matric suctions of the compacted soil specimens were measured using the contact filter paper method (ASTM D5298 – 16) as well as estimated from its soil-water characteristic curve (ASTM D6836-16). The test data highlighted the problem in the friction angle used in existing tensile strength models. Using a general form of the suction stress and the extended Mohr-Coulomb criterion with the Brazilian test Mohr circle, a new tensile strength model applicable to both coarse-grained and fine-grained soils was proposed. The proposed model was shown to perform better than existing models using independent data.

Literature review on tensile strength models for unsaturated soils is given in section 2.4.2.3 and summarised in Table 2-8. In most of the models shown in Table 2-8, the tensile strength σ_t of unsaturated soils is usually obtained from the Mohr-Coulomb criterion with the y-axis intercept given by the apparent cohesion c_{app} as shown in Figure 7.7. The σ_t is obtained from either the direct uniaxial tensile test or the indirect tensile test. If the uniaxial tensile test is used, the Mohr circle will have the minor principal stress as $-\sigma_t$ and major principal stress as 0. Based on Figure 7-7, this case will give

$$\sigma_t = \frac{2 \cos \phi_t}{1 + \sin \phi_t} c_{app}^* ; \text{ or alternatively } \sigma_t = 2 \tan \left(\frac{\pi}{4} - \frac{\phi_t}{2} \right) c_{app}^* \quad (7-1)$$

The parameters of ϕ_t and c_{app}^* are defined in Figure 7-7.

Figure 7-7: Representations of uniaxial and Brazilian tensile strength tests using the Mohr-Coulomb failure criterion.

If an indirect tensile test such as the Brazilian tensile test (ASTM D3967-16) is used, the Mohr circle will have the minor principal stress as $-\sigma_t$ and the major principal stress as about $3.1\sigma_t$ (Li and Wong 2013, Sivakugan et al. 2014, Consoli et al. 2015). In this case, σ_t is given by Equation 7-2.

$$\sigma_t = \frac{\cos \phi'}{(2.05 - 1.05 \sin \phi')} c_{app} \quad (7-2)$$

The parameters of ϕ' and c_{app} are defined in Figure 7-7.

Except for Snyder and Miller (1985) in Table 2-8, the other tensile strength models can be expressed in the form of either Equation 7-1 or 7-2 where c_{app} can be expressed in the form of the total cohesion in the extended Mohr-Coulomb criterion for unsaturated soils (Fredlund et al. 1978) as shown in Equation 7-3.

$$c_{app} = c' + (u_a - u_w) \tan \phi^b \quad (7-3)$$

where c' is the effective cohesion, u_a is pore-air pressure, u_w is pore water pressure and ϕ^b is the angle indicating the rate of shear strength increase due to matric suction ($u_a - u_w$). Equation 7-3 can also be expressed more generally in terms of a suction stress σ^s as shown in Equation 7-4.

$$c_{app} = c' + \sigma^s \tan \phi' \quad (7-4)$$

In the tensile strength model of Morris et al. (1992), the coefficient of 0.5 based on Frydman (1967) and Baker (1981) and the coefficient of $(0.5 \text{ to } 0.7) \cot \phi'$ based on Bagge (1985) shown in Table 2-8 can be examined in the context of Equations 7-2 and 7-3. Figure 7-8 shows a plot of the coefficient $\left(\frac{2 \cos \phi_t}{1 + \sin \phi_t} \right)$ from Equation 7-1 and coefficient $\left[\frac{\cos \phi'}{(2.05 - 1.05 \sin \phi')} \right]$ from Equation 7-2 versus the friction angle ϕ_t or ϕ' for the typical range from 15° to 50° . It can be seen that the coefficient $\left(\frac{2 \cos \phi_t}{1 + \sin \phi_t} \right)$ is close to $0.7 \cot \phi'$ for the range of $\phi' = \phi_t$ from 28° to 40° and the coefficient $\left[\frac{\cos \phi'}{(2.05 - 1.05 \sin \phi')} \right]$ is close to 0.5 for the full range of friction angle.

For the other tensile strength models in Table 2-8, Equation 7-1 is used and only differs in the definition of σ^s and the presence or absence of a cohesion term. The tensile strength models of Tang et al. (2015), Yin and Vanapalli (2018), and Wang et al. (2020) originated from Lu et al. (2009). The differences amongst these models lie in the definition of σ^s which can be expressed in the general form of Equation 7-5.

$$\sigma^s = S_e^k (u_a - u_w) + f \quad (7-5)$$

where S_e is the effective degree of saturation $\left(= \frac{S - S_r}{1 - S_r} \right)$, S_r is the residual degree of saturation and exponent k is a parameter. The first term on the right-hand side of Equation 7-4 is attributed to capillary suction while the second term (f) is attributed to the surface tension of the “capillary bridges” (Yin and Vanapalli, 2018; Wang et al., 2020). Only the exponent k in Yin and Vanapalli (2018) model is not unity.

Yin and Vanapalli (2018) relate k to be a function of van Genuchten (1980) soil-water characteristic curve (SWCC) equation parameter n and capillary degree of saturation S_e which is dependent on the coefficient of uniformity C_u . (The equations are shown in Table 2-8). The effective degree of saturation S_e is defined differently in Varsei et al. (2016) who make use of a microscopic degree of saturation S_r^m following Alonso et al. (2010) instead of residual degree of saturation S_r as shown in Equation 7-6.

Figure 7-8: Comparison of coefficients in Equations 7-1 and 7-2 with Morris et al. (1992) model

$$S_e^m = \frac{S - S_r^m}{1 - S_r^m} = (S)^{\alpha^*} \quad (7-6)$$

Instead of determining S_r^m , Varsei et al. (2016) proposes Equation 7-7 where α^* is a material parameter and is always equal to or greater than 1.

$$S_e^m = (S)^{\alpha^*} \quad (7-7)$$

The term f is zero in Lu et al. (2009), Trabelsi et al. (2012), Tang et al. (2015), and Varsei et al. (2016). Wang et al. (2020) followed Yin and Vanapalli (2018) formulation of f to be a product of surface tension of water T_s and the specific air-water interfacial area per pore volume a_{aw} , i.e. $f = T_s \cdot a_{aw}$. The equations are given in Table 2-8.

Lu et al. (2009), Tang et al. (2015), Yin and Vanapalli (2018) and Wang et al. (2020) make use of the van Genuchten (1980) SWCC equation in their tensile strength model. However, Wang et al. (2020) propose empirical equations to estimate van Genuchten (1980) SWCC equation parameters α and n , as shown in Table 2-8.

As the models of Lu et al. (1992), Yin and Vanapalli (2018), and Wang et al. (2020) were developed for cohesionless soils, the cohesion term is zero. However, three out of ten soils that Yin and Vanapalli (2018) used in the development of their model are fine-grained soils.

In contrast, the models of Trabelsi et al. (2012), Tang et al. (2015), and Varsei et al. (2016) were developed for clayey soils and hence contain a cohesion term. Only Varsei et al. (2016) kept the cohesion term as an effective cohesion. In Trabelsi et al. (2012) model, the cohesion term is a function of porosity while in the Tang et al. (2012) model, the cohesion is given by the residual tensile strength at full saturation. However, Tang et al. (2015) model was developed for soils compacted at various water contents to the same dry density. Thus, Tang et al. (2015) model assume that the influence of the soil structure on σ_t , α , and n is negligible.

Wang et al. (2020) developed a tensile strength model by using basic soil properties. Unlike the tensile strength models proposed by Lu et al. (2009) and Yin and Vanapalli (2018), the authors' model does not require a higher friction angle than that measured. Wang et al. (2020) tensile strength model followed closely the tensile strength model proposed by Yin and Vanapalli (2018). Two terms are used to represent the tensile strength contributions from the matric suction and air-water interfacial area given essentially by $S_e(u_a - u_w)$ and γa_{aw} , respectively. The tensile strength contribution from matric suction becomes zero at $S_e = 0$ i.e., at the residual degree of saturation S_r^r , leaving only the contribution from the air-water interfacial area.

The authors did not propose any equation to estimate S_r^r and hence, it is unclear how the S_r^r values listed in Table 7-3 were obtained. The S_r^r is an artefact from the use of the van Genuchten (1980) SWCC equation to provide a better fit to the experimental data (Leong and Rahardjo 1997). The SWCC of all soils should reach zero degree of saturation ($S_r = 0$) at maximum suction. Fredlund and Xing (1994) SWCC equation assumed that soils reached $S_r = 0$ at maximum suction of 1 GPa based on thermodynamic considerations, while Lu and Khorshidi (2015) suggested that the maximum suction could be between 475 to 1180 MPa. The authors' tensile strength model uses basic particle gradation parameters to estimate the van Genuchten (1980) SWCC equation parameters n_{vg} [Equation (6) in the original paper] and α_{vg} [Equation (7) in the original paper] and hence, the tensile strength of the granular soils [Equation (17) in the original paper]. However, determining SWCC experimentally should not be made redundant as estimating SWCC equation parameters from equations such as Equation (6) and Equation (7) always involve an error.

This was illustrated by re-analysing the results presented by Wang et al. (2020) using SWCC equation parameters obtained from curve fitting the experimentally measured SWCC. Out of the seven granular soils presented by Wang et al. (2020), experimentally measured SWCC is

not available for the silica sand and hence, Esperance sand reported by Lu et al. (2009) was used in place of the silica sand in Table 7-3. The properties of Esperance sand are: $d_{50} = 0.23$ mm, $d_{60} = 0.24$ mm, $C_u = 1.88$, $e = 0.67$, and $\phi = 50^\circ$ (Lu et al. 2009). Without an equation to estimate S_r^r , the only possible way to obtain S_r^r is by curve fitting the experimentally measured SWCC. For Esperance sand, Equation (7) and Equation (6) were used to estimate α_{vg} ($= 3.621$ kPa) and n_{vg} ($= 4.903$), respectively, and using these values of α_{vg} and n_{vg} , S_r^r ($=0.127$) was obtained by curve fitting the experimental SWCC data. The SWCC parameters α_{vg} , n_{vg} and S_r^r from the authors are summarized in Table 7-3 together with the curve fitted SWCC equation parameters α_{vg} , n_{vg} and S_r^r .

The tensile strength of Esperance sand was obtained in the same manner as for Perth sands. The comparison of the results for the SWCC and total tensile strength for Esperance sand are shown in Figure 7-9. The error between the experimentally measured tensile strength σ_{t_expt} and estimated tensile strength σ_{t_est} are represented using the root mean square error (RMSE) given by Equation 7-8.

$$RMSE = \sqrt{\frac{\sum_{j=1}^n (\sigma_{t_expt} - \sigma_{t_est})^2}{n}} \quad (7-8)$$

where j is an index and n is the number of data.

Table 7-3: Summary of van Genuchten (1980) SWCC equation parameters

Granular soil name	Wang et al. (2020)			Curve experimentally fitted SWCC measured			Curve fitted obtained by Chin et al. (2010) method		
	α_{vg} (kPa) Eq. 7	n_{vg} Eq. 6	S_r^r	α_{vg} (kPa)	n_{vg}	S_r^r	α_{vg} (kPa)	n_{vg}	S_r^r
Limestone aggregates	9.989	5.980	0.2	11.257	14.095	0.15	8.138	4.532	0.18
Glass balls	5.794	9.210	0.05	4.765	9.850	0.05	3.742	4.764	0.08
Ottawa sand F-75	3.621	4.554	0.17	2.247	3.545	0.09	2.766	4.383	0.14
Perth medium sand	1.849	7.076	0.1	1.521	5.121	0.04	1.433	3.604	0.08
Perth fine sand	4.828	7.076	0.05	3.945	5.230	0.06	4.163	6.270	0.06
Perth silty sand	7.242	5.232	0.02	3.143	3.013	0.01	4.760	6.104	0.05
Esperance Sand	3.621	4.903	0.09	1.206	5.109	0.13	2.587	6.521	0.10

(a) Predicted and fitted SWCCs

(b) Predicted tensile strength

Figure 7-9: Illustration of predicted SWCC and tensile strength for Esperance sand.

The comparison of RMSE between Wang et al. (2020) σ_{t_est} and σ_{t_est} obtained from curve fitted SWCC parameters are summarized in Table 7-4. From Table 7-4, it can be observed that σ_{t_est} using the curve fitted SWCC parameters always give the lower RMSE compared to the σ_{t_est} from Wang et al. (2020).

If experimental SWCC is not available, it is suggested that an alternative method of obtaining α_{vg} , n_{vg} (without the use of Equations (7) and (6), respectively), and most importantly S_r^r which was missing from Wang et al. (2020). The Chin et al. (2010) method is an alternative method to obtain the SWCC. Chin et al. (2010) method uses the Fredlund and Xing (1994) SWCC

equation to estimate the volumetric water content based SWCC. Volumetric water content can be converted to degree of saturation and vice versa using porosity.

Table 7-4: Comparison of RMSE

Granular soil name	Wang et al. (2020)	Scenario 1	Scenario 2
Limestone aggregates	1.669	0.836	2.450
Glass balls	0.734	0.677	0.861
Ottawa sand F-75	0.498	0.244	0.308
Perth medium sand	0.110	0.097	0.126
Perth fine sand	1.342	1.100	1.146
Perth silty sand	2.010	0.825	1.157
Esperance sand	1.274	0.435	0.865

For granular materials, the void ratio can be assumed to be constant. To apply Equation (17) of Wang et al. (2020) to predict the tensile strength, the van Genuchten (1980) SWCC equation parameters (α_{vg} , n_{vg} and S_r^r) are obtained by curve fitting the van Genuchten (1980) SWCC equation to the SWCC obtained by Chin et al. (2010) method as summarized in Table 7-3. Using the SWCC equation parameters together with Equation (17), the σ_{t_est} are generally more accurate than the σ_{t_est} from Wang et al. (2020) as shown by the lower RMSE in Table 7-4 and more importantly, this method resolved the missing residual degree of saturation S_r^r estimation in Wang et al. (2020).

In summary, the macroscopic approach tensile strength models showed variations only in the definition of the suction stress P_s and the presence of a cohesion term c for clayey soils only. The general form of the macroscopic approach tensile strength model is expressed in Equation 7-9.

$$\sigma_t = 2 \tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right) (\sigma_s \tan \phi + c) \quad (7-9)$$

The suction stress σ^s consists of either one term (Lu et al. 2009, Tang et al. 2015, Varsei 2016) or two terms (Yin and Vanapalli 2018, Wang et al. 2020).

In the development of the tensile strength model for unsaturated soils, the grain size distribution curve, SWCC, and effective shear strength parameters for the saturated soil, c' and ϕ' , are required. The tensile strength tests should also be performed for a soil either undergoing drying or wetting process and not on soils prepared at various water contents to avoid effects of soil

structure on the test results. Tensile strength test data with SWCC are more commonly found for coarse-grained (cohesionless) soils than for fine-grained (clayey) soils in the literature, hence tensile strength tests and SWCC tests were performed for fine-grained soils in section 7.2.2.

7.3.1: Model development

As mentioned earlier, Equation 7-1 is generic to the tensile strength models of Lu et al. (2009), Trabelsi et al. (2012), Yin and Vanapalli (2018), Tang et al. (2015), Varsei et al. (2016) and Wang et al. (2020). However, the friction angle is not ϕ' for the models of Lu et al. (2009), Tang et al. (2015) and Yin and Vanapalli (2018). By assuming that the extended Mohr-Coulomb criterion is tangent to the BTS test Mohr circle, Equation 7-2 was adopted instead. The biggest difference in the tensile strength models lies in the definition of σ^s . The σ^s can be made more general by adopting the form given in Equation 7-10.

$$\sigma^s = A.S^{k_1}.(u_a - u_w) \quad (7-10)$$

In Equation 7-10, A is a constant and is function of basic soil properties; degree of saturation S is used which implies that $S_r = 0$; exponent k_1 is used instead of k to indicate that it is a different exponent from that used in Yin and Vanapalli (2018). A similar non-linear suction stress-matric suction relationship as given in Equation 7-10 has been suggested by others, e.g. Kim et al. (2010). The proposed model is thus given by Equation 7-10.

$$\sigma_t = \frac{\cos \phi'}{(2.05 - 1.05 \sin \phi')} [\sigma^s \tan \phi' + c'] \quad (7-11)$$

According to Yin and Vanapalli, the exponent k is heavily related to the SWCC through van Genuchten (1980) SWCC equation parameter n and the capillary degree of saturation corresponding to the AEV (S_c) and somewhat related to the grain size gradation parameter C_u . Hence, it is reasonable to suggest that k_1 is a function of parameters associated with void ratio e, grain size distribution and SWCC. Both A and k_1 likely have different expressions for coarse and fine-grained soils. Hence, expressions for A and k_1 were obtained separately for coarse and fine-grained soils. The soil properties used to obtain the proposed model parameters are α and n of VG SWCC equation ($S_r = 0$), void ratio e, gradation parameters, d_{50} , C_u and C_c , and percentage passing no. 200 sieve, P_{200} .

A summary of the soil parameters used is tabulated in Table 7-5. For the coarse-grained soils, Ottawa sand from Goulding (2006), Perth sand, silty and fine sand from Lu et al. (2007) and medium and Esperance sand from Lu et al. (2009) was used to develop the relationships for

the proposed model parameters, A and k_1 . For the fine-grained soils, the data from Kiiza (2019) together with the mine tailings from Narvaez et al. (2015) were used to develop the relationships for the proposed model parameters, A and k_1 .

To determine the relationships of parameters, A and k_1 , with basic soil properties, an iterative optimization process was adopted. In the first iteration, the best-fit values of A and k_1 to the calibration dataset were determined. A correlation matrix between A and k_1 with the basic soil properties and combinations (α , n, e, d_{50} , C_c , C_u , P_{200} , $\alpha.C_u$, $\alpha.C_c$, $n.C_u$, $n.C_c$, $e.P_{200}$, $e.n$) was determined. A regression analysis (using common functions of linear, exponential, power, and quadratic polynomial) was then performed separately on A and k_1 with the basic soil property or combination having the highest correlation value in the correlation matrix. When two basic soil properties or combinations have the highest correlation values within 0.01, regression analysis was performed on each soil property or combination. Once the regression equation was obtained for the parameter (either A or k_1) with the highest correlation, its value is given by the regression equation and the values of the other parameter are optimized again for the calibration dataset. A new correlation matrix is then obtained for the newly optimized parameter with the basic soil property or combination. Subsequently, a regression analysis (using common functions of linear, exponential, power and quadratic polynomial) was then performed on the parameter with the basic soil property or combination having the highest correlation value. Finally, the coefficients of the regression equations for parameters A and k_1 are optimized with the calibration data set to obtain the final regression equations.

The correlation matrices for the coarse-grained and fine-grained soils are shown in Tables 7-6 and 7-7, respectively. For coarse-grained soils, parameter A has the highest correlation with C_c (0.9247) while parameter k_1 has the highest correlation with $n.C_c$ (-0.7230). As the correlation between parameter A and C_c is higher than the correlation between k_1 and $n.C_c$, a regression analysis was performed for parameter A with C_c as the independent variable to give Equation 7-12.

$$A = 9.973(C_c)^{6.2} \quad (R^2 = 0.9010) \quad (7-12)$$

Using Equation 7-12 to give the value of parameter A for each calibration dataset, values of k_1 were then optimized again to give k_{1_2} and a new correlation matrix is formed for k_{1_2} as shown in Table 7-6. The values of k_{1_2} have the highest correlation with $n.C_c$ (-0.7890). This correlation value is higher than the previous correlation between k_1 and $n.C_c$ (-0.7230). Hence,

a regression equation was obtained for k_{1_2} with $n.C_c$ as the independent variable to give Equation 7-15.

Finally, the coefficients of the regression equations for parameters A and k_1 (Equations 7-12 and 7-14, respectively) were optimised with the calibration dataset to give the final equations, Equations 7-13 and 7-15, respectively.

$$A_1 = 9.869(C_c)^{6.405} \quad (7-13)$$

$$k_1 = 2.643 \exp(-0.326 \cdot n \cdot C_c) \quad (7-14)$$

$$k_{1_2} = 2.835 \exp(-0.341 \cdot n \cdot C_c) \quad (R^2 = 0.7031) \quad (7-15)$$

For fine-grained soils, parameter A has the highest correlations with C_c (0.9859) and $n.C_c$ (0.9815) while parameter k_1 has the highest correlation with $e.n$ (-0.9289) (Table 7-7). As parameter A has a higher correlation value than k_1 , a regression analysis was performed for parameter A with C_c and $n.C_c$ as an independent variable in turn. The regression equation for A adopted was the one with the highest coefficient of determination, R^2 , shown in Equation 7-16.

$$A = 0.0255(C_c)^2 + 0.971C_c + 0.249 \quad (R^2 = 0.9722) \quad (7-16)$$

Fixing parameter, A to the values given by Equation 7-16, values of k_1 are then optimized for the calibration dataset again to give k_{1_2} and a new correlation matrix is formed for k_{1_2} as shown in Table 7-7. The values of k_{1_2} have the highest correlation with n (-0.9198) and $e.n$ (-0.9294). Hence, a regression equation was obtained for k_{1_2} with n and $e.n$ as the independent variable in turn. The regression equation for k_{1_2} adopted was the one with the highest coefficient of determination, R^2 , shown in Equation 7-17.

$$k_{1_2} = 11.293 \exp(-1.121n) \quad (R^2 = 0.9279) \quad (7-17)$$

Finally, the coefficients of the regression equations for parameters A and k_1 (Equations 7-16 and 7-17, respectively) were optimised with the calibration dataset to give the final equations, Equations 7-18 and 7-19, respectively. Note the slight changes in coefficients between Equations 7-16 and 7-18 for parameter A and between Equations 7-17 and 7-19 for parameter k_1 .

$$A = -0.395(C_c)^2 + 1.784C_c + 0.041 \quad (7-18)$$

$$k_1 = 14.115 \exp(-1.241n) \quad (7-19)$$

Table 7-5: Parameters used in the development of the proposed tensile strength model.

Reference	Soil type	$\phi'(^{\circ})$	$c'(\text{kPa})$	$\alpha (\text{kPa}^{-1})$	n	S_c	S_r	e	$d_{50} (\text{mm})$	C_u	C_c	$P_{200} (\%)$
Test data Set A*	Fine-grained (MH)	27.3	8	0.0009	1.710	0.91	0.001	0.45	0.005	6.00	0.296	72
Test data Set B	Fine-grained (MH)	28.2	10	0.0010	1.725	0.92	0.014	0.48	0.005	6.00	0.296	72
Test data Set C	Fine grained (MH)	28.6	2	0.0013	1.682	0.90	0.0	0.52	0.005	6.00	0.296	72
Test data Set D	Fine grained (MH)	27.5	5	0.0012	1.675	0.91	0.001	0.49	0.005	6.00	0.296	72
Test data Set E	Fine grained (CL)	30.0	10	0.0012	1.949	0.90	0.058	0.44	0.020	714.29	0.286	58
Test data Set F	Fine grained (CL)	29.0	10	0.0014	1.671	0.90	0.0	0.44	0.020	714.29	0.286	58
Test data Set G	Fine grained (CL)	26.0	10	0.0014	1.757	0.92	0.007	0.51	0.020	714.29	0.286	58
Goulding (2006)	F-75 Dense sand	27.3	0	0.3011	4.527	0.95	0.015	0.65	0.210	1.733	0.926	39
	F-75 Loose sand	27.3	0	0.2741	4.527	0.95	0.015	0.75	0.210	1.73	0.926	39
	F-55 Dense sand	28.0	0	0.3960	4.261	0.95	0.001	0.65	0.250	1.5	0.996	24
	F-55 Loose sand	28.0	0	0.3960	4.361	0.93	0.001	0.75	0.250	1.5	0.996	24
	F-40 Dense sand	30.0	0	0.3050	5.002	0.94	0.001	0.65	0.330	1.52	0.825	15
	F-40 Loose sand	30.0	0	0.3050	5.002	0.92	0.001	0.75	0.330	1.52	0.825	15
Lu et al. (2007)	Silty Sand	48.0	0	0.2500	3.750	0.92	0.020	0.82	0.110	1.83	0.742	96
	Fine Sand	48.0	0	0.1811	3.409	0.94	0.048	0.82	0.170	1.31	0.652	83
Lu et al. (2009)	Medium Sand	48.0	0	0.7091	3.760	0.93	0.001	0.67	0.450	1.52	0.747	0
	Esperance Sand	48.0	0	0.8295	4.109	0.92	0.093	0.82	0.230	2.00	0.781	40
Narvaez et al. (2015)	Tailing A (ML)	35.2	0	0.0056	2.145	0.91	0.0	0.84	0.012	9.5	1.684	98
	Tailing B (ML)	35.0	0	0.0125	2.468	0.92	0.010	0.79	0.030	14	2.294	99
	Tailing C (ML)	35.0	0	0.0018	2.100	0.92	0.001	0.81	0.021	20.00	0.800	100

*Test Data Sets A to G were from Kizza (2019)

7.3.2: Validation

For validation of the proposed tensile strength model, three coarse-grained soils from Win (2006), Kim and Sture (2008) and Jindal et al. (2016) and four fine-grained soils from Zeh and Witt (2005), Wong et al. (2017), Murray and Tarantino (2019) and the current study (section 7.2) were used. These data were not used in the derivation of the proposed tensile strength model. Most important of all, these data are for soils of the same soil structure and contain the SWCC information. The parameters of the validation dataset used in the proposed model are summarized in Table 7-8.

The performances of the modified tensile strength model, Lu et al. (2009) and Yin and Vanapalli (2018) models are shown in Figures 7-10 and 7-11 for coarse and fine-grained soils, respectively. The proposed model showed good agreement for the coarse and the fine-grained soils in Figures 7-10 and 7-11 except for Figure 7-11a at a high degree of saturation ($> 80\%$). The data in Figure 7-11a was for a medium plasticity clay that shrinks on drying (Zeh and Witts, 2005). For such a soil it is expected that the soil remains fully saturated as the soil shrinks on drying and only becomes less than fully saturated near the air-entry value. Hence, the data at a high degree of saturation were affected by an error in the volume measurement. The RMSE for each set of data is shown in Table 7-9. Figures 7-10, 7-11 and 7-12, and Table 7-9 show that the proposed tensile strength model outperforms Lu et al. (2009) and Yin and Vanapalli (2018) tensile strength models.

Table 7-6: Correlation matrix for coarse-grained soils

Parameter	α	n	e	d ₅₀	C _u	C _c	P ₂₀₀ (%)	a.C _u	a.C _c	n.C _u	n.C _c	e.P ₂₀₀	e.n
A	-0.0922	0.0677	-0.5721	0.0231	-0.1077	0.9247	-0.4212	-0.1441	0.1568	0.0441	0.2954	-0.4791	-0.1210
k ₁	0.0737	-0.5950	0.4269	-0.2261	-0.2739	-0.6682	0.4884	0.0740	-0.0531	-0.6723	-0.7230	0.5096	0.0741
k _{1_2}	0.1316	-0.7070	0.5667	-0.3692	-0.1289	-0.5765	0.6072	0.1368	0.0226	-0.7391	-0.7890	0.6351	0.1451

Table 7-7: Correlation matrix for fine-grained soils

Parameter	α	n	e	d ₅₀	C _u	C _c	P ₂₀₀ (%)	a.C _u	a.C _c	n.C _u	n.C _c	e.P ₂₀₀	e.n
A	0.9416	0.9568	0.8165	0.4889	-0.3912	0.9859	0.7942	-0.3126	0.9246	-0.3820	0.9815	0.8237	0.9107
k ₁	-0.7812	-0.8649	-0.8982	-0.5768	0.3166	-0.8554	-0.8163	0.1937	-0.7471	0.3197	-0.8422	-0.8814	-0.9189
k _{1_2}	-0.7950	-0.9198	-0.8685	-0.5613	0.3507	-0.8763	-0.8321	0.2750	-0.7733	0.3406	-0.8664	-0.8732	-0.9204

Table 7-8: Parameters of validation data used in the proposed tensile strength model.

Reference	Win (2006)	Kim and Sture (2008)	Jindal et al. (2016)	Zeh and Witt (2005)	Wong et al. (2017)	Murray and Tarantino (2019)	Current study
Soil type	Clayey sand	Medium sand	Silica sand	Clay	Clay	Vitreous Clay	Clayey silt
Tensile test	Direct	Direct	Uniaxial	Direct	Uniaxial	Uniaxial	Indirect
c' (kPa)	0.0	0.0	0	16.8	27.9	0	6.6
ϕ' (°)	25.0	32.0	48.0	25.0	32.3	27.8	30.2
n	1.655	4.145	4.492	1.578	2.253	2.115	1.814
α (kPa ⁻¹)	0.0005	0.4051	0.402	0.0007	0.0024	0.0006	0.0016
S _r	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
e	0.53	0.80	0.61	0.80	0.72	0.53	0.73
P ₂₀₀ (%)	47	0	0	98	100	55	58
C _u	50.00	1.79	1.84	23.33	5.00	64.29	20.77
C _c	0.81	0.83	0.89	0.80	0.288	1.14	0.26

(a) Data from Win (2006)

(b) Data from Kim and Sture (2008)

(c) Data from Jindal et al. (2016)

Figure 7-10: Validation of proposed tensile strength model for coarse-grained soils

(a) Data from Zeh and Witt (2005)

(b) Data from Wong et al. (2017)

(c) Data from Murray and Tarantino (2019)

Figure 7-11: Validation of proposed tensile strength model for fine-grained soils

Figure 7-12: Validation of proposed tensile strength model for the current study

Table 7-9: Comparison of RMSE of the validation data for proposed, Lu et al. (2009) and Yin and Vanapalli (2018) tensile strength models.

Reference	Proposed	Lu at al. (2009)	Yin and Vanapalli (2018)
Win (2006)	10.16	862.90	673.84
Kim and Sture (2008)	0.24	0.46	0.42
Jindal et al. (2016)	0.11	1.40	0.95
Zeh and Witt (2005)	119.36	1394.45	509.37
Wong et al. (2017)	6.65	113.68	217.56
Murray and Tarantino (2019)	89.03	243.43	235.09
Current study (2020)	6.80	416.70	282.75

7.3.3: Summary

Using the data collated from the literature, a new tensile strength model was developed for both unsaturated coarse-grained and fine-grained soils. The proposed model uses the Brazilian tensile test Mohr Circle and the extended Mohr-Coulomb criterion. Parameters of the suction stress in the proposed model were related to the grain size distribution and soil-water characteristic curve. Using a validation data set of three coarse-grained soils and four fine-

grained soils, the proposed tensile strength model was shown to perform better than existing tensile strength models.

7.4: Concluding remarks

This chapter has investigated the tensile strength of compacted soils. The effects of soil suction, structure and void ratio on the strength of statically compacted soils have been examined. Two tensile strength models were also evaluated using data from the literature and a new tensile strength model for coarse and fine-grained soils was developed. The main findings are:

- 1) The method of compaction (static or dynamic) does not affect the trend of the tensile strength of unsaturated soils.
- 2) The tensile strength of soil increases as the soil dries to a maximum value and remains constant then drops at very low degrees of saturation, possibly due to shrinkage cracks.
- 3) The shape of the variation of BTS with compaction water content is similar to the compaction curve.
- 4) The tensile strength of specimens compacted at the wet of optimum and dried are higher than those of as compacted specimens. This is attributed to the influence of soil structure on the tensile strength of the soil.
- 5) Soil strength is influenced by an interaction of soil structure, void ratio and matric suction. For dry soils, there is minimal influence on the strength from matric suction. In other terms, as the soil dries, matric suction is more significant up to the air entry value beyond which, its contribution towards further strength increase is minimal.
- 6) The two tensile strength models evaluated were not able to capture the tensile strength of the compacted soils. A new tensile strength model has been developed for both coarse and fine-grained soils and the proposed tensile strength model was shown to perform better than existing tensile strength models.

Chapter 8: Conclusion and Recommendations

The main motivation for this study has been exploring simple, economical and accessible means of obtaining unsaturated soil properties with the purpose to apply unsaturated soil mechanics in practice. This study focused on suction (determination and control) and strength of compacted and reconstituted soils. The conclusion and recommendations for this study are summarised below.

8.1: Conclusion

8.1.1: Suction measurements

The filter paper technique has been the main focus of suction measurement because it is economical, easily accessible, easy to use, and can measure total or matric suction over a wider suction range. Both the initially dry and initially wet filter papers in contact and non-contact modes were studied. The study concluded that:

- 1) If a strict protocol is observed, the filter paper method is still very useful for suction measurement.
- 2) It has been demonstrated that the limitation of measuring moisture content of the filter paper can be overcome by measuring electrical resistivity of the filter paper to infer the suction. As electrical resistivity is affected by the salinity of the water, this can be only applied to the non-contact filter paper method.
- 3) It was found that the equilibration time for initially wet filter paper was within the same ranges with the initially dry filter paper showing slightly lower equilibration time at high suctions. However, the equilibrium time at high suctions requires more experimental data to support the observations in this study.
- 4) There is good agreement between suction measured using the filter paper method on the as-compacted soil specimens with suction estimated from the corresponding SWCC of the soil specimen. A similar observation was made with suction measurements using the WP4, the recommended benchmark for suction measurements greater than 1000 kPa.
- 5) No contamination of filter paper was observed in this study as there was a negligible difference in the dry masses of the contact and non-contact filter papers after use. Therefore, sacrificial filter papers in the contact method may not be needed.

8.1.2: Suction control

The osmotic technique of controlling matric suction was evaluated. Furthermore, the volume change of soil that occurs due to a difference in salt concentration in the pore water between

different zones of the soil, termed as osmotic consolidation, has also been investigated in this study. The following conclusions can be made:

- 1) Osmotic technique is still valuable in controlling matric suction of soil if a correct combination of the salt solution and membrane is used. In this study, a reverse osmosis membrane and sodium chloride solutions was presented as the alternative to cellulosic membrane and polyethylene glycol solutions which suffer from cellulosic membrane degradation and molecular breakdown, respectively, during long-duration tests.
- 2) Higher matric suctions can be more easily achieved with the osmotic technique than the axis-translation technique to induce matric suction in a soil specimen within a reasonably shorter time. The osmotic technique is easy to implement and cheap as compared to other methods of matric suction control.
- 3) Soil-water characteristic curve of a soil specimen under constant normal stress can be determined using the oedometer set-up with the osmotic technique when a volume or mass measurement device is added to the set-up to measure the water inflow or outflow from the soil specimen.
- 4) While both the osmotic technique and osmotic consolidation involve the use of a salt solution, different types of suctions are applied to the specimen. It can be observed that a more definite deformation or volume change is observed when a matric suction is applied to a specimen as compared to an osmotic suction. Since soil strength is related to volume changes, matric suction plays a bigger role than osmotic suction towards the strength development in unsaturated soils.

8.1.3: Strength measurements

There is a relationship between matric suction, soil structure and void ratio in influencing soil strength. One key observation is the fact that as soils dry, the influence of matric suction on strength becomes less past the air-entry value. As the water phase progressively becomes discontinuous, the incremental matric suction contribution to additional strength increase becomes negligible.

Constant suction test is easier to analyse compared to constant water content test as both net normal and suction stresses are controlled in a constant suction test. Hence, analysis of constant suction direct shear tests is first relooked, and a new shear strength equation is proposed. A conventional direct shear test apparatus was used to conduct constant water content tests for as-compacted soil specimens as well as on the saturated soil specimens. The main findings are listed below.

8.1.3.1: Constant suction direct shear tests

Constant-suction direct shear test for unsaturated soils was reviewed. Data from the literature were collated and re-analysed resulting in the development of a new shear strength equation.

The following conclusions are made:

- 1) Interpreting the constant-suction direct shear tests by plotting the stress path where both net normal and shear stresses are corrected for the shear area is useful to identify the failure line especially in cases where there is no peak or maximum shear stress with shear displacement.
- 2) It was shown that it is more correct to plot corrected shear stress versus corrected net normal stress for constant-suction direct shear tests with a failure line at a gradient of $\tan \phi'$. For each matric suction, a higher net normal stress can cause the void ratio of a soil to decrease and hence ϕ' to increase.
- 3) Based on the extended Mohr-Coulomb extended envelope, a new shear strength equation using suction stress incorporating parameters from soil-water characteristic curve and grain size distribution to account for the contribution of matric suction to the shear strength of unsaturated soils was proposed.

8.1.3.2: Constant water content direct shear tests

Constant water content direct shear tests have been conducted on compacted soils and compacted soils dried from wet of optimum condition. The following conclusions are made:

- 1) Stress history, suction, void ratio, strain rate and structure have a significant influence on the shear strength behaviour of unsaturated soils.
- 2) Shear strength of compacted soils can be determined using constant water content direct shear tests and interpreted using the extended Mohr-Coulomb failure criterion for unsaturated soils.
- 3) Similar to constant suction direct shear tests, the contribution of matric suction to shear strength can be expressed in terms of suction stress but usually, the soil-water characteristic curve is unavailable when constant water content direct shear test was reported. In this case, suction stress is expressed as a product of an exponential function of degree of saturation and matric suction.
- 4) It has been demonstrated that using the proposed suction stress for constant water content direct shear tests, the matric suction of compacted soil can be estimated and good agreement was obtained when the estimated matric suctions were compared with the soil-water characteristic curves of the soils.

- 5) Shear strength of compacted soil can be estimated if the initial matric suction of the compacted soil is known.
- 6) Shear strength of as-compacted soils is lower than the shear strength of compacted soils dried from the wet of optimum and hence are useful for determining shear strength necessary for conservative design in unsaturated soils.

8.1.3.3: Tensile strength tests and model

Indirect tensile strength tests were conducted using the Brazilian tensile strength tests on compacted soils. As-compacted soil specimens and soil specimens initially compacted wet of optimum and allowed to dry to various water contents were tested. The study also reviewed the tensile strength models proposed for unsaturated soils. The following conclusions are made:

- 1) Method of compaction (static or dynamic) does not affect the tensile strength of unsaturated soils.
- 2) Tensile strength of soil increases as the soil dries to a degree of saturation near its air-entry value and remains constant then drops at very low degrees of saturation.
- 3) The shape of the variation of tensile strength with the compaction water content curve is similar to the compaction curve.
- 4) Tensile strength of specimens compacted at the wet of optimum and dried are higher than those of as-compacted specimens. This explains the influence of soil structure on the strength of the soil.
- 5) For dry soils, there is minimal influence on the strength from matric suction. In other words, as the soil dries, the contribution of matric suction to shear strength is more significant up to the air-entry value, beyond which its contribution towards the tensile strength is minimal.
- 6) Two recent tensile strength models based on the macroscopic approach incorporating SWCC for unsaturated soils were evaluated. Both were not able to model the tensile strength for a wide range of soil types. A new tensile strength model has been developed for both coarse and fine-grained soils and the proposed tensile strength model was shown to perform better than existing tensile strength models.

8.2: Recommendations

Due to the limited timeframe, not all interesting, related research were explored. Thus, the following recommendations are presented for future study:

- 1) Progress has been made on the use of filter paper method in laboratory measurements of soil suction. The sensitivity of the filter paper measurements using electrical resistivity can be improved if the filter paper is treated with an ionic solution such as a diluted hydrochloric acid.
- 2) Further research is recommended in investigating the equilibrium time of initially wet filter papers especially at high suctions.
- 3) In the osmotic technique, other salt solutions, such as potassium chloride and sodium sulphate, can also be investigated as well as other commercial forward osmosis and reverse osmosis membranes to see if additional benefits can be derived. The use of other types of membranes (e.g., a micro-filtration membrane) with sodium chloride solution can be investigated in further studies.
- 4) The use of constant water content test in obtaining unsaturated shear strength properties is very promising and requires further investigation on different soils, especially at high matric suctions. Measurements such as suction changes and volume changes during shearing are necessary to validate the observations and assumptions made.
- 5) The influence of strain rate and net confining/normal stress on soils at a high degree of saturation in constant water content direct shear tests needs to be studied further.
- 6) The proposed shear strength models for constant suction and constant water content direct shear tests should be validated with more soils.
- 7) Other types of tensile tests for unsaturated soils should be evaluated and compared with the Brazilian tensile tests as it is known that there are differences in the tensile strength amongst the various tensile tests.
- 8) The proposed tensile strength model should be validated with more soils.
- 9) Applicability of the form of the suction stress proposed in this thesis should be evaluated for constant suction and constant water content triaxial tests.

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Appendices

Appendix A-Water content and suctions for compacted soil specimens

Appendix Table 1: Water contents and suctions of dynamically compacted soil specimens

Compaction moisture content	Contact FP	Non-contact FP	Matric suction	Average	Total Suction	Average
26	50.60	50.18	56.28	58.47	0.00	0.03
	51.64	50.18	53.28		0.00	
	46.71	45.30	63.30		0.05	
	49.07	45.30	61.01		0.05	
24	44.11	43.50	94.71	92.40	0.13	0.13
	44.52	43.50	88.92		0.13	
	44.04	43.61	95.74		0.12	
	44.42	43.61	90.23		0.12	
22	40.42	39.97	167.67	168.50	0.80	1.04
	40.32	39.97	170.39		0.80	
	40.45	39.07	166.91		1.27	
	40.37	39.07	169.01		1.27	
21	38.30	38.25	233.11	223.21	1.94	2.12
	38.86	38.25	213.77		1.94	
	38.79	37.92	215.96		2.29	
	38.38	37.92	230.02		2.29	
20	37.09	36.88	280.95	272.04	3.89	5.21
	37.56	36.88	261.19		3.89	
	37.03	35.87	283.77		6.54	
	37.54	35.87	262.25		6.54	
18	33.36	32.87	501.24	506.08	30.19	40.00
	33.16	32.87	516.73		30.19	
	33.28	31.89	506.91		49.82	
	33.38	31.89	499.42		49.82	
16	30.82	30.76	742.14	701.54	89.06	120.62
	31.28	30.76	691.87		89.06	
	31.73	29.71	644.94		152.18	
	30.96	29.71	727.23		152.18	
14	27.11	25.95	1320.00	1421.63	1069.18	1286.62
	26.05	25.95	1555.88		1069.18	
	26.64	24.26	1418.96		1504.06	
	26.77	24.26	1391.67		1504.06	

Appendix Table 2: Water contents and suctions of standard Proctor dynamically compacted soil specimens.

Compaction water content	FP condition	Water content			Suction			Average Suction
10	Contact	25.00	26.14	24.30	1829	1533	2039	
		24.89	24.78	24.32	1861	1893	2035	1864.96
	Non-contact	24.26	23.91	24.18	2054	2166	2079	2099.72*
					1506	1615	1530	1550.49
13.5	Contact	28.02	27.16	27.50	1147	1309	1242	
		27.89	27.81	27.61	1169	1185	1221	1212.07
	Non-contact	26.86	27.29	27.42	1373	1283	1258	1304.80*
					655	524	491	556.67
15.3	Contact	32.79	32.81	27.50	547	545		
		32.69	33.52	27.61	556	488		534.23
	Non-contact	31.94	32.84	27.42	625	543		583.77*
					49	31		39.72
17	Contact	38.56	37.93	39.13	224	247	205	
		38.56	37.46	37.60	224	266	260	237.51
	Non-contact	37.87	37.30	38.68	249	272	220	246.96*
					2	3	2	2.35
17.3	Contact	38.21	39.06	39.18	236	207	203	
		38.14	39.25	38.28	239	201	234	220.05
	Non-contact	37.74	38.71	38.53	254	219	225	232.54*
					3	2	2	1.90
20.4	Contact	45.70	45.08	44.59	74	81	88	
		44.24	45.61	45.50	93	75	76	81.29
	Non-contact	43.82	44.01	44.33	99	96	92	95.62*
					0	0	0	0.10
24	Contact	57.57	57.95	56.49	39	38	41	
		57.02	57.61	57.47	40	39	39	39.43
	Non-contact	55.82	54.30	54.13	43	46	47	45.24*
					0	0	0	0.00
25.7	Contact	69.52	70.52	69.08	21	20	21	
		70.28	69.71	71.59	20	21	19	20.12
	Non-contact	67.50	67.73	68.53	23	23	22	22.58*
					0	0	0	0.00

* - Suctions computed using matric suction equation (Equation 4.1).

Appendix B: Shear stress – horizontal displacement curves for statically and dynamically compacted saturated specimens

(a) Compaction water content 26%

(b) Compaction water content 22%

(c) Compaction water content 18%

(d) Compaction water content 14%

Appendix Figure 1: Shear stress and vertical displacement versus shear displacement for dynamically compacted soil specimens.

Appendix C: Shear stress – horizontal displacement curves for statically and dynamically compacted unsaturated specimens

(a) Compaction water content 24%

(b) Compaction water content 22%

(c) Compaction water content 20%

(d) Compaction water content 18%

Appendix Figure 2: Shear stress and vertical displacement versus shear displacement for dynamically compacted soil specimens.

(a) Compaction water content 19%

(b) Compaction water content 22.7%

(c) Compaction water content 19%

(d) Compaction water content 22.7%

Appendix Figure 3: Shear stress and vertical displacement versus shear displacement for statically compacted soil specimens.