



Central University of Technology, Free State

CHARACTERISATION OF SHEAR STRESS PROPERTIES OF COMPACTED, UNSATURATED HEAVING SOILS

by

ARMAND AUGUSTIN FONDJO TAKOUKAM

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Promoter: Prof. Elizabeth Theron

Co-promoter: Prof. Richard P. Ray

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DECLARATION

I, Armand Augustin Fondjo Takoukam, I.D. No. ##### hereby declare that the thesis titled '**CHARACTERISATION OF SHEAR STRESS PROPERTIES OF COMPACTED, UNSATURATED HEAVING SOILS**' submitted for a Doctoral degree of Engineering in Civil Engineering with specialisation in Geotechnical Engineering to the Central University of Technology, Free State, is my independent research work and has not been submitted to another University or Faculty to obtain a degree.

Armand Augustin Fondjo Takoukam

Signature

Date: June 2024

Bloemfontein, South Africa

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ABSTRACT

The shear strength of soil is a fundamental property that governs the stability of soil material under structural loads. As such, it is required to determine the shear strength parameters and stiffness of soil support for foundation design. The literature review revealed that research on the influence of swelling on the shear strength of expansive soils is limited. The swelling stress influence on the shear strength of expansive soils is generally ignored in engineering practice. The measurement of unsaturated shear strength is challenging and time-consuming. Geotechnical predictive models can be utilised as an alternative to assess the geotechnical properties of these soils.

The formulation of a predictive shear strength constitutive relation and associated models for compacted unsaturated expansive soils were summarised in this thesis. The geotechnical properties of soil samples were determined using laboratory tests. The tested soils portray a range of liquid limits from 40 % to 79 %, plastic limits from 19 % to 57 %, a free swell index ranging from 36 % to 133 %, and a free swell ratio range varying between 1.17 to 2.50. A total suction ranging from 110 kPa to 11283 kPa, a matric suction of 80 kPa to 8745 kPa, and an air entry value ranging from 6 kPa to 20 kPa. The reported unsaturated shear strength (τ_u) values range from 119 kPa to 1512 kPa, while saturated shear strength (τ_s) values range from 94 kPa to 941 kPa. The τ_u values are 1.1 to 1.78 times higher than the τ_s . The friction angle values ϕ' range from 22° to 53°, the angle related to matric suction ϕ^b ranges from 1.41° to 10°, the angle indicating the rate of reduction in shear strength with respect to a variation of swell stress ϕ^s ranges from 16.96° to 69°, and the cohesion (c') ranges from 35 kPa to 78 kPa.

The findings indicated that soils BFS, WIS, and WES belong to the high plastic clay (CH) category, while BES falls under the low plastic clay (CL) category. The X-ray diffraction test quantified the mineralogical constituents controlling the swelling behaviour of identified soils and validated the soil classification tests. The SWRC Fit (Soil water retention curves software) was used to establish the soil-water characteristic curves (SWCCs) of tested soils based on models by Seki (BL), Durner (DB), Van Genuchten (VG), Fredlund and Xing (FX), and Kosugi (LN). Among the models, BL and DB portray the best-fitting SWCCs. Multivariate regression analysis was performed using NCSS 12 (Statistical analysis software for researchers) on data obtained from 60 tested soil specimens to design the geotechnical predictive models.

The modified-unsaturated shear strength (τ_{mu}) was established from the equilibrium of three stress variables: the effective normal stress ($\sigma_n - u_a$), matric suction ($u_a - u_w$), and swelling stress (P_s). The reported τ_{mu} values fall within the range of 31 kPa to 1162 kPa at the dry optimum. Nonetheless, τ_{mu} exhibits marginal values beyond the dry optimum. The assessment of τ_{mu} was conducted by comparing τ_{mu} with three precedent constitutive relations of τ_u established by Fredlund and Rahardjo (1993), Garven and Vanapalli (2006), and Bulolo and Leong (2019). Previous unsaturated shear strength constitutive relations may overestimate the shear strength of compacted unsaturated expansive soils because the stress state induced by the swelling stress within these soils is ignored in design.

A novel approach, applicable to unsaturated compacted expansive soils, was established by bridging classical soil mechanics (CSM) and unsaturated soil mechanics (USM) through mathematical relations. This approach aims to determine the unsaturated shear strength characteristics using basic geotechnical index properties. Geotechnical predictive models were developed for soil mineralogy, soil suction, compaction features, and unsaturated shear strength parameters. The predicted values were found to exhibit an acceptable level of accuracy.

Lastly, the research work clearly illustrates that the modified-unsaturated shear strength constitutive relation proposed in this study is an efficient tool to characterise the shear strength of expansive unsaturated soils. The influence of the swelling stress on the shear strength of expansive soils should not be overlooked in design. Taking into account the influence of swelling stress on shear strength of expansive soils can enhance the durability of constructions. The geotechnical predictive models designed in this study can predict the geotechnical parameters and reduce the cost and time required for laboratory tests. The developments explored in this research project contribute to the advances in unsaturated soil mechanics and design enhancement.

Keywords: Shear strength, unsaturated shear strength, modified-unsaturated shear strength, stress state variables, expansive soils, soil suction, swelling stress, soil water characteristic curve, geotechnical index properties, geotechnical predictive models, multivariate-regression analysis.

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LIST OF ABBREVIATIONS

AEV	Air-entry value
AIC	Akaike information criterion
ASCE	American Society of Civil Engineering
ASTM	American Society for Testing and Material
ATT	Axis- translation technique
BES	Bethlehem soil
BFS	Bloemfontein soil
BL	Seki model
CD	Consolidated drained test
CH	High plastic clay
CL	Low plastic clay
CMH	Chilled mirror hygrometer
CSIR	Center of Scientific Innovation and Research
CSM	Classical soil mechanics
CU	Consolidated undrained test
DB	Wolfgang Durner model
ES	Expansive soils
FP	Filter paper
FPT	Filter paper technique
FSI	Free swell index
FSR	Free swell ratio
FX	Fredlund and Xing Model
GPM	Geotechnical predictive model
HAE	High air entry
LL	Liquid limit
LN	Kosugi model
LS	Linear shrinkage
MCFE	Mohr-Coulomb failure envelope
MRA	Multivariate regression analysis
PI	Plasticity index
PSD	Particle size distribution
RDW	Relative density of water
RHS	Relative humidity sensor
SANS	South African National Standards
SP	Swelling potential
SRP	Standardised residual pattern
SWCC	Soil water characteristic curve
TDR	Time-domain reflectometry
USCS	Unified soil classification system
USM	Unsaturated soil mechanics
UU	Unconsolidated undrained test
VG	Van Genuchten Model

WES	Welkom Soil
WIS	Winburg Soil
XRD	X-ray diffraction
ZST	Zero swelling test

NOTATIONS AND SYMBOLS

Roman letters

$c''_{\max}$	Maximum capillary cohesion
C_0	Velocity of light in vacuum
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
c'	Effective cohesion of soil material
c_t	Total cohesion of soil material
a	Soil parameter related to the air entry of the soil
A	Long term attractive stress
a_c	Effective area of interparticle contacts
C	Correction factor
e	Natural constant 2.718
e_c	Unit electron charge
g	Grams or Gravity (9.81 m. s^{-2})
G_s	Specific gravity
h_c	Capillary height
h_s	Osmotic head
K	Boltzmann's constant
k	Soil water characteristic fitting parameter
k_o	Number of estimated parameter
K_o	Stiffness of soil
L_r	Beam ratio
m	Number of relevant soil parameter
M	Total mass
m_1	Mass of wet filter paper + cold tare
M_1	Empty mass of volumetric flask
m_2	Mass of wet filter paper + hot tare mass
M_2	Mass of pycnometer + oven dry soil
M_3	Mass of pycnometer + oven dry soil+ filled water
M_4	Mass of pycnometer + filled with water only
M_a	Mass of air
M_c	Mass of the contractile skin
m_f	Soil parameter related to the residual water content condition
M_f	Mass of dry filter paper
m_i	Dimensionless curve shape parameters
M_i	Unit mass of surcharge
M_m	Mass of mould and base plate
M_s	Mass of solids
M_{soil}	Mass of dry soil
M_t	Mass of mould, base plate, and wet soil
M_v	Mass of water in the filter paper

M_w	Mass of water
M_w	Mass of water to be added
N	Number of blows
n	Number of surcharges
n_f	Soil parameter related to the rate of desaturation
n_i	Dimensionless curve shape parameters
P	Actual vapour pressure of air in equilibrium with liquid phase
P_o	Saturation vapour pressure
P_s	Swelling stress
$Q(x)$	Complementary cumulative normal distribution function
R	Radius of capillary tube
R_1, R_2	Radius of curvature of warped membrane
R^2	Correlation coefficient
R_d	Relative density of water according to temperature
R_s	Sheath radius of curvature/Radius of curvature of the meniscus
R_T	Universal gas constant
S	Degree of saturation
S_e	Effective saturation
t	Two layers thicknesses
T	Temperature
T_c	Cold tare mass
T_h	Hot tare mass
T_k	Absolute temperature
T_s	Tension surface
u	Hydrostatic pressure
U_a	Pore air pressure
U_w	Pore water pressure
V	Electromagnetic wave velocity in the soil
V	Total volume
V_a	Volume of air
V_c	Volume of contractile skin
V_d	Volume of soil specimen read from the graduated cylinder containing distilled water.
V_f	Final volume of specimen
V_i	Initial volume of specimen
V_k	Volume of soil specimen read from the graduated cylinder containing Kerosene
V_m	Volume of mould
V_s	Volume of solids
V_w	Volume of water
W	Moisture content
W_1	Mass of container + wet soil
W_2	Mass of container + wet soil

W_c	Mass of container
W_f	Water content of filter paper
W_i	Initial water content
OMC	Optimum moisture content
W_t	Targeted moisture content
Y	Outcome variable
Z	Elevation difference between the piezometer and the point being referred
Z_i	Covariates, $i = 1, 2, \dots, n$

Greek letters

$\bar{u}_v$	Partial pressure of pore
$\bar{u}_{vo}$	Saturation pressure of water steam over a flat surface of pure water at the same temperature
σ_i	Standard deviation of distribution
χ_m	Effective stress parameter for matric suction
χ_{max}	Bishop parameter of maximum average skeleton
χ_s	Effective stress parameter for osmotic suction
α_1	Angle of contact
α_i	Scaling factor that determines the position of max pore sizes
γ_d	Dry density
γ_{dmax}	Maximum dry density
$\frac{\delta(\gamma_d)}{\delta w}$	Differential function
δ_{ij}	Kroneker delta or substitution tensor
ϵ_c	Dielectric constant medium
ϵ_s	Dielectric number of soil
ϵ_w	Water dielectric number of soil
θ_d	Dimensionless water content defines as θ/θ_s
θ_r	Residual volumetric water content
θ_s	Volumetric water at saturation
θ_ω	Volumetric water content
$\xi_i, \eta_i, \mu_i, \alpha_i$	Multi-regression analysis coefficients, $i = 1, 2, \dots, n$
$\xi_o, \eta_o, \mu_o, \alpha_o$	Intercepts
ρ_w	Density of water
σ'	Effective stress
σ_n	Total normal stress
σ_c	Confining pressure
σ_{ij}	Total stress tensor
σ_x	Total normal stress parallel to direction (x)
σ_y	Total normal stress parallel to direction (y)
σ_z	Total normal stress parallel to direction (z)
τ_{mu}	Modified-unsaturated shear strength

τ_s	Saturated shear strength
τ_u	Unsaturated shear strength
τ_{xy}	Shear strength on the plan (y,z), perpendicular to direction (x)
τ_{xz}	Shear strength on the plan (y,z), perpendicular to direction (x)
τ_{yx}	Shear strength on the plan (x,z), perpendicular to direction (y)
τ_{yz}	Shear strength on the plan (x,z), perpendicular to direction (y)
τ_{zx}	Shear strength on the plan (x,y), perpendicular to direction (z)
τ_{zy}	Shear strength on the plan (x,y), perpendicular to direction (z)
ψ_{m_i}	Soil fitting parameters
ψ_m	Matric suction
ψ_o	Osmotic suction
ψ_t	Total soil suction
ω_1	Weighting factor for the sub curve
ϕ'	Angle of internal friction of effective normal stress
ϕ^b	Angle indicating the rate of increase in shear strength with respect to a change in matric suction
ϕ^s	Angle indicating the rate of reduction in shear strength with respect to a change in swelling stress
ΔU	Difference in stress on a two-dimension curved arc
ΔV	Initial change in volume of a specimen
β	Angle between the tension surface and horizontal
γ	Bulk dry unit weight
δ	Swelling stress fitting parameter
ε	Random error representing the discrepancies in the approximation
η	Electrolyte concentration
θ	Volumetric water content
λ	Box-cox transformation
ν	Cation valence
σ	Normal stress
φ	Internal diameter of the consolidation ring
χ	Bishop parameters
ψ	Soil suction
$\phi(x)$	Normalised form of the cumulative normal distribution function

CHAPTER 1: INTRODUCTION

1.1 Overview

Lightweight constructions are more vulnerable to damages initiated by the movements of expansive soils because lightweight constructions are inadequate to control the differential movements and the uplifting forces. Other countries reporting expansive soils problems include Morocco, Mexico, Israel, Spain, Turkey, Iran, India, Great Britain, Ethiopia, Ghana, Australia, USA, and Argentina. In general, the more arid the climate, the more severe is the swelling soil problem. These soils present significant structural and geotechnical challenges worldwide. Figure 1.1 shows the reported expansive soils zones worldwide. Expansive soils are highly plastic materials containing a relevant fraction of clay and silt and are very sensitive to moisture change. Repair works of construction damages caused by expansive soils were evaluated at R100 million annually in South Africa (Williams *et al.* 1985). The repair cost was estimated at £400 million per year in the United Kingdom (Driscoll and Crilly, 2000). The American Society of Civil Engineers (ASCE) estimated that 25 % of houses exhibit some damages caused by expansive soils in the US (Lee and Ian, 2012). Expansive soils induce more financial losses in a year than hurricanes, floods, earthquakes, and tornadoes combined (Nelson and Miller, 1992).

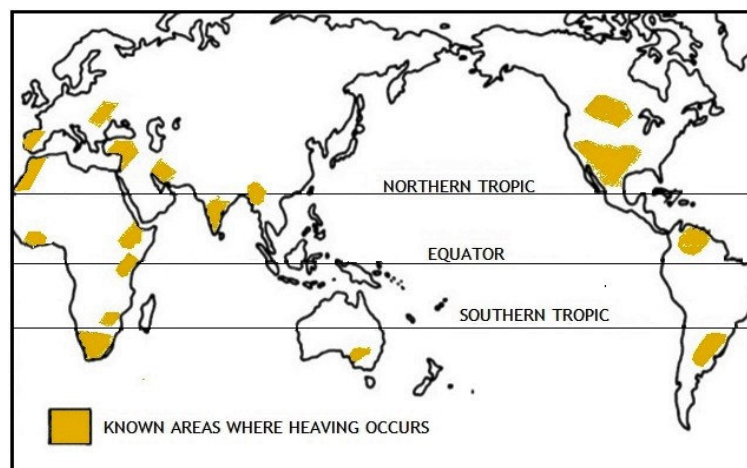


Figure 1.1: Distribution of reported expansive soils zones (Chen, 1975).

The success in the practice of soil mechanics can be mainly attributed to the ability of Geotechnical Engineers to describe soil behaviour for stress state conditions in saturated soils. However, the implementation of unsaturated soil mechanics (USM) has lagged behind. This is not necessarily the case when dealing with the problem

associated with expansive soils. USM has been extensively studied worldwide, especially in areas where mean evaporation exceeds mean precipitation yearly, and in regions that experiences seasonal moisture deficiencies (Fredlund and Rahardjo 1993, Fredlund et al. 2012, Blight, 2013, Fondjo et al. 2021a, Fondjo et al. 2023). Considering that over 33 % of the world's areas are located in semi-arid climates, USM becomes a fundamental discipline in geotechnical engineering (Barbour, 1998). The problems of expansive soils can be addressed utilising the concepts of USM, as their hydro-mechanical behaviour is significantly governed by soil suction. The foundations of lightweight structures are often situated above the groundwater table, in a state of unsaturated conditions. Therefore, predicting the unsaturated shear strength of expansive soils using USM concepts becomes a practical approach. The fundamental geotechnical property of soil material is the shear strength. The design of engineered structures relies on the shear strength and stiffness of the soil support. Bearing limit and slope stability are examples of geotechnical applications related to the shear strength of the soil. This is commonly demonstrated by building codes through a two-tier design philosophy: (1) Ultimate conditions or strength design, and (2) Serviceability conditions or stiffness design. In accordance with classical soil mechanics (CSM), the capacity of foundation support consists of the soil's shear strength (friction and cohesion) and the kinematic action of the support system (failure mechanism). An additional, vital concept is effective stress, where soil strength depends on confinement of the soil skeleton or matrix. The presence of water (saturation) generates positive pore pressures, reducing the confining stress in the soil matrix, thereby reducing strength. The action of pore pressures is often the cause of bearing capacity, and slope stability failures since effective strength is reduced and loading actions are increased. USM examines the contribution of negative pore pressures within the soil matrix. Soil strength and stiffness are influenced in more subtle ways when unsaturated conditions are present. Punmia (2011) stated that a detailed understanding of the interactions between pore water, soil strength and stiffness are required to accurately interpret the application of laboratory results to field conditions. Therefore, a thorough understanding of the physical properties, swelling parameters, and hydro-mechanical behaviour of expansive soils is essential to produce safe and economical geotechnical designs.

1.2 Problem statement

The expansive soils problem in South Africa is significant since, in several communities in Free State province such as Bloemfontein, Winburg, Welkom, Bethlehem, expansive soils have caused damage to lightweight construction (subsidised housing, pavements) resulting in economic losses and social upheaval (Fondjo, 2018). Highly expansive soils have been reported in Free State, KwaZulu-Natal, South Gauteng, North West province (Diop *et al.* 2011). Much of the problem can be attributed to an insufficient consideration of expansive soils behaviour. Soil and climate conditions in these areas contribute to shrink-swell behaviour, which occurs in cycles and unevenly distributed manner within the supporting subgrade. Consequently, this leads to cyclic, uneven, and vertical movements that inevitably damage structures. In engineering practice, soil supports are predominantly in unsaturated conditions, as foundations of lightweight construction are generally located above the groundwater table. The process of rainfall and evapo-transpiration exerts a strong influence on these soils. As rainfall infiltrates the soil, negative pore pressures decrease, allowing the soil matrix to expand, resulting in the lifting of the foundations immediately above. Subsequently, during evapo-transpiration throughout the seasons, pore water volume reduces, causing pore pressures to become more negative and inducing shrinkage. As a consequence, the supported foundations will then settle, leading to non-uniform movement beneath the building causing further differential movement.

The CSM approach fails to consider the significant effects of negative pore pressures. Therefore, a better approach to assess the behaviour of such soils is through USM. By doing so, one considers the influence of negative pore pressures on soil strength, stiffness, and volume change behaviour. Abeb and Vermeer (2009) investigated the behaviour of expansive soils using geotechnical modelling and found that the analysis of behaviour of expansive soils is more precise using USM.

Compaction at carefully controlled moisture content enhances the shear strength, bearing limit, and reduces the compressibility of soils that support buildings, roads, and earth dams. Therefore, a thorough understanding of the shear strength of compacted expansive soils related to the unsaturated soil parameters, soil index

properties, and swelling soil parameters leads to an efficient design, enhances the service life, and reduces the repair cost.

The extended Mohr-Columb failure envelope (MCFE) for unsaturated soils, proposed by Fredlund and Rahardjo (1993), was developed by adding the matric suction contribution to the original saturated soils MCFE initially developed by Terzaghi (1936). Nonetheless, when unsaturated soils exhibit swelling behaviour, the analysis of the impact of swelling stress on the shear strength characteristics of expansive soils becomes essential. Reproducing the mechanical behaviour of unsaturated soils is fundamental in soil mechanics. The challenge lies in the assessment of shear strength characteristics, and complex circumstances arise when the soil state is unsaturated and expansive. The principles that control the behaviour of expansive soils are difficult to apply directly to classical soil models. As a practical measure, correlations between the unsaturated moisture conditions and soil strength behaviour may be applied.

Blight (2013) reported several challenges related to the shear strength measurement for unsaturated soils. These challenges include the need for several tests to establish the strength variation with matric suction. Achieving matric suction equilibrium in soil specimens before testing requires a considerable amount of time, and specialised equipment is necessary for unsaturated soil testing. Due to the expense and complexity involved, researchers sometimes opt for a prediction approach, as it is simpler and less time-consuming. The axis-translation technique (ATT), applied effectively on triaxial cells for shear strength testing of unsaturated soils, exhibits a matric suction range of 0 to 1500 kPa. Rao and Revanasiddappa (2000) mentioned that in compacted soil clay specimens, the matric suction value ranges from 50 kPa to 8000 kPa at saturation values of 35 % to 90 %. In a similar study, Fondjo *et al.* (2021) reported that compacted expansive soils have a matric suction range of 40 kPa to 7700 kPa, and the total suction values range from 55 kPa to 10000 kPa. Designing geotechnical predictive models for unsaturated shear strength characteristics can be an efficient tool to address these challenges.

Significant research has been conducted on the shear strength properties of many soils. However, much of the research was focused on very specific soils and conditions, making it difficult to generalise into a more comprehensive approach.

Nevertheless, the contribution of the swelling stress to unsaturated shear strength should be investigated.

1.3 Research objective

1.3.1 General objective

The aims of this study is to formulate a predictive shear strength constitutive relation and associated models for compacted unsaturated expansive soils.

1.3.2 Specific objectives

The specific objectives are as follows:

- I. Characterise soil samples collected across the study area using particle size distribution analysis, consistency limits test, and the X-ray diffraction test to quantify the mineralogical constituents of identified soils.
- II. Evaluate the geotechnical properties of soil samples according to established protocols and standards testing. These tests include free swell index test, free swell ratio test, specific gravity test, modified Proctor compaction test, zero swell tests, soil suction measurement using filter paper technique, soil-water characteristic curves modelling, and advanced triaxial testing.
- III. Evaluate the compressive stress-strain behaviour, and the saturated shear strength of soil samples. Analyse the shear strength and matric suction relationship and investigate the relationship between the shear strength and swelling stress of identified soils.
- IV. Determine the unsaturated shear strength parameters of compacted soil samples and investigate their correlation with geotechnical index properties and soil mineralogy. Establish a modified-unsaturated shear strength constitutive relation for unsaturated expansive soils. Validate the modified-unsaturated shear strength relation and investigate the modified-unsaturated shear strength parameters' response to geotechnical index properties and soil mineralogy.

- V. Design geotechnical predictive models using NCSS 12 (Statistical analysis software for researchers). Compare the predicted values with laboratory-measured values. The models are designed for soil mineralogy, soil suction, compaction features, unsaturated shear strength, and modified-unsaturated shear strength.

1.4 Research scope

The research work includes an extensive and comprehensive literature survey on previous works related to the shear strength of unsaturated soils, soil suction estimation techniques, and shear strength measurement methods.

The research work is primarily based on laboratory investigations, which also includes geotechnical modelling as described in the general and specific objectives of the study.

Soil samples exhibiting various swelling clay fractions are selected to obtain different gradations and consistency limit values, encompassing a wide range of soil suction and swelling stress values. The shear strength parameters, swelling stress, and soil suction are evaluated at different moisture content.

Other problematic soils, such as dolomite, non-engineered fills, collapsible soils, organics/peat soils, and soluble ground, are not in the scope of this study.

1.5 Thesis layout

The thesis has been organised in the following manner:

Chapter One covers the introduction, problem statement, research objectives, research scope, and thesis layout.

Chapter Two focuses on the behaviour of unsaturated soils.

Chapter Three presents the previous studies on shear strength of unsaturated soils.

Chapter Four describes the shear strength and suction measurement of unsaturated soils.

Chapter Five describes the experimental research methods.

Chapter Six covers the discussion of geotechnical test results and modelling.

Chapter Seven covers the summary, conclusions and recommendations.

CHAPTER 2: BEHAVIOUR OF UNSATURATED SOILS

2.1 Overview

The origins of soil mechanics were empirical rules developed by builders and engineers based on the collective experience of craft guilds dating back centuries. The best examples of recent historical studies come from works by Coulomb (1776), Collin (1846), and Rankine (1857). It was attempted to support empirical rules with more rigorous examinations of how soil behaves as an engineering material. While the authors acknowledged the difficulty of generalising their studies, it was established an approach that was followed by later investigators. The earlier comprehensive treatment of soil mechanics by Terzaghi (1925) created a substantial starting point for engineers and researchers to advance the discipline. Analytical mechanics have successfully applied continuum mechanics to describe the response of soil to external stimuli of force and displacement. In saturated soil, the two phases are the mineral solid and the fluid (water). An unsaturated soil is a multiphase system comprising fluid (water), air, soil solids (particles), and contractile skin. Fredlund and Morgenstern (1977) reported that although there has been a suggestion of a fourth phase, the contractile skin, which is the interface between water and air, has a small volume. Fredlund and Rahardjo (1993) and Fredlund *et al.* (2012) mentioned that in unsaturated soil, the contractile skin can be ignored in terms of volume-mass relations, and its mass considered as a part of the mass of water. Although the concept of the fourth phase seems intuitive for stress analysis of unsaturated soil, there is no explicit consideration of the fourth phase in constitutive equations in literature. Figure 2.1 shows a visualisation aid for the generalised world of soil mechanics (Fredlund, 1996).

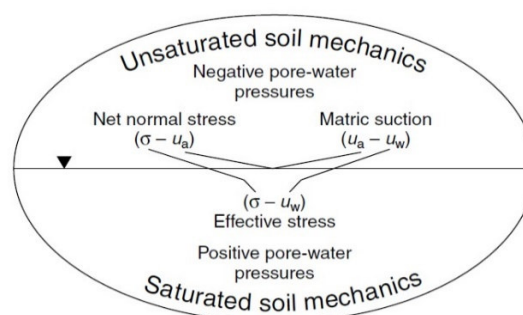


Figure 2.1: A visualisation aid for the generalised world of soil mechanics (Fredlund, 1996)

The justification for this research was brought up in arid and semi-arid regions where evaporation is more prominent than the yearly precipitation. This phenomenon leads to the water content variation profile at the soil surface in a characteristic depth called an active zone. Interparticle water in fine soils can deliver negative pore stress through matric suction, sorption, and double-layer attraction. The phenomenon leads to a more complex stress condition inside the soil matrix and significantly affects the stress-strain and the volume change behaviour. Soil materials utilised in construction are generally situated above the groundwater table and may experience negative pore pressure. Figure 2.2 shows the climatic categorisation of excessively arid and semi-arid spaces in the world. Approximately 33% of the world's region is recognised to be unsaturated (Dregne, 1976; Fredlund and Rahardjo, 1993).

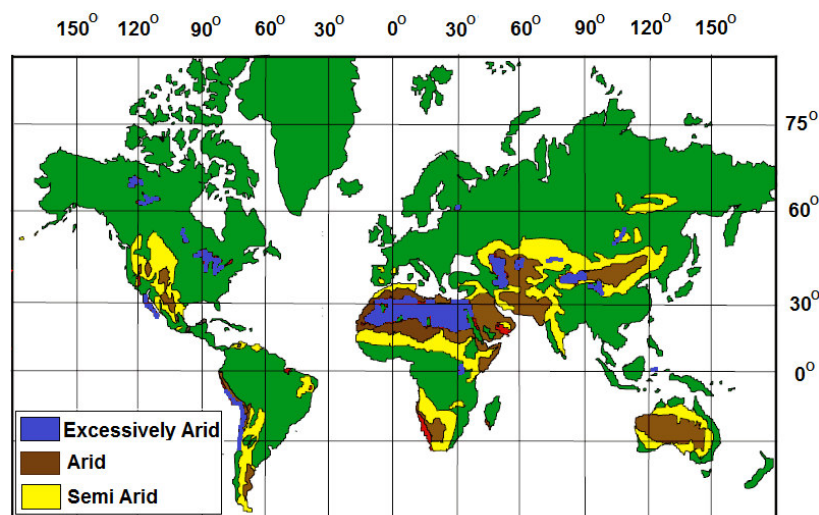


Figure 2.2: Excessively arid, and semi-arid regions of the world
(Dregne, 1976; Fredlund & Rahardjo, 1993)

Both saturated and unsaturated zones are affected by climatic factors, such as precipitation, transpiration, and evaporation. The differentiating feature of soil in an unsaturated zone is the contribution of soil suction. The static groundwater level (Figure 2.3) may be designated by the location where seasonal variations in moisture do not influence its presence. The actual groundwater level (zone of saturation) may vary seasonally due to changes in precipitation from above and drainage features (river channels, ponds, lakes.) from below. However, over a longer period of time (say, 50-100 years), this level remains saturated. The static level may be influenced by aquifer pumping as well as geomorphological events. The moisture within the zone above the static groundwater level constantly changes due to the interactions of

precipitation, infiltration, evapo-transpiration, and runoff. With reductions in moisture, negative pore pressures become increasingly negative. As moisture levels increase toward saturation, negative pore pressures become less negative, reaching zero as full saturation conditions are attained.

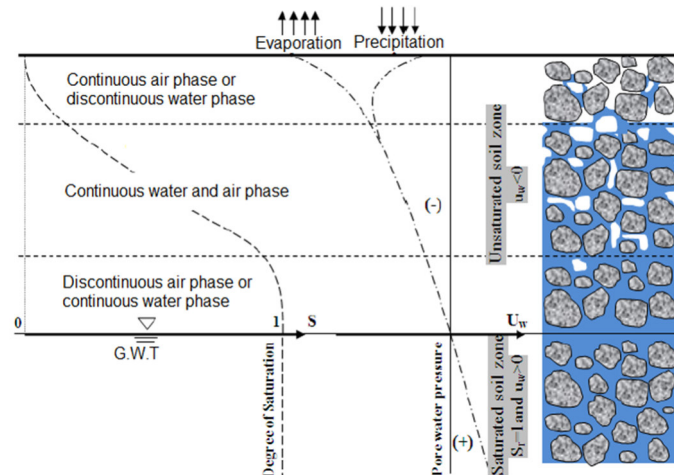


Figure 2.3: Unsaturated soil region

2.2 Nature and phase properties of unsaturated soils

2.2.1 Phases of unsaturated soils

Expansive soils are ordinarily found in unsaturated conditions due to moisture deficiency. In unsaturated soils, water in the void spaces is irregular, and the system consists of three phases: solid, pore water, and pore air, as presented in Figure 2.4.

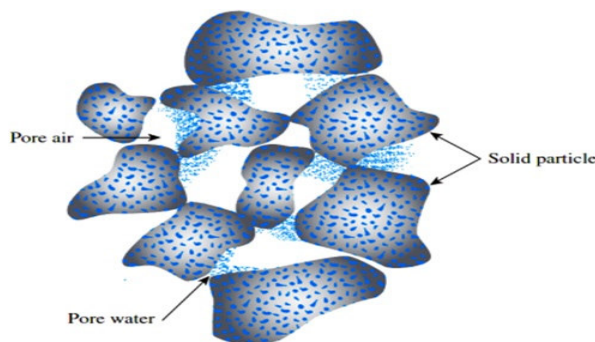


Figure 2.4: Unsaturated soils

As mentioned earlier, while the contractile skin phase is not a state variable in formal sense, it is worthy to recognise its usefulness in analysing unsaturated soil (Fredlund and Morgenstern, 1977). Thus, unsaturated soils may be described as a four-phase system to account for fundamental role of contractile skin on soil behaviour. The air-water interface is a thin layer interlaced between and within the voids of soils, creating

a fixed partition between the water phase and air phase. The variations of water content, shear strength, and volume are affected by the stress state variation of the contractile skin. Figure 2.5 shows a component of unsaturated soil with a continuous air phase and the fourth-phase unsaturated soil system.

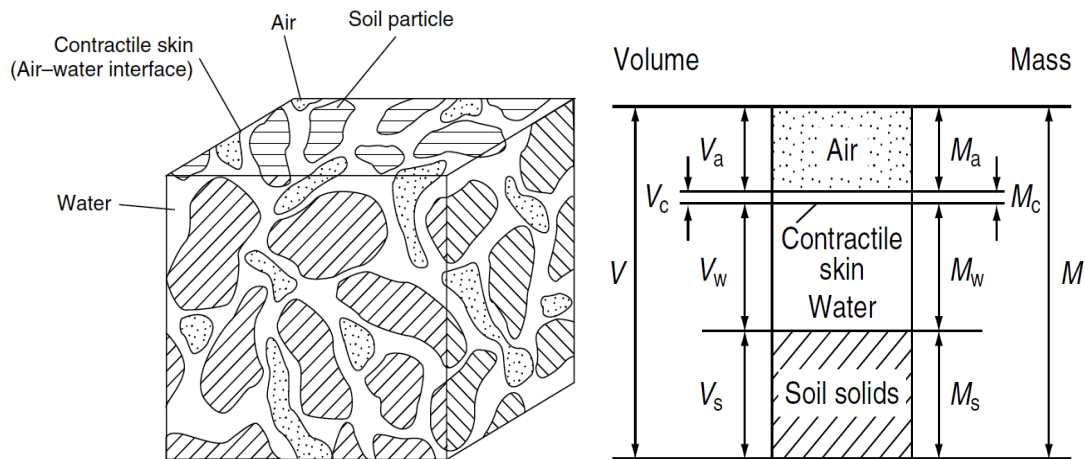


Figure 2.5: (a) An element of unsaturated soil with continuous air phase, (b) Four-phase system of an unsaturated soil (Fredlund and Rahardjo, 1993)

2.2.1.1 Contractile skin (Air-water interface)

The fundamental property of the air-water interface is its ability to exert a tensile action. It acts as if it is a flexible sheet joined between the whole structures of the solid soil matrix. Most of the contractile skin features appear to be different from that of continuous water phases (Davies and Rideal, 1963). Acknowledging the uniqueness of the air-water interface (fourth phase) helps to understand the state of stress variable for unsaturated soils (Fredlund and Morgenstern, 1997). Many investigations performed on the nature of the air-water interface point toward its essential and independent role on unsaturated soils (Wang and Fredlund, 2003). Recent research recommends that the thickness of the contractile skin ranges from 1.5 to 2 molecules of water in diameter ($5\text{\AA}$) (Israelachvili, 1991). Figure 2.6 shows that the water molecules' distribution over contractile skin appears as a hyperbolic tangent function.

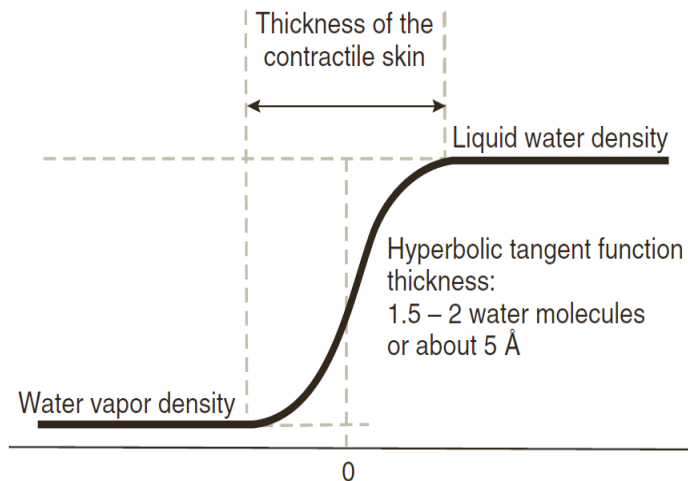


Figure 2.6: Density distribution over air-water interface (Kyklema, 2000)

2.2.1.2 Surface tension

The air-water interface exhibits a property called surface tension. The phenomenon of surface tension is a result of intermolecular forces acting on particles in the contractile skin. These actions are not equal to those forces exerted on molecules inside water shown in Figure 2.7a. The surface tension induces the water-air interface to act as a flexible membrane. The air-water interface acts like an inflated balloon with higher pressure inside than outside.

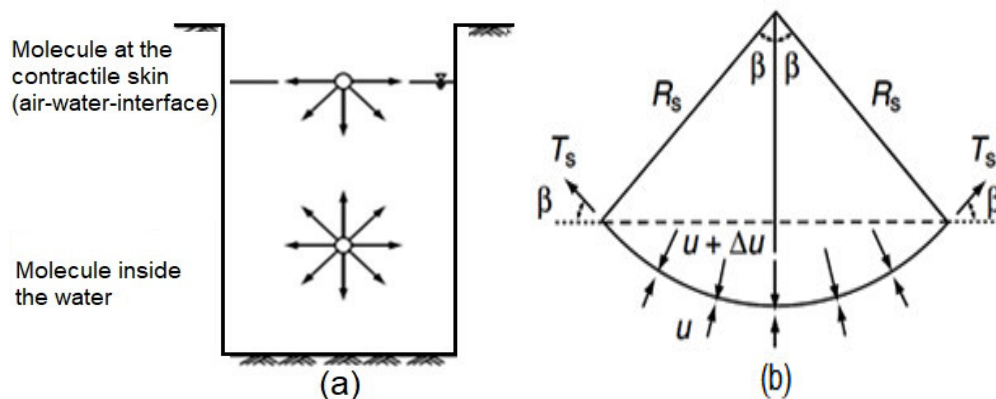


Figure 2.7: Surface tension phenomenon on contractile skin. (a) Intermolecular forces at the air-water interface and water. (b) Pressures and surface tension acting on a curved two-dimension surface (Fredlund and Rahardjo, 1993)

Figure 2.7b shows the pressure difference over the surface of the curve that can be correlated to the curved membrane radius and the surface tension. $U + \Delta U$ are the stresses acting on the membrane. R_s is the membrane radius of curvature, and T_s is the surface tension. Equation 2.1 gives the equilibrium in the vertical direction.

$$2 T_S \sin \beta = 2 \Delta U R_S \sin \beta \quad [2.1]$$

Where:

$2 \Delta U R_S \sin \beta$ = Length of the membrane projected onto a plane surface

Rearranging Equation 2.1, yields:

$$\Delta U = \frac{T_S}{R_S} \quad [2.2]$$

Equation 2.3 gives the difference in stress on a two-dimensional curved area with surface tension T_S and a radius R_S .

Table 2.1: Surface tension of contractile skin at several temperatures
(Kaye and Laby, 1973)

Temperature (°C)	Surface Tension, T_s (mN/m)
0	75.70
10	74.20
20	72.75
30	71.20
40	69.60
60	66.20
80	62.60
100	58.80

For a warped three-dimensional membrane, Equation 2.3 describes a two-dimensional membrane, and can be expanded using the Laplace membrane.

$$\Delta U = T_S \left(\frac{1}{R_1} + \frac{1}{R_2} \right) \quad [2.3]$$

Where:

R_1, R_2 = Radii of curvature of a warped membrane according to the
orthogonal principal planes [m]

T_S = Surface tension as illustrated in Figure 2.8 [$\text{n} \cdot \text{m}^{-1}$]

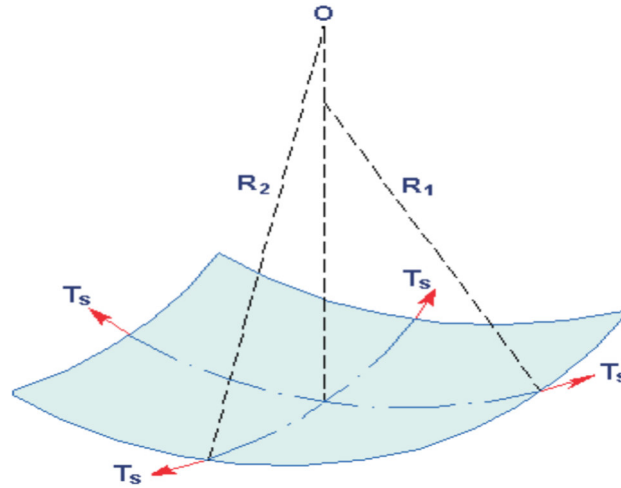


Figure 2.8: Surface tension on the three-dimension warped membrane (Fredlund and Rahardjo, 1993)

Figure 2.9 represents a three-dimensional illustration of four spherical particles and the pore-water menisci collected at points of contact, as well as the surface tension forces acting on water menisci. The state of capillary water at the contact points between particles can be described as a center-pinch cylinder with two extricated portions of spheres, one from its top and another from the base.

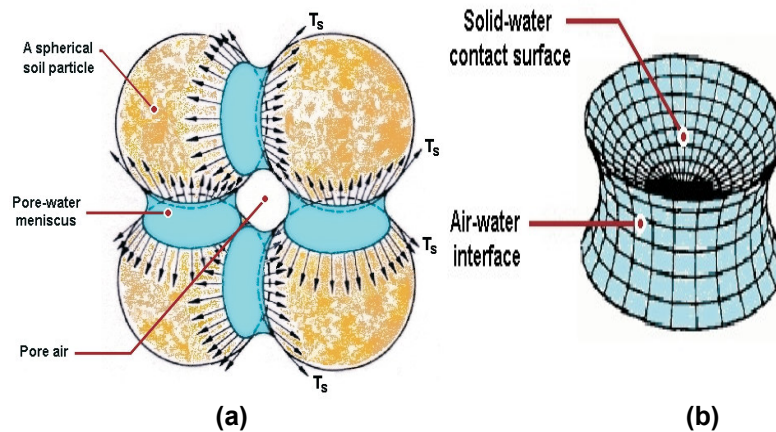


Figure 2.9: Representation of the numerical model: (a) forces acting on spherical particles due to surface tension, and (b) water meniscus at one point of contact between two particles (Farouk *et al.* 2004)

In unsaturated soil, the air-water interface is governed by pore water pressure (u_w) smaller than pore air pressure (u_a). The stress difference ($u_a - u_w$) is the matric suction. Equation 2.4 gives the difference in stress created by the contractile layer due to twist to a curvature. It is called Kelvin's capillary model equation.

$$U_a - U_w = \frac{2T_s}{R_s} \quad [2.4]$$

2.2.2 Capillary phenomenon

The capillary phenomenon is related to the matric suction component of total suction. The height of water rises and the radius of curvature have direct ramifications on the water content versus matric suction relationship in soils (the soil-water characteristic curve).

2.2.2.1 Capillary height

Consider the vertical equilibrium force of capillary water in a cylinder as shown in Figure 2.10 and described in Equation 2.5. The vertical part of the surface force ($2\pi r T_s \cos\alpha_1$) supports the weight of the water column, which has an elevation ($\pi r^2 h_c \rho_w g$).

$$\pi r^2 h_c \rho_w g = 2\pi r T_s \cos\alpha_1 \quad [2.5]$$

Where:

- α_1 = angle of contact [°]
- r = radius of the capillary tube [m]
- T_s = water surface tension [N.m⁻¹]
- h_c = capillary height [m]
- g = gravitational acceleration = 9.8 [m.s⁻²]
- ρ_w = density of water = 1000 [kg.m⁻³]

Equation 2.5 can be transposed to give the ultimate level of liquid in the capillary tube (h_c).

$$h_c = \frac{2 T_s}{\rho_w g R_s} \quad [2.6]$$

Where:

- R_s = radius of curvature of the meniscus ($\frac{r}{\cos\alpha_1}$) [m]

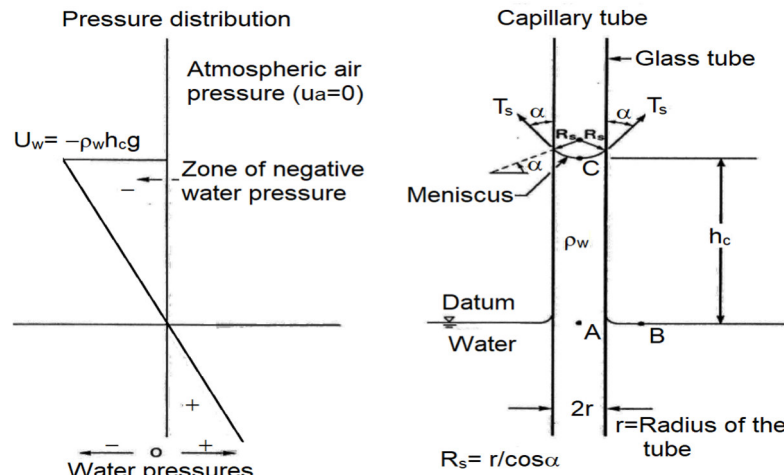


Figure 2.10: Physical model and phenomenon related to capillarity
(Fredlund and Rahardjo, 1993)

2.2.2.2 Capillary pressure

Points C, B, and A in the capillary system outlined in Figure 2.10 are in hydrostatic equilibrium. The air-water pressure occurs at points B and A. The height of points A and B above the water surface relies upon the datum or reference elevation of the system (zero elevation). Hence, the hydraulic head at points B and A is proportionate to zero elevation. Point C is at distance h_c above reference. The hydrostatic equilibrium among points C, B, and A is satisfied when the hydraulic head of the three points is the equivalent. The phenomenon infers the pressure head at point C, and it is equivalent to the negative value of the rising head at point C. Equation 2.7 describes water pressure at C.

$$u_w = - \rho_w g h_c \quad [2.7]$$

Where:

u_w = water pore pressure [kPa]

h_c = capillary height [m]

g = gravitational acceleration = 9.8 [m.s⁻²]

ρ_w = density of water = 1000 [kg.m⁻³]

The pressures of water above point A in the capillary cylinder are negative, as shown in Figure 2.10. In the capillary tube, water is exposed to tension. Nevertheless, water pressure underneath the point A is positive due to the conditions of hydrostatic

pressures. At point C, air pressure is atmospheric ($u_a = 0$) and water pressure is negative ($u_w = -\rho_w g h_c$). The matric suction ($u_a - u_w$) at point C can be expressed as follows:

$$u_a - u_w = \rho_w g h_c \quad [2.8]$$

The substitution of Equation 2.6 into Equation 2.8 gives another expression for the magnitude of the matric suction:

$$u_a - u_w = \rho_w g \times \frac{2 T_s}{\rho_w g R_s} = \frac{2 T_s}{R_s} \quad [2.9]$$

The surface strain can support a water column (h_c) in a capillary cylinder where pressure area is joined with water-air interface responses, as portrayed in Figure 2.11. The vertical component of the reaction force produces compressive stress that hangs on the wall of the cylinder. When the soil material exhibits a capillary zone, the water-air interface results in a growth of the compression of solid matrix. Hence, matric suction in unsaturated soils induces a volume decrease and an increase of shear strength.

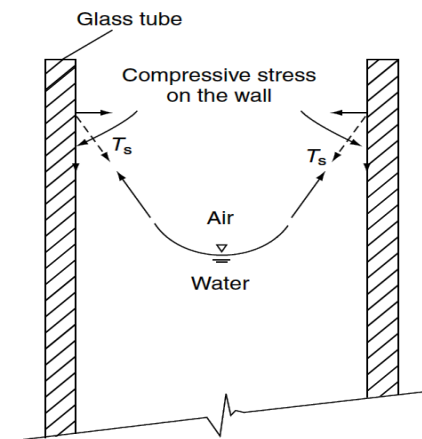


Figure 2.11: Forces acting on capillary tube (Fredlund and Rahardjo, 1993)

2.3 Theory of suction

There are numerous synonymous for suction, such as soil moisture deficiency, soil water pressure deficiency, capillary suction, matric suction, capillary potential, capillary water stress, pores water tension, soil water-free energy, and soil moisture tension. Soil suction is a free energy state of water inside the soil (Edlefsen and Anderson, 1943). The free water is characterised as water containing no dissolved solutes, having no connections with other phases that impart the curvature to the air-water interface, and having no external forces other than gravity (Lu and Likos, 2004). This free energy of water in the soil can be assessed using the partial vapour pressure of soil water. Equation 2.10 gives the thermodynamic relationship between the soil moisture deficiency and the partial pressure of pore water vapour.

$$\psi_t = -\frac{R T_K}{v_{wo}\omega_v} \ln\left(\frac{\bar{u}_v}{\bar{u}_{vo}}\right) \quad [2.10]$$

Where:

ψ_t = total soil suction [kPa]

R = universal (molar) gas constant (i. e., 8.31432) [J/(mol K)]

T_K = absolute temperature [i. e., $T_K = 273.15 + T(^{\circ}\text{C})$] [$^{\circ}\text{K}$]

v_{wo} = specific volume of water or the inverse of the density
of water $\left(\frac{1}{\rho_w}\right)$ [$\text{m}^{-3} \cdot \text{kg}$]

ω_v = molecular mass of water vapour (i. e., 18.016) [kg/kmol]

$\bar{u}_v$ = partial pressure of pore – water vapour [kPa]

$\bar{u}_{vo}$ = saturation pressure of water steam over a flat surface of
pure water at the same temperature [kPa]

2.3.1 Components of soil suction

The total suction (ψ_t) can be evaluated in terms of relative humidity next to the water surface. There are two essential components of soil suction namely the matric suction ($u_a - u_w$) and osmotic suction (ψ_o). Equation 2.11 gives the constitutive algebraic relation between the constituent elements of soil suction.

$$\psi_t = \psi_m + \psi_o \quad [2.11]$$

Where:

ψ_t = total soil suction [kPa]

ψ_m = matric suction ($u_a - u_w$) [kPa]

u_a = pore air pressure [kPa]

u_w = pore water pressure [kPa]

ψ_o = osmotic suction [kPa]

Figure 2.12 represents the general notion of total suction and its components as identified as the free energy of the soil water.

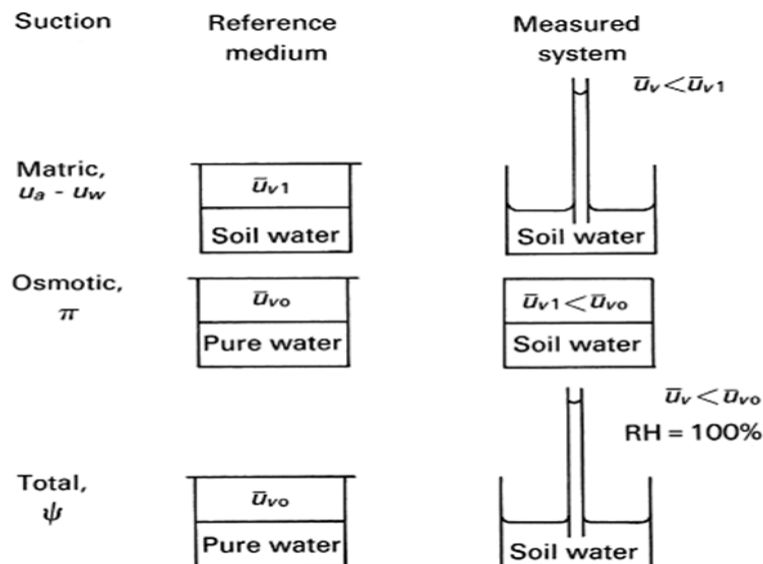


Figure 2.12: Total suction and its components: matric and osmotic suction (Fredlund and Rahardjo, 1993)

Consider a tube of glass filled with soil water. The region of water in the tube of glass is curved and is called the meniscus. Nonetheless, comparative soil water will have a level surface when put in a large container. The partial water vapour pressure above the curved surface of soil water ($\bar{u}_v$) is less than the partial water vapour pressure above a plane surface of identical soil water ($\bar{u}_{v1}$). In other words, RH in the soil will decrease because of the curved water surfaces produced by the capillary phenomenon. The water vapour pressure or RH diminishes as the radius of the water curvature surface decreases. Accordingly, the radius of curvature is inversely

proportional to the difference between the air and water pressures across the surface, and $(u_a - u_w)$ is called matric suction. Subsequently, the other component of the total suction is osmotic suction, and it contributes to decrease the relative humidity. Figure 2.13 illustrates the relative significance of changes in osmotic suction as compared to matric suction on compacted Regina Clay (Krahn and Fredlund, 1972).

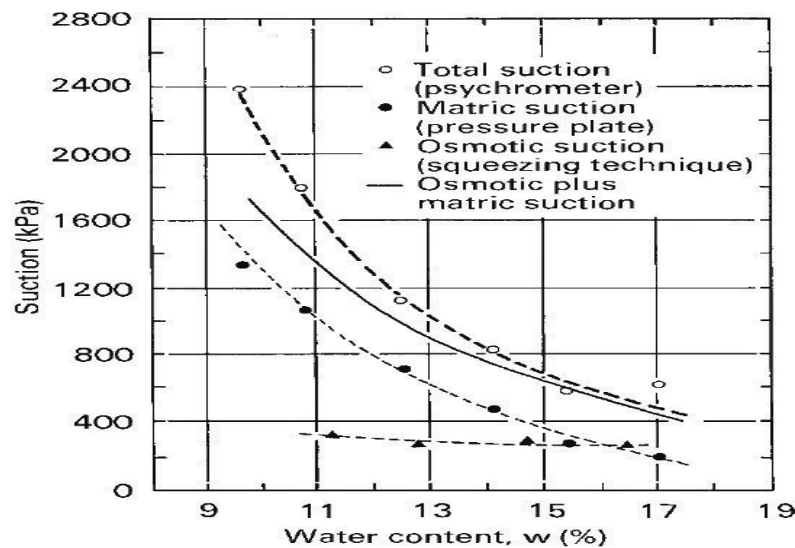


Figure 2.13: Total, matric and osmotic suction measurements on compacted Regina Clay (Krahn and Fredlund, 1972)

One can notice that unless fine silt or clay particles were present in the pore liquid, the osmotic suction remains practically unchanged throughout the range of water content. The change in the total suction can be attributed exclusively to the variation in matric suction.

2.4 Unsaturated soil stress state variables

The term “state variables” is defined as the non-material variables required to describe the physical behaviour of the material under consideration. As indicated in soil mechanics, the behaviour of soil material relies on the stress variables that control the equilibrium of soil material. The stress variables necessary to describe the soil structure equilibrium are the stress state variables for the soil. The stress variables must be quantifiable, for example, the total stress (σ) the air-water pressure (u_a) and water pore pressure (u_w). Stress equilibrium can be assessed on unsaturated soil by considering the state of stress at a point in the soil.

2.4.1 Stress state variables

The stress variables controlling the soil structure equilibrium are the stress state variables that control the mechanical behaviour of soils. Fredlund and Morgenstern (1977) reported that the stress variables related to the soil structure equilibrium are equivalent to the stress variables that control the contractile skin equilibrium. The normal stress is composed of three independent sets (surface tension) obtained from the equilibrium equations for the soil structure to shape the stress state variables. The three stress state variables are $(\sigma - u_a)$, $(u_a - u_w)$, and (u_a) . The stress variable (u_a) can be eliminated if the soil particles are assumed to be incompressible. Consequently, the stress state variables for the contractile skin and soil structure in unsaturated soils are $(\sigma - u_a)$ and $(u_a - u_w)$. The stress state variables act in three directions, and the variables can be utilised to shape two independent stress tensors. The two autonomous stress tensors can be composed as the stress state at a point in unsaturated soil.

$$\begin{bmatrix} (\sigma_x - u_a) & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & (\sigma_y - u_a) & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & (\sigma_z - u_a) \end{bmatrix} \quad [2.12]$$

$$\begin{bmatrix} (u_a - u_w) & 0 & 0 \\ 0 & (u_a - u_w) & 0 \\ 0 & 0 & (u_a - u_w) \end{bmatrix} \quad [2.13]$$

The pore-air pressure appears in both stress tensors; however, it is the difference between stress components that allows the two tensors to qualify as independent stress state variables. The stress variables in Tensors 2.12 and 2.13 can be placed on the surface of a cube to give the stress state at a point, as presented in Figure 2.14.

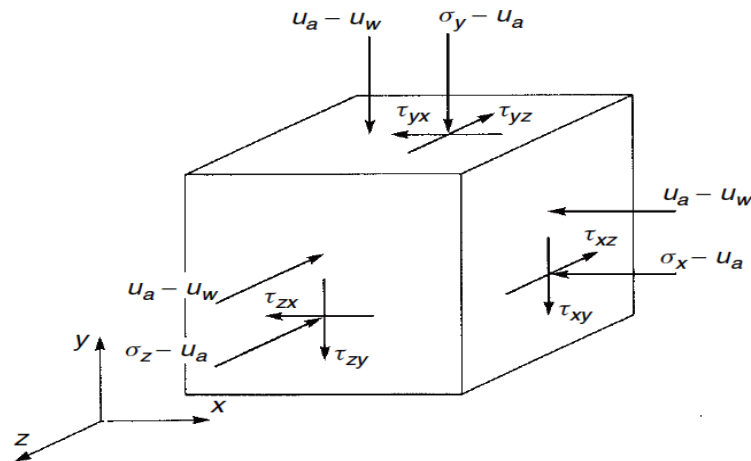


Figure 2.14: Stress state variables for unsaturated soil (Fredlund et al. 2012)

2.4.2 Other combinations of stress state variables

The three stress state variables combinations shown in Table 2.2 are obtained from the equations of soil equilibrium derived from three reference stresses (σ , u_a and u_w). Nonetheless, the combination of $(\sigma - u_a)$ and $(u_a - u_w)$ is considered suitable for engineering practice (Fredlund, 1979; Fredlund and Rahardjo, 1987).

Table 2.2: Possible combinations of stress state variables for unsaturated soils (Fredlund and Hasan, 1979)

Reference Pressure	Stress State Variables
Air pressure, u_a	$(\sigma - u_a)$ and $(u_a - u_w)$
Water pressure, u_w	$(\sigma - u_w)$ and $(u_a - u_w)$
Total stress, σ	$(\sigma - u_a)$ and $(\sigma - u_w)$

Another research carried out by Leong (2019) on controversy and consensus of stress state variables for unsaturated soils indicated that in recent years, it has been proposed that more than two stress state variables may be required to describe the constitutive behaviour of unsaturated soils.

2.5 Soil-water characteristic curve

2.5.1 Definition

The soil-water characteristic curve (SWCC) describes the thermodynamic potential of the pore water in soil related to the free water as a function of a fraction of water consumed by the soil material (Lu and Likos, 2004). The fraction of water can be described as volumetric water content (θ), gravimetric water content (w), and the level

of saturation (S). The SWCC provides a conceptual comprehension of the energy state of the water phase and the mass or volume of water in the soil. The SWCC is an interpretive model that uses the elementary capillary model to describe the water distribution in the voids. The effects of the soil texture, gradation, and void ratio have also become part of the measured data interpretation of laboratory SWCC. Many properties of unsaturated soils, including the coefficient of permeability, shear strength, volume strain, pore size distribution, and the quantity of water within a pore at a suction value, can be estimated from the SWCC. Figure 2.15 shows a typical SWCC drying curve. The three stages: capillary saturation zone, de-saturation zone, and residual saturation zone characterise behaviour under saturated or nearly-saturated conditions, unsaturated conditions with pore water continuity, and unsaturated with pore water discontinuity. In the capillary saturation zone, water in the pores is in tension. However, the soil remains saturated before the air entry value. The stage stops at air entry value, and the suction dominates the larger pores inside the soil material. The air entry value of the soil is the value of the matric suction at which the air starts to fill the larger pores. When the suction value is higher than the air entry value, the degree of saturation reduces quickly upon relatively small suction values and decreases gradually when the soil suction increases. There is a wide range of air entry value corresponding to different pore size distributions. If there is a high percentage of larger pores, the air-entry phase will occur over a larger range of moisture content. If pores are generally smaller, the transition is more rapid. The denser the soil, the greater the air entry value.

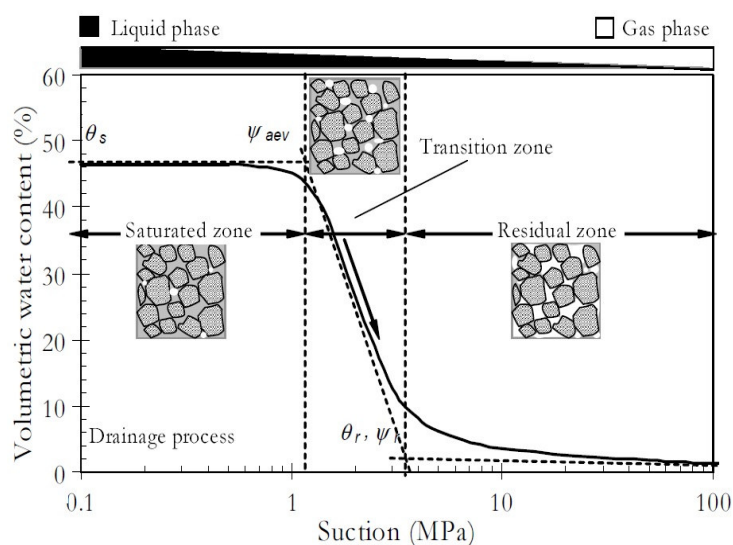


Figure 2.15: Typical soil water characteristic curve showing distinct zones (Vanapalli and Fredlund, 2000)

The transition zone is an area where water is drawn from the soil matrix by evaporation on the boundary or other removal mechanisms. This stage stops at the residual water content (θ_r), where pore-water becomes discontinuous. At this point, the hydraulic conductivity is decreased by several orders of magnitude, driven by a combination of vapour and fluid transport. Inside the residual zone, water is adsorbed tightly onto soil particles, and flows occurs only by vapour transport. A higher suction value change is required to pull out the extra water in the soil. This stage is attained when the moisture content is equal to oven dryness. When the soil material is heated to 105 °C, the water portion in the soil material is zero, and soil suction is approximately 1.10^6 kPa (Fredlund and Rahardjo, 1993).

Soil parameters such as the grain size distribution, void ratio, density, organic matter content, clay content, and mineralogy influence the shape and features of the SWCC. Clay soils tend to have a larger total pore volume as well as a more broadly distributed pore size. This means clay soils exhibit higher volumetric water content at saturation. The wider range of pore sizes means that clay soils require much higher suction levels to reach residual moisture conditions. Silt-sized soils generally behave somewhere between clay and sand. Figure 2.16 describes the typical SWCCs for sand, silt, and clay.

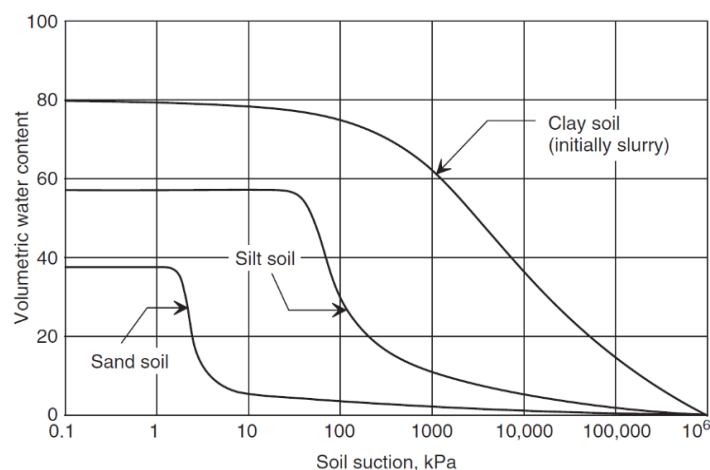


Figure 2.16: Typical SWCCs for different soil types (Fredlund and Xing, 1994)

2.5.2 Fitting equations for soil water characteristic curve

The soil-water characteristic curves (SWCC) can be utilised to estimate various parameters related to the unsaturated soil's behaviour. Some empirical equations

were proposed to best-fit the laboratory data for the SWCCs. Among these models, VG (Van Genuchten, 1980), DB (Durner, 1994), FX (Fredlund and Xing, 1994), LN (Kosugi, 1996) and BL (Seki, 2007).

Van Genuchten (1980) developed a mathematical equation model (VG) as a fitting equation for the soil-water characteristic curve that is written in $\theta(\psi)$ functional form.

$$\theta(\psi) = \theta_r + \frac{(\theta_s - \theta_r)}{[1 + (\alpha \psi)^{n_i}]^{1-1/n_i}} \quad [2.14]$$

Where:

- ψ = soil suction [kPa]
- θ = volumetric water content [%]
- θ_r = residual volumetric water content [%]
- θ_s = saturated volumetric water content [%]
- α = fitting parameter inversely related to soil suction
- n_i = fitting parameter that affect the shape of the curve

The SWCC model (DB) proposed by Durner (1994) is given in Equation 2.15. The volumetric water content at any suction is written in $\theta(\psi)$ functional form.

$$\theta(\psi) = \theta_r + \omega_1 \left[\frac{(\theta_s - \theta_r)}{1 + (\alpha_1 \psi)^{n_1}} \right]^{m_1} + (1 - \omega_1) \left[\frac{(\theta_s - \theta_r)}{1 + (\alpha_2 \psi)^{n_2}} \right]^{m_2} \quad [2.15]$$

Where:

- θ = volumetric water content [%]
- θ_r = residual volumetric water content [%]
- θ_s = saturated volumetric water content [%]
- ω_1 = weighting factor for the subcurve
- α_i = scaling factor that determines the position of the max pore sizes, and
- n_i, m_i = dimensionless curve shape parameters

Fredlund and Xing (1994) developed a mathematical equation model (FX) as a fitting equation for the soil-water characteristic curve that is written in $\theta(\psi)$ functional form.

$$\theta(\psi) = C(\psi) \frac{\theta_s}{\left[\ln \left[e + \left(\frac{\psi}{a} \right)^n \right] \right]^m} = \left[\frac{\ln \left(1 + \frac{\psi}{C_r} \right)}{\ln \left[1 + \left(1 + \frac{10^6}{C_r} \right) \right]} \right] \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{\psi}{a} \right)^n \right] \right\}^m} \quad [2.16]$$

Where:

θ = volumetric water content [%]

θ_s = saturated volumetric water content [%]

e = Napier's constant

C_r = an input value related to the residual suction

which can be estimated at 1500 kPa for most cases [kPa]

$C(\psi)$ = correction factor

ψ = soil suction [kPa]

a = soil parameter related to the air entry of the soil

n = soil parameter related to the rate of desaturation

m = soil parameter related to the residual water content conditions

The SWCC model (LN) proposed by Kosugi (1996) is given in Equation 2.17. The volumetric water content at any suction is written in $\theta(\psi)$ functional form.

$$\theta(\psi) = \theta_r + Q \left[\frac{(\theta_s - \theta_r) \ln \left(\frac{\psi}{\psi_{m_i}} \right)}{\sigma_i} \right] \quad [2.17]$$

Where:

θ = volumetric water content [%]

θ_r = residual volumetric water content [%]

θ_s = saturated volumetric water content [%]

ψ = soil suction [kPa]

Q = complementary cumulative normal distribution function defined by $Q(x) = 1 - \Phi(x)$ in which $\Phi(x)$ is a normalised form of the cumulative normal distribution function,

σ_i = standard deviation of the distribution, and

ψ_{m_i} = soil fitting parameter

Seki (2007) developed a mathematical equation model (BL) as a fitting equation for soil-water characteristic curve that is written in $\theta(\psi)$ functional form.

$$\theta(\psi) = \theta_r + (\theta_s - \theta_r) \omega_1 Q \left[\left(\frac{\ln\left(\frac{\psi}{\psi_{m1}}\right)}{\sigma_1} \right) + (1 - \omega_1) Q \left(\frac{\ln\left(\frac{\psi}{\psi_{m2}}\right)}{\sigma_2} \right) \right] \quad [2.18]$$

Where:

- θ = volumetric water content [%]
- θ_r = residual volumetric water content [%]
- θ_s = saturated volumetric water content [%]
- ω_1 = weighting factor for the subcurve
- Q = complementary cumulative normal distribution function defined by $Q(x) = 1 - \Phi(x)$ in which $\Phi(x)$ is a normalised form of the cumulative normal distribution function
- σ_i = standard deviation of the distribution
- ψ_{m_i} = soil fitting parameters

2.5.3 Hysteresis in soil-water characteristic curve

Fredlund et al. (2012), and Lu and Likos (2004) mentioned that the relation between moisture and suction is not unique for a given soil due to the distribution of pore spaces within its fabric. The network of pore spaces is not uniform; therefore, pore water may traverse alternately large and small pore spaces as it moves in and out of the soil. The result of these changing pore sizes means the drying and wetting processes are not mirror images of each other; rather, there is hysteresis where the wetting cycle will exhibit lower suction at a given moisture content when compared to the drying phase. Additionally, air pockets may be “caught” within the pore network, resulting in a lower water content as the wetting curve approaches near-zero suction. Thus, consideration should be given to this effect when applying the SWCC in geotechnical design. The wetting and drying branches bracket the limits of the water content versus soil suction relationship. Fredlund et al. (2012), and Pham et al. (2003) reported an infinite number of intermediate wetting and drying scanning curves presented in Figure 2.17, but it becomes asymptotic on the bounding curves. The drying and wetting curves should both be considered if there is a significant difference between them. Modern software can allow for this; however, the hysteresis behaviour may also be influenced by time

since vapour migration and condensation may occur as well. Finally, the requirements of the specific modelling objective (contaminant transport, slope stability, foundation swelling) will often dictate the requirements on how to treat hysteresis effects. Therefore, the geotechnical engineer must settle on a choice for the procedure to be simulated and afterward utilise the suitable evaluated unsaturated soil property function (Tami *et al.* 2004).

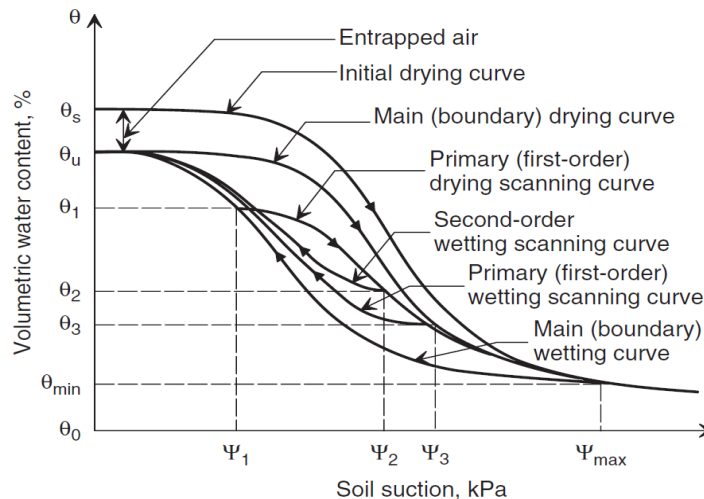


Figure 2.17: Commonly used definition for hysteretic SWCC (Pham *et al.* 2003)

Hoa *et al.* (1977) studied the impact of hysteresis on transient flows in saturated and unsaturated porous media. It was concluded that disregarding the hysteresis effect may introduce a significant error in the estimation of soil suction and water content. Laboratory apparatus should be developed to assess the hysteresis features in unsaturated soil material properties.

2.6 Conclusion

Based on the current investigation of the behaviour of unsaturated soils, it can be concluded that the differentiating feature of soil in unsaturated soils is the contribution of the negative pore pressure (soil suction). Soil suction is a free energy state of water inside the soil and is formed with two components, the matric suction and osmotic suction. Unsaturated soil system consists of three phases (solid, pore water, and pore air). The air-water interface (contractile skin) is a thin layer creating a fixed partition between the water phase and air phase, portraying a property called surface tension acting as a flexible membrane. The stress state variables describe the unsaturated

soil structure equilibrium, and the stress state variables for the contractile skin and soil structure in unsaturated soils are $(\sigma - u_a)$ and $(u_a - u_w)$. The SWCCs provide a conceptual comprehension of the energy state of the water phase and the mass or volume of water in the soil. Soil parameters like the shear strength, volume strain, AEV, and pore size distribution can be obtained from SWCCs. The following models, Seki (BL) and Durner (DB), Van Genuchten (VG), Fredlund and Xing (FX), and Kosugi (LN) proposed the best-fit laboratory data for SWCCs. Finally, to avoid hysteresis, the filter paper must be oven-dried to remove moisture and ensure that the same wetting path is followed in each case.

CHAPTER 3: PREVIOUS STUDIES ON SHEAR STRENGTH OF UNSATURATED SOILS

3.1 Overview

This chapter reviews past works on the behaviour of unsaturated soils, focusing on shear strength. The unsaturated behaviour has received attention from many researchers whose common objective has been to develop an expression to adequately model the shear strength of soil. This chapter reviews the historical development of the equations describing the shear strength in saturated and unsaturated soils. The bearing capacity, slope stability, lateral earth pressures are some examples of geotechnical applications related to the shear strength of the soil. Foundations for engineered structures are generally located above the groundwater table and are subjected to unsaturated conditions. The shear strength for unsaturated soils was developed as an extension of the saturated condition. The contribution of the matric suction is commonly added to the shear strength equation of saturated soil material to describe the shear strength of unsaturated soils.

3.2 Effective stress in saturated soils

As a multi-phase particulate medium, soil behaviour consists of the interaction of all physical and chemical forces between soil particles, air, and water. A fully comprehensive evaluation of these forces is not presently possible. However, a simplified expression for the intergranular pressure can be determined by considering the force balance within a level surface in saturated soil at a given depth (Figure 3.1). Equation 3.1 describes the vertical equilibrium of forces.

$$\sigma a + Aa + \dot{A}a_c = ua + Ca_c \quad [3.1]$$

Where

- a = average total cross sectional area [m^2]
- a_c = effective area of interparticle contacts [m^2]
- σ = applied vertical stress [kPa]
- u = hydrostatic pressure [kPa]
- A = longterm attractive stress
(i. e., van der Waals and electrostatic attractions) [kPa]
- $\dot{A}$ = shortrange attractive stress
(i. e., primary valence bonding and cementation) [kPa]
- C = repulsive stress resulting from hydration and born repulsion [kPa]

The division by (a) of all the terms in Equation 3.1 transforms all the forces into stresses described in Equation 3.2.

$$\sigma = (C - \lambda) \frac{a_c}{a} + u - A \quad [3.2]$$

The term $(C - \lambda) \frac{a_c}{a}$ represents the intergrain force divided by the gross area, referred to as intergranular stress (σ_i). Equation 3.2 can be reformulated as:

$$\sigma_i = \sigma + A - u \quad [3.3]$$

(u) is the pore water pressure at the genuine interparticle zone.

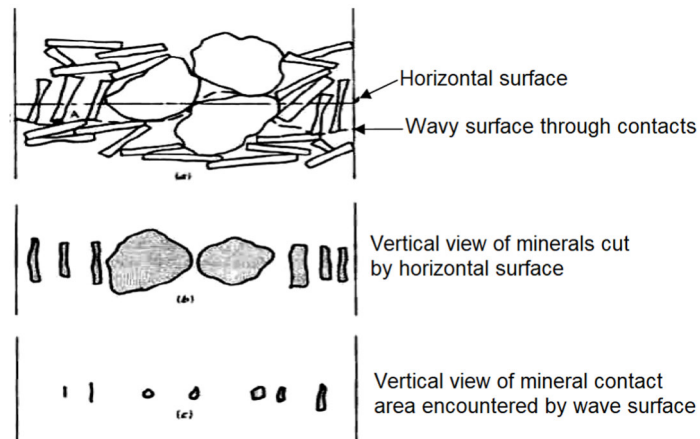


Figure 3.1: Surface through a soil mass (Mitchell, 1993)

The pore water pressure is not quite the same as the pore water pressure estimated by a piezometer or other pressure gauge devices (u_w). To accurately assess the effective stress, Bernoulli's equation is utilised to infer a correlation between (u) and (u_w) as follows:

$$u = u_w - Z \gamma_w - h_s \gamma_w \quad [3.4]$$

Where

Z = elevation difference between the piezometer and the point being referred [m]

γ_w = water unit weight [kg. m^{-3}]

h_s = osmotic head induces by the difference in ionic concentrations between points away from soil particles and points near them [m]

Assuming the marginal difference in elevation and utilising (u) from Equation 3.4, Equation 3.5 is formulated as follows:

$$\sigma_i = \sigma + A - u_w + h_s \gamma_w \quad [3.5]$$

Since the salt concentration between particles will be greater than at points away from them, the osmotic component ($h_s \gamma_w$) is negative and denoted (R) in previous investigations (Lambe 1960, Mitchell 1962, Balasubramonian, 1972). Thus, Equation 3.6 is given as:

$$\sigma_i = \sigma + A - u_w - R \quad [3.6]$$

Comparing the outcome in Equation 3.6 with the one utilising Terzaghi effective stress ($\sigma_i = \sigma - u_w$), the difference between the two relations are as follows:

$$\sigma_i - \sigma = A - R \quad [3.7]$$

The two equations give comparable outcomes if A and R are very small. A and R are very small in the case of sands, silts, and low plasticity clays. Nevertheless, in the scenario where either A or R is huge or significantly different, the results of the two formulae would not be comparable. Terzaghi (1936) successfully developed a single effective stress equation ($\sigma - u_w$) for saturated soils. Because of two phases (air and water) in soil pores, Terzaghi's effective stress concept could not be valid in unsaturated soils. Skempton (1960) stated that although the simple Terzaghi's effective stress formula does not give a genuine effective stress value, the stress formula is an excellent approximation for saturated soils.

3.3 Effective stress in unsaturated soils

In saturated soil materials or dry soils, Terzaghi's effective stress rule describes the effective stress as the difference between the pore fluid pressure and the total stress. The concept of effective stress for saturated soils as explained by Terzaghi (1936) is: "The stresses in any point of a section through a mass of soil can be computed from the total principal stresses, σ_1 , σ_2 , σ_3 , which act at this point. If the voids of the soil are filled with water under a stress, u_w , the total principal stresses consist of two parts. One part, u_w , acts in the water and in the solid in every direction with equal intensity. It is called the neutral pressure. The balance $\sigma'_1 = \sigma_1 - u_w$, $\sigma'_2 = \sigma_2 - u_w$, $\sigma'_3 = \sigma_3 - u_w$ represent an excess over the neutral stress, u_w , and has its seat exclusively in the solid phase of the soil. All the measurable effects of a change in shearing resistance are exclusively due to changes in the effective stress, σ'_1 , σ'_2 , σ'_3 ." The definition of effective stress by Terzaghi (1936) qualifies it as a stress state variable as defined in continuum mechanics and thermodynamic references. In the case of three-phase systems (unsaturated soils), Terzaghi's effective stress principle is no longer valid. The air in the soil pores induces a new force balance that incorporates capillary forces acting at the soil air-water interface. These forces directly affect the forces acting on particle contacts and highly influence the macroscopic behaviour of the soil. The unsaturated soils effective stress depends on the relative amounts of total stress (σ), air pressure (u_a) and water pressure (u_w). Numerous analysts have proposed various types of effective stress equations. Table 3.1 presents a synopsis of some effective stress equations for unsaturated soils.

Table 3.1: Effective stress equation for unsaturated soils

Authors	Equation	Notes
Croney <i>et al.</i> (1958)	$\sigma' = \sigma - \beta' u_w$	β' = a measure of the number of bonds under tension.
Bishop (1959)	$\sigma' = (\sigma - u_a) + \chi(u_a - u_w)$	χ = a parameter related to the degree of saturation of the soil.
Aitchison (1961)	$\sigma' = \sigma - \psi \cdot P''$	P'' = pore water pressure deficiency. ψ = a parameter with values ranging from zero to one.
Jennings (1961)	$\sigma' = \sigma - \beta \cdot P''$	P'' = negative pore water pressure taken as positive value. β = an experimentally measured statistical factor of the same type of the contact area.
Richards (1966)	$\sigma' = (\sigma - u_a) + \chi_m (u_a - u_w)_m + \chi_s (u_a - u_w)_s$	χ_m, χ_s = effective stress parameters for matric and osmotic suction, respectively. $(u_a - u_w)_m, (u_a - u_w)_s$ = matric and osmotic suction, respectively.
Aitchison (1965)	$\sigma' = \sigma + \chi_m p''_m + \chi_s p''_s$	χ_m, χ_s = soil parameters ranging from 0 to 1 depending on stress path. p''_m, p''_s = matric and osmotic suction, respectively.
Allam & Sridharan (1987)	$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) - R' + \gamma T_s$	T_s = surface tension γ = interphase perimeter R' = osmotic suction
Öberg & Sällfors (1997)	$\sigma' = (\sigma - u_a) + S(u_a - u_w)$	S = equivalent to χ in Bishop equation
Jommi (2000)	$\sigma_{ij}^* = \sigma_{ij} - (S u_w + (1 - S) u_a) \delta_{ij}$	δ_{ij} = Kroneker delta σ_{ij}^* = Bishop's average soil skeleton stress S = degree of saturation
Sheng <i>et al.</i> (2002)	$\sigma' = (\sigma - u_a) - \delta_{ij}(S)(u_a - u_w)$	δ_{ij} = Kroneker delta S = degree of saturation

The formula recommended by Bishop (1959) is the most general form since it incorporates a term for the gas phase pressure. It holds an understanding that the effective stress is formed by two components: one resulting from total normal stress ($\sigma - u_a$), and the other from stresses applied by the liquid in the soil pores ($u_a - u_w$) (Jennings and Burland, 1962).

$$\sigma' = (\sigma - u_a) + \chi (u_a - u_w) \quad [3.8]$$

Where:

σ' = effective stress [kPa]

σ = total stress [kPa]

u_a = pressure of pore air [kPa]

u_w = pressure of pore water [kPa]

$u_a - u_w$ = matric suction [kPa]

χ = Bishop parameters

0 for dry soil and 1 for saturated soil.

The intermediate values of Bishop parameters (χ) rely upon the level of saturation (S). However, Bishop parameter values can be affected by various elements, for instance, soil structure, shrinkage, swelling, and the variation of the stress state. Figure 3.2 describes the correlation between parameter (χ) and the level of saturation (S).

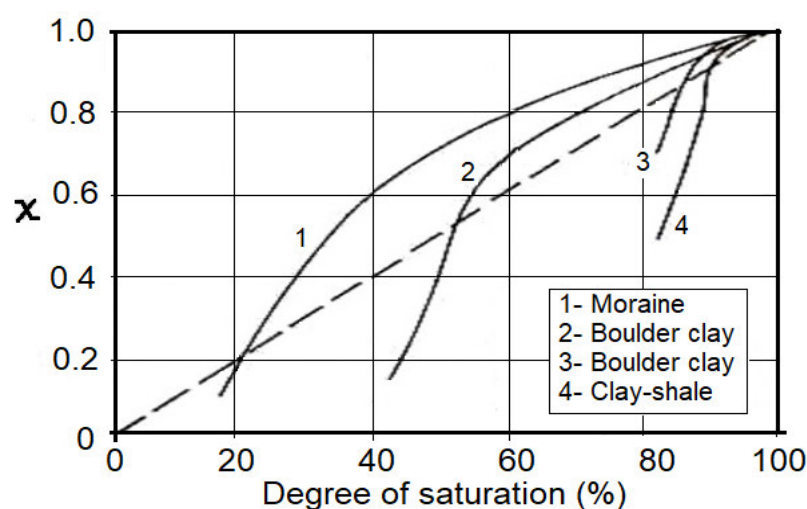


Figure 3.2: Relationship between parameter (χ) and the degree of saturation for compacted soils (Blight, 1961)

Seven years later, Richard (1966) recommended Equation 3.9 for effective stress, where he incorporated the contribution of osmotic suction. The subscripts m and s denoted the matric and osmotic suction. Besides, the result of the investigation led by Katte (2015) on the shear strength and suction correlation of granular material reported that even though the osmotic suction may indirectly impact the effective stress, it does not contribute to it directly. Therefore, the Bishop's equation portrays the stress condition of unsaturated soils adequately.

$$\sigma' = (\sigma - u_a) + \chi_m (u_a - u_w)_m + \chi_s (u_a - u_w)_s \quad [3.9]$$

Other equations similar to Bishop's Equation have been proposed by several researchers in developing models for unsaturated soils behaviour. Jommi (2000) developed the effective stress Equation 3.10 for unsaturated soils.

$$\sigma_{ij}^* = \sigma_{ij} - (S \cdot u_w + (1 - S) u_a) \delta_{ij} \quad [3.10]$$

Where:

σ_{ij} = total stress tensor [kPa]

δ_{ij} = Kroneker delta or substitution tensor

σ_{ij}^* = Bishop's average soil skeleton stress [kPa]

S = degree of saturation [%]

Sheng *et al.* (2002) proposed another equation for the effective stress formulated as follows:

$$\sigma' = \sigma + \delta_{ij}(S)(u_w) \quad [3.11]$$

When the $u_a > 0$ the equation is exactly Bishop's equation and given as

$$\sigma' = (\sigma - u_a) - \delta_{ij}(S)(u_a - u_w) \quad [3.12]$$

Where:

σ' = effective stress [kPa]

σ = total stress [kPa]

S = degree of saturation [%]

This section has reviewed the historical development of the equations describing the effective stress in unsaturated soils. The first effective stress equation was proposed by Croney *et al.* (1958) utilising the number of bonds under tension and the pore water pressure. Bishop (1959) proposed an effective stress formula by adding the contribution of the suction matrix to the initial formula proposed by Terzaghi (1936). Other researchers such as Aitchison (1961), Jennings (1961), and Aitchison (1965) proposed effective stress equations based on the pore water pressure within the soil. Richards (1966), Allam and Sridharan (1987) developed effective stress equations by incorporating the contribution of the osmotic suction, building upon the concept introduced by Bishop's equation. Other more recent formulae similar to the Bishop's equation was proposed by Allam and Sridharan (1987), Öberg and Sällfors (1997), Jommi (2000), and Sheng *et al.* (2002). By and large, Bishop's equation describes the effective stress conditions in unsaturated soils adequately, and it is easy to use in practice.

3.4 Shear strength of saturated soils

Terzaghi (1936) used the effective stress variable and the Mohr-Coulomb failure criterion (Mohr, 1914) to formulate a mathematical equation to predict the shear strength of saturated soils.

$$\tau_s = c' + (\sigma_f - u_{wf})_f \tan(\phi') \quad [3.13]$$

Where:

τ_s = saturated shear strength on the failure plane at failure [kPa]

c' = effective cohesion, which is the shear strength intercept
when effective normal stress is equal to zero [kPa]

$(\sigma_f - u_{wf})_f$ = effective normal stress on the failure plane at failure [kPa]

σ_f = total normal stress on the failure plane at failure [kPa]

u_{wf} = pore water pressure at failure [kPa]

ϕ' = effective angle of internal friction [°]

Equation 3.13 describes a linear correlation between the effective stress and shear stress shown in Figure 3.3. The line tangent to the Mohr circles is the failure envelope

since it represents the possible combinations of the effective normal stress and the shear strength on the failure plane at failure.

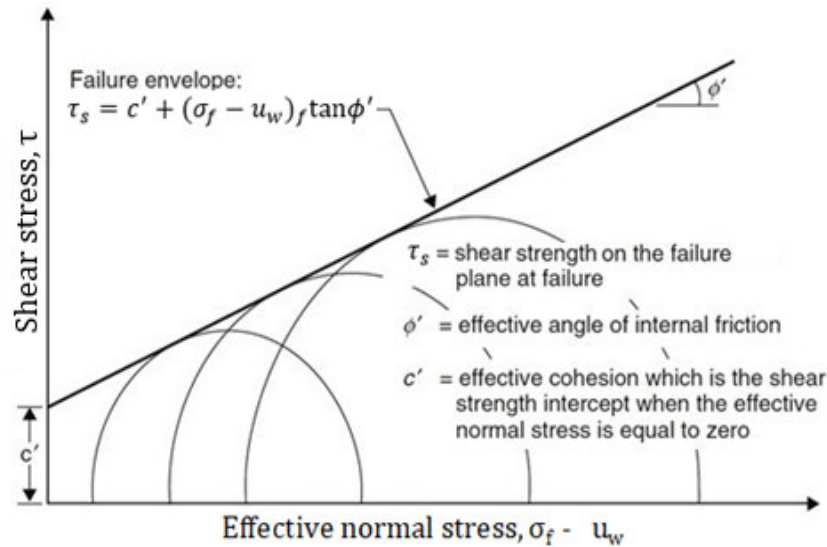


Figure 3.3: Mohr-Coulomb failure envelope for saturated soils (Fredlund et al. 2012)

Each Mohr circle shows different deviator stress, characterised by the diameter of the circle. The failure envelope illustrates the shear strength of the soil for each effective normal stress when the pore water stress acts isotropically on all planes. The shear strength portrayed by the failure envelope represents the combination of effective normal stress and shear stress that results in plastic failure. The failure envelope is achieved by plotting a tangent line to a series of Mohr circles describing the failure conditions. The slope of the lines gives the effective friction angle (ϕ'), and its intercept on the ordinate is known as the effective cohesion (c'). The orientation of failure plane in the soil is obtained by joining the pole point to the point of intersection between the Mohr circle and the failure envelope. The tangent point on Mohr circle at failure represents the stress state on the failure plane. This failure criterion is well-founded for saturated soils.

3.5 Shear strength of unsaturated soils

The failure condition under shear loading may be written as an extension of the Mohr-Coulomb failure criteria. The shear strength failure criterion can be utilised to extend the representation of saturated soil shear strength to unsaturated soil conditions. The principle of shear strength of partially saturated soils is described as an extension of the concept and mathematical equations utilised to formulate the shear strength for saturated soils, such as the Mohr-Coulomb failure criterion. The Mohr-Coulomb

criterion extension is known as the extended Mohr-Coulomb failure criterion for unsaturated soil. The stress condition of partially saturated soils can be represented utilising the suction matric ($u_a - u_w$) and the net normal stress ($\sigma - u_a$). The shear strength of unsaturated soils is formulated by adding the matric suction to the stress condition of saturated soils. Tables 3.2 and 3.3 present some equations proposed for the shear strength of unsaturated soils.

Table 3.2: Shear strength equations for unsaturated soils

Authors	Equation
Bishop <i>et al.</i> (1960)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \chi(u_a - u_w) \tan(\phi')$
Fredlund <i>et al.</i> (1978)	$\tau_u = c' + (\sigma - u_w) \tan(\phi') + (u_a - u_w) \tan(\phi'')$
	$\tau_u = c' + (\sigma - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b)$
Lamborn (1986)	$\tau_u = c' + (\sigma_f - u_a)_f \tan(\phi') + (u_a - u_w)_f (\theta_w) \tan(\phi^b)$
Fredlund and Rahardjo (1993)	$\tau_u = c' + (\sigma - u_a)_f \tan(\phi') + (u_a - u_w)_f \tan(\phi^b)$
Fredlund <i>et al.</i> (1996)	$\tau_u = c' + (\sigma_f - u_a) \tan(\phi') + (u_a - u_w)_f (\theta_d^k) \tan(\phi')$
Vanapalli <i>et al.</i> (1996)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \left(\frac{\theta - \theta_r}{\theta_s - \theta_r} \right) (u_a - u_w) \tan(\phi')$
Öberg and Sällfors (1997)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (S)(u_a - u_w) \tan(\phi')$
Khalili and Khabbaz (1998)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (\chi)(u_a - u_w) \tan(\phi')$
	$\chi = \left[\frac{(u_a - u_w)}{AEV} \right]^{-0.55}$
Bao <i>et al.</i> (1998)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (\xi)(u_a - u_w) \tan(\phi')$
	$\xi = \frac{\log(RS) - \log(u_a - u_w)}{\log(RS) - \log(AEV)}$
Vanapalli and Fredlund (2000)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \chi(u_a - u_w) \tan(\phi')$
	$\chi = (S_r)^k$ $k = 0.98 + 0.0874 (PI) - 0.001 (PI)^2$
Aubeny and Lytton (2003)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + f_1(\theta)(u_a - u_w) \tan(\phi')$
	if $S = 100\%$: $f_1 = \frac{1}{\theta}$
	if $85\% \leq S \leq 100\%$: $f_1 = 1 + \frac{S - 85}{15} \left(\frac{1}{\theta} - 1 \right)$ if $S \leq 85\%$: $f_1 = 1$

Table 3.3: Shear strength equations for unsaturated soils

Authors	Equation
Tekinsoy <i>et al.</i> (2004)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \tan(\phi') (AEV + P_{at}) \ln \left[\frac{(u_a - u_w) + P_{at}}{P_{at}} \right]$
Garven and Vanapalli (2006)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \theta^k (u_a - u_w) \tan(\phi')$ $k = 0.00161 I_p^2 + 0.0975 I_p + 1$
Sheng <i>et al.</i> (2008)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b)$ where $\tan(\phi') = \tan(\phi^b) \text{ if } (u_a - u_w) < (u_a - u_w)_{sa}$ where $\tan(\phi^b) = \tan(\phi') \left[\frac{(u_a - u_w)_{sa}}{(u_a - u_w)} + \left(\frac{(u_a - u_w)_{sa} + 1}{(u_a - u_w)} \right) \ln \frac{(u_a - u_w) + 1}{(u_a - u_w)_{sa} + 1} \right]$ if $(u_a - u_w) > (u_a - u_w)_{sa}$ $(u_a - u_w)_{sa}$: saturation suction
Guan <i>et al.</i> (2010)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b)$ where $\phi' = \phi^b$ if $(u_a - u_w) < AEV$ $\tau_u = c' + [(\sigma_n - u_a) + AEV] \tan(\phi') + [(u_a - u_w) - AEV] b \theta^k \tan(\phi')$ if $(u_a - u_w) \geq AEV$ $k = [\log(u_a - u_w) - \log(AEV)]^y$ For drying: $y_d = 0.502 \ln(I_p + 2.7) - 0.387$ $b_d = -0.254 \{ \ln[n_d(I_p + 4.4)] - 0.387 \}^2 + 2.114 \{ \ln[n_d(I_p + 4.4)] \} - 3.522$ For wetting: $y_w = 3.55 y_d - 3.00$ $b_w = 0.542 b_d \left(\frac{n_d}{n_w} \right) + 0.389$
Ahmadi and Toker (2019)	$\tau_u = c' + (\sigma - u_a) \tan(\phi') + \left[1 - e^{\frac{-\psi}{\psi_t}} \right] \cdot \psi_t \cdot \tan(\phi')$ $c''_{max} = \tan(\phi') \cdot \psi_t \text{ and } \chi = \frac{\psi}{\psi_t} \cdot \left[1 - e^{\frac{-\psi}{\psi_t}} \right]$
Bulolo and Leong (2019)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \chi S_r \tan(\phi')$
Gao <i>et al.</i> (2020)	$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \chi S \tan(\phi') \quad S \leq S_{max}$ $\tau_u = \tau_{max} + (\chi S - \chi_{max} S_{max} \tan(\phi_s)) \quad S > S_{max}$

3.5.1 Bishop *et al.* (1960) studies on shear strength

Bishop *et al.* (1960) conducted research on the deformation characteristics and shear strength of unsaturated soils. Experimental work was performed to investigate the principles of unsaturated shear strength. Bishop *et al.* (1960) results showed a significant impact of the matric suction on shear strength. Building on Terzaghi's effective-stress hypothesis for saturated soils, Bishop *et al.* (1960) proposed a constitutive Equation 3.14 to predict the shear strength of unsaturated soils.

$$\tau_u = c' + [(\sigma_n - u_a) + \chi(u_a - u_w)] \tan(\phi') \quad [3.14]$$

Where:

- τ_u = unsaturated shear strength [kPa]
- c' = effective cohesion intercept [kPa]
- σ_n = total normal stress on the failure plane [kPa]
- u_a = pressure of pore air [kPa]
- u_w = pressure of pore water [kPa]
- ϕ' = effective angle of internal friction [°]
- χ = Bishop parameter

Based on results of experimental assessment of the parameter (χ), Bishop considered that the values of (c') and (ϕ') are independent of the degree of saturation. The values (c') and (ϕ') can be obtained from experiments performed on saturated soil specimens. The parameter (χ) represents the influence of matric suction on shear strength of unsaturated soils. Bishop adopted this process for compacted soil materials. He understood that the air-water term indeed affected the strength characteristics of the soil and included its contribution through the use of the χ -factor (chi-factor).

3.5.1.1 Experimental determination of (χ)

Bishop's experiments focused on silt samples from Braehead placed in a triaxial device as saturated, then drained by suction until the material had an effective cohesion (c'). The material was relatively permeable with a saturation coefficient of

permeability ($K = 3.3 \cdot 10^{-7} \text{ m.s}^{-1}$). The values of (χ) were calculated from Equation 3.14 using data obtained from the experiments conducted at full saturation. This method was not entirely successful because, for an identical history of stress, the rate of volume changes at failure varies with the degree of saturation and over-consolidation. The values of (χ) are calculated from tests conducted on three compacted soils with clay fractions of 22 %, 18 %, and 4 %. Figure 3.4 shows a small increment of effective cohesion and demonstrates the uniqueness of the rupture envelope.

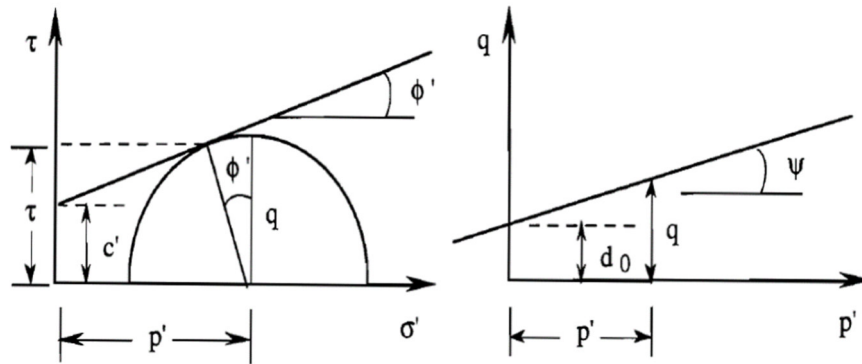


Figure 3.4: Effective cohesion and rupture envelope (Gueye, 1993)

Equation of lines are:

$$P' = [1/2 (\sigma_1 + \sigma_3) - u_w] \text{ and } q = \frac{1}{2}(\sigma_1 - \sigma_3) \quad [3.15]$$

$$\tau = q \cos \phi' = c' + (P' - q \sin \phi') \tan \phi' \quad [3.16]$$

Where:

- τ = shear strength [kPa]
- c' = effective cohesion intercept [kPa]
- σ = normal stress, σ_1, σ_3 [kPa]
- P' = average normal stress [kPa]
- u_w = pressure of pore water [kPa]
- ϕ' = effective angle of internal friction, $\sin \phi' = \tan \psi'$ [°]
- q = deviator stress [kPa]

In Figure 3.5, the slope ψ of the effective stress line passing through these points is connected to the slope of the Mohr-Coulomb envelope. When the results of unsaturated soil tests are plotted on a diagram, $1/2 (\sigma_1 - \sigma_3)$ is a function of

$[1/2 (\sigma_1 + \sigma_3) - u_w]$, and the value of (χ) is directly obtained in terms of the relative distance of these two lines of failure of the effective stress rupture lines for saturated soil specimens.

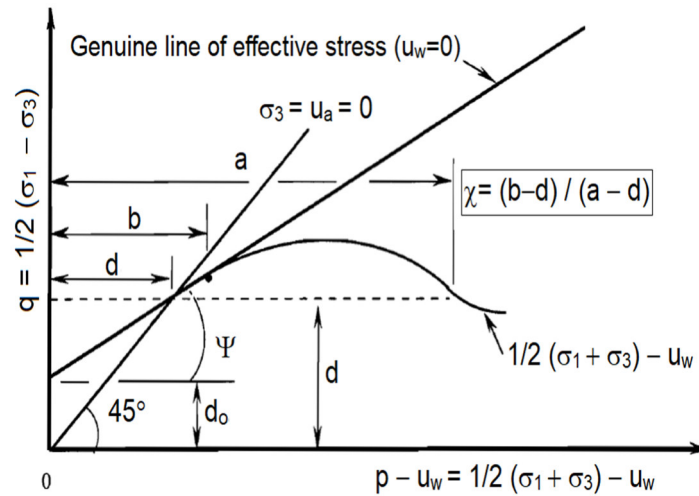


Figure 3.5: Method of calculating χ from the failure stress curve tests with water content variable $\sigma_3 = u_a$ (Bishop *et al.* 1960)

There is a difference between the curves $1/2 (\sigma_1 - \sigma_3)$ as a component of $[1/2 (\sigma_1 + \sigma_3) - u_w]$ in saturated and unsaturated soils. The extrapolated curves could produce a negative cohesion for saturated specimens tested upon steady water content and variable confining pressure (σ_3). These observations show that it is essential to consider the air pressure to obtain a reasonable failure envelope. The test outcomes show that (χ) relies upon the level of saturation. The relationship was developed from compacted soils and could not be further generalised. Later, Vanapalli and Fredlund (2000) proposed a mathematical equation related to laboratory tests to determine the value of (χ) .

$$\chi = (S)^k \quad [3.17]$$

Where:

S = degree of saturation of the soil [kPa]

k = fitting parameter predominantly a function of PI

$$k = 0.98 + 0.0874 (PI) - 0.001 (PI)^2$$

PI = plasticity index [%]

3.5.2 Fredlund's research on shear strength

Fredlund *et al.* (1978) characterise the shear strength of unsaturated soil (τ_u) in two unique ways:

$$\tau_u = c' + (\sigma - u_w) \tan(\phi') + (u_a - u_w) \tan(\phi'') \quad [3.18]$$

$$\tau_u = c' + (\sigma - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) \quad [3.19]$$

Where:

- τ_u = unsaturated shear strength [kPa]
- c' = effective cohesion when
 $(u_a - u_w) = 0$ and $(\sigma - u_a) = 0$, [kPa]
- σ = total normal stress [kPa]
- u_a = pore air pressure [kPa]
- u_w = water pore pressure [kPa]
- ϕ' = effective angle of internal friction of $(\sigma - u_a)$ [°]
- ϕ^b, ϕ'' = effective angle of $(u_a - u_w)$ [°]

3.5.2.1 Shear strength as a function of $(\sigma - u_a)$ and $(u_a - u_w)$

The shear strength can be described as an inclined surface in a three-dimensional space $[\tau, (u_a - u_w), \text{ and } (\sigma - u_a)]$ in the structure proposed in Figure 3.6a. Any parallel section to the plane $[\tau, (\sigma - u_a)]$ appears in Figure 3.6b. Therefore, the term $c' + (u_a - u_w) \tan(\phi^b)$ can be considered as a cohesion term.

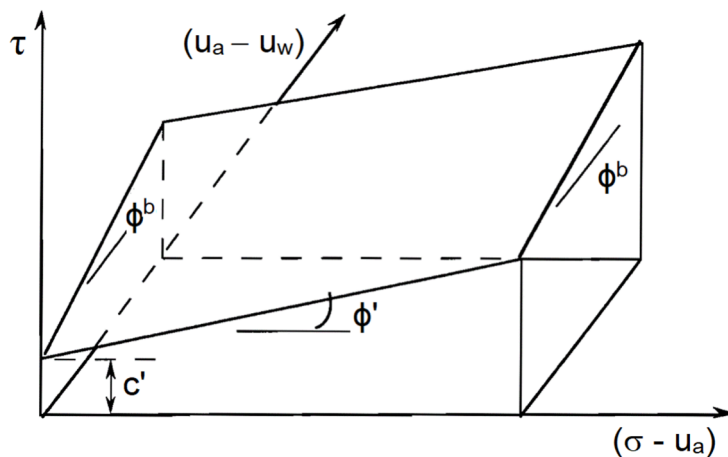


Figure 3.6a: Extended Mohr-Coulomb failure surface (Fredlund, 1978)

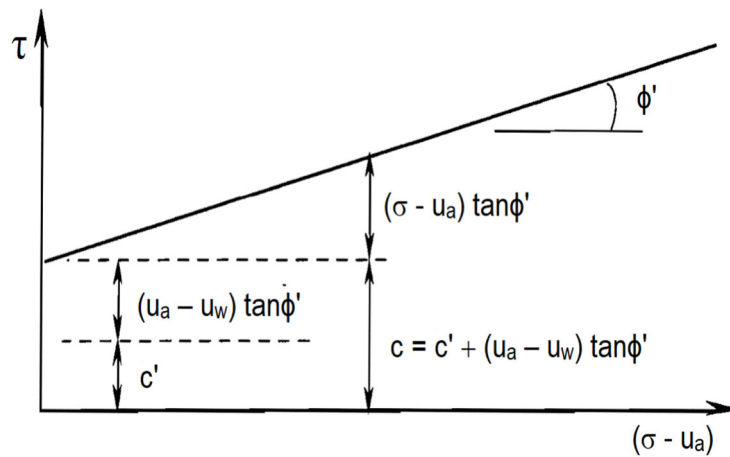


Figure 3.6b: Increased resistance due to the matric suction (Gueye, 1993)

Fredlund *et al.* (1978) reported that two out of the three possible stress variables can uniquely characterise the state of stresses in unsaturated soils. The possible combinations are as follows:

$$((\sigma - u_w) \text{ and } (u_a - u_w)); ((\sigma - u_a) \text{ and } (u_a - u_w)); ((\sigma - u_a) \text{ and } (\sigma - u_w)).$$

The study demonstrated that the shear strength of partially saturated soils is described as a function of a combinations of two independent stress state variables. The combination of Equations 3.18 and 3.19 is used for both cases. The first stress state variables used are $(\sigma - u_w)$ and $(u_a - u_w)$. The advantage of this combination of variables is that it allows the visualisation of the transition from the unsaturated case to the saturated case. The disadvantage is that the interstitial water pressure varies and influences the two variables of the stress state. However, the second combination $(\sigma - u_a)$ and $(u_a - u_w)$ has the advantage that only one stress variable is affected by interstitial water pressure changes. In this case, the shear strength is the same for a particular soil with values of (σ, u_a, u_w) . For saturated soils, the Mohr-Coulomb stress circles of failure conditions are shown in a plane as follows: The shear stress (τ) is a function of the effective normal stress $(\sigma - u_w)$. A series of tests gives the line of failure. In unsaturated soils, the condition of failure is described within a three-dimensional space. The axis in the horizontal plane is the state variables of stress, and the ordinate is the shear stress shown in Figures 3.6a and 3.6b.

3.5.2.2 Shear strength as a function of $(\sigma - u_w)$ and $(u_a - u_w)$

The vertical plane of $(\sigma - u_w)$ function of (τ) , with $(u_a - u_w) = 0$, corresponds to the situation where the soil is saturated. When the matric suction is positive, $(u_a - u_w) > 0$, a third dimension is needed to draw the circle of stress. For unsaturated soils, it is assumed that the area defined by a series of tests is level. Nonetheless, the equation describing the failure surface is written as an extension of the conventional case of saturated soils. The shear strength can be described as an inclined surface in a three-dimensional space $[\tau, (u_a - u_w), \text{ and } (\sigma - u_w)]$ in the structure proposed in Figure 3.7a.

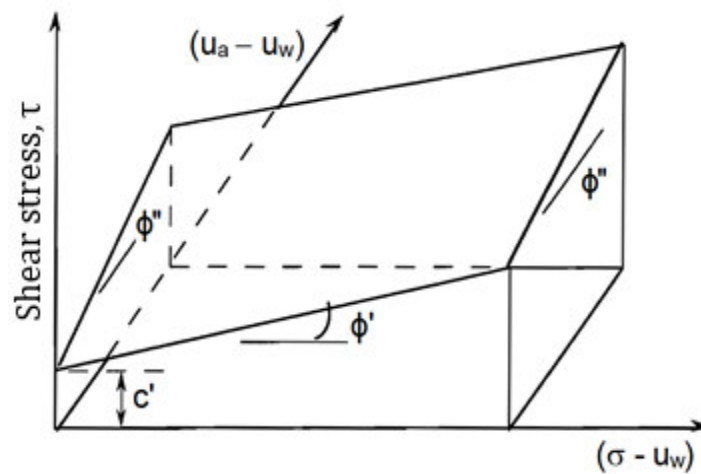


Figure 3.7a: Extended Mohr-Coulomb failure surface (Fredlund, 1978)

$$\tau_u = c' + (\sigma - u_w) \tan(\phi') + (u_a - u_w) \tan(\phi'') \quad [3.20]$$

Where:

- τ_u = unsaturated shear strength [kPa]
- c' = effective cohesion [kPa]
- σ = total normal stress [kPa]
- u_a = pressure of pore air [kPa]
- u_w = pressure of pore water [kPa]
- ϕ' = effective angle of internal friction of $(\sigma - u_w)$ [°]
- ϕ'' = effective angle of internal friction of $(u_a - u_w)$ [°]

Conventionally, using the results of triaxial tests, the point stress at the top of each Mohr circle can be considered. Figure 3.7b shows the coordinates:

$$[1/2 (\sigma_1 - \sigma_3) - u_w] \text{ and } (u_a - u_w)$$

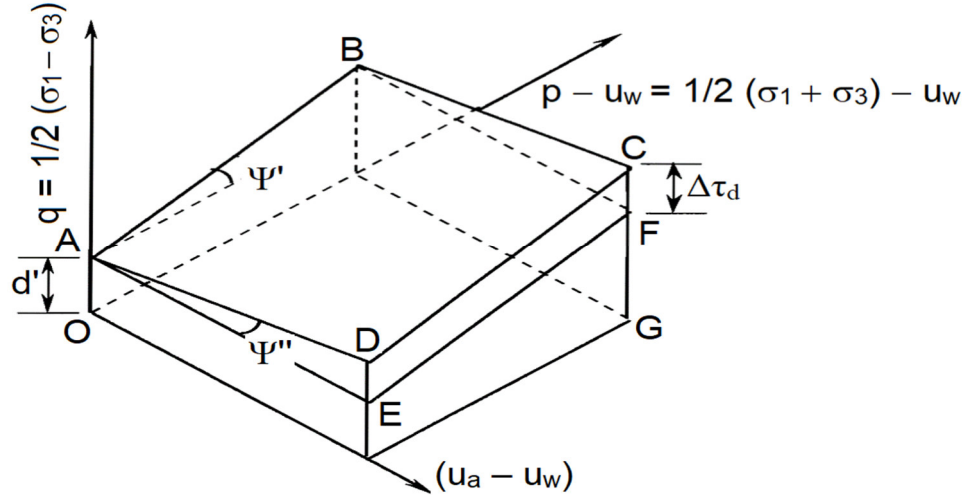


Figure 3.7b: Three-dimensional failure surface using stress points (Gueye, 1993)

When using the stress point method, the failure surface for saturated soils is given by the ABFE plane. If the matric suction does not affect the shear strength ($\Delta\tau_d = 0$), the surface failure appears as the line AB in Figure 3.7b. The equation of the plane for unsaturated soils is written as follows:

$$\frac{1}{2} (\sigma_1 - \sigma_3) = d' + \left[\frac{1}{2} (\sigma_1 + \sigma_3) - u_w \right] \tan \psi' + (u_a - u_w) \tan \psi'' \quad [3.21]$$

Say $P = 1/2 (\sigma_1 + \sigma_3)$ and $q = 1/2 (\sigma_1 - \sigma_3)$, Equation 3.21 becomes

$$q = d' + (p - u_w) \tan \psi' + (u_a - u_w) \tan \psi'' \quad [3.22]$$

Where:

$d' =$ ordinate at the origin in the plane (p, q) [mm]

$\psi' =$ angle between the plane stress point and the axis
[p - u_w] when [u_a - u_w] constant [°]

$\psi'' =$ angle between the plane stress point and the axis
[u_a - u_w] when [p - u_w] constant [°]

The relation between a plane passing through the stress points and the failure surface for unsaturated soils is given by the ABCD plane, as presented in Figure 3.7c. Therefore, the shear strength given in Equation 3.19 is applied in this case:

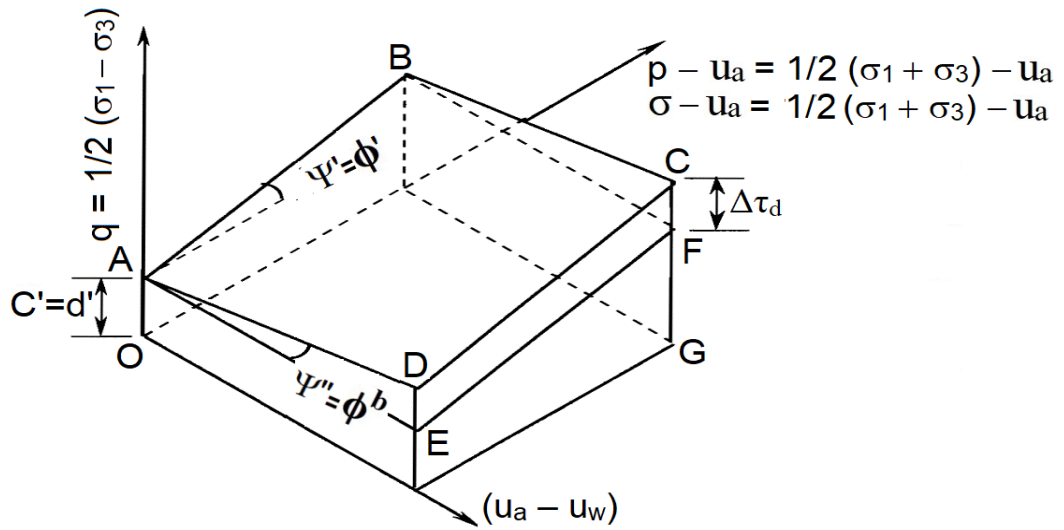


Figure 3.7c: Tridimensional failure surface using stress points (Gueye, 1993)

$$\tau_u = c' + (\sigma - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) \quad [3.23]$$

Equations 3.18 and 3.19 give a similar value of shear strength. However, it is possible to match them and obtain a correlation between various angles of friction as given in Equation 3.24.

$$\tan(\phi') = \tan(\phi^b) - \tan(\phi'') \quad [3.24]$$

Fredlund *et al.* (1978) suggested the last form.

$$\tau_u = c' + (\sigma - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) \quad [3.25]$$

It is best practice that soil mechanics laboratory tests performed on expansive soils are necessary to understand the shear strength of unsaturated expansive soils. The comparison of the shear strength according to Bishop's and Fredlund's equations is as follows:

$$\text{(Bishop)} \quad \tau = c' + [(\sigma_n - u_a) + \chi(u_a - u_w)] \tan(\phi') \quad [3.26]$$

$$\text{(Fredlund)} \quad \tau = c' + (\sigma - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) \quad [3.27]$$

$$\chi \tan(\phi') = \tan(\phi^b) \quad \Rightarrow \quad \chi = \frac{\tan(\phi^b)}{\tan(\phi')} \quad [3.28]$$

3.5.3 Lamborn's works on shear strength

Lamborn (1986) developed a shear strength equation for unsaturated soils by extending a micro-mechanics model based on concepts of irreversible thermodynamics to the energy versus the volume correlation in a multiphase material comprising fluid (water), air, soil solids (particles), and contractile skin. The findings indicates that shear strength for unsaturated soils is often represented using either bilinear or linear approximations, with non-linear approximation being more realistic and accurate.

$$\tau_u = c' + (\sigma - u_a) \tan(\phi') + (u_a - u_w)\theta \tan(\phi') \quad [3.29]$$

The volumetric moisture content (θ) is characterised as the proportion of the volume of water over the soil's total volume. The volumetric moisture content (θ) reduces when the soil suction increases, and it is a non-linear function of soil suction. Lamborn (1986) mentioned that the majority of shear strength equations for unsaturated soils are either bilinear or linear approximations, and the non-linear equation is more realistic and gives a more accurate approximation.

3.5.4 Fredlund and Rahardjo's studies on shear strength

Fredlund and Rahardjo (1993) established the shear strength equation from the direct test shear results and extended Mohr-Coulomb failure envelope for unsaturated soils shown in Figure 3.8. The shear strength equation depends on stress variables formulated as follows:

$$\tau_u = c' + (\sigma - u_a)_f \tan(\phi') + (u_a - u_w)_f \tan(\phi^b) \quad [3.30]$$

Where:

- τ_u = unsaturated shear strength at failure [kPa]
- c' = intercept of the extended Mohr-Coulomb failure envelope on the shear strength axis where the net normal stress c and the matric suction at failure are equal to zero also called "effective cohesion" [kPa]
- $(\sigma - u_a)_f$ = net normal stress state on the failure plane at failure [kPa]
- u_a = pore air pressure on the failure plane at failure [kPa]
- ϕ' = angle of internal friction associated

with the net normal stress state variable [$^{\circ}$]

ϕ^b = angle indicating the rate of increase in shear strength
with respect to a change in matric suction $(u_a - u_w)_f$ [$^{\circ}$]

$(u_a - u_w)_f$ = matric suction on the failure plane at failure [kPa]

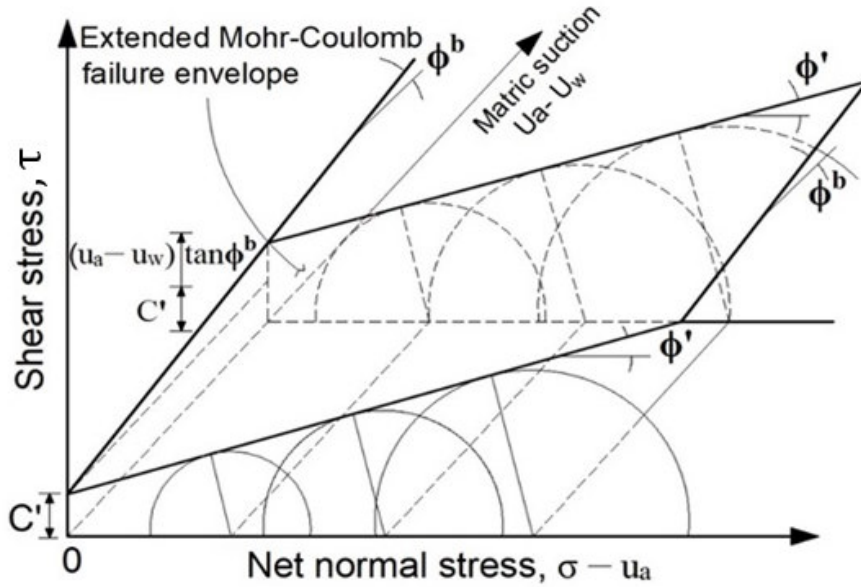


Figure 3.8: Extended Mohr-Coulomb failure envelope for unsaturated soils
(Fredlund and Rahardjo, 1993)

The extended Mohr-Coulomb failure envelope defines the shear strength of unsaturated soils. The failure envelope can be described by Equation 3.30. Figure 3.8 shows a planar failure envelope that intersects the shear strength axis, giving a cohesion intercept (c'). The envelope slope angles are (ϕ' and ϕ^b) for normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) axes. The increase in shear strength due to the increment of net normal stress is described by the friction angle (ϕ'). The increment in shear strength caused by the increment in matric suction is characterised by the angle (ϕ^b) when assuming linear failure conditions.

3.5.5 Fredlund et al.'s (1996) research on shear strength

Fredlund et al. (1996) determined the shear strength using a non-linear equation by integrating the soil-water characteristic curve (SWCC) in terms of dimensionless water content (θ_d) into the shear strength equation of unsaturated soils. The dimensionless water content is raised to a power (k) as a fitting parameter.

$$\tau_u = c' + (\sigma_f - u_a) \tan(\phi') + (u_a - u_w)_f (\theta_d^k) \tan(\phi') \quad [3.31]$$

Where:

θ_d = dimensionless water content defined as θ/θ_s
where θ is any volumetric water content

k = SWCC fitting parameter.

3.5.6 Garven and Vanapalli's (2006) studies on shear strength

Garven and Vanapalli (2006) developed a shear strength non-linear equation by integrating the volumetric water content (θ^k) into the unsaturated soil shear strength equation. The volumetric water content is raised to a power (k) that depends on the plasticity index (PI) of the soil material.

$$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \theta^k (u_a - u_w) \tan(\phi') \quad [3.32]$$

$$k = 0.00161 \text{ PI}^2 + 0.0975 \text{ PI} + 1$$

Where:

θ = volumetric water content [%]

k = fitting parameter depending on the plasticity index

PI = plasticity index [%]

3.5.7 Sheng *et al.*'s (2008) works on shear strength

Sheng *et al.* (2008) developed a mathematical equation to predict the shear strength of unsaturated soils very similar to the equation proposed by Fredlund and Rahardjo (1993) in Equation 3.30. Sheng *et al.* (2008) used a new approach to determine the friction angle (ϕ') for the effective normal stress and friction angle (ϕ^b) for matric suction. Sheng *et al.* (2008) compared the matric suction ($u_a - u_w$) to the matric suction value at saturation $(u_a - u_w)_{sa}$.

$$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) \quad [3.33]$$

if $(u_a - u_w) < (u_a - u_w)_{sa}$

$\tan(\phi') = \tan(\phi^b)$ if $(u_a - u_w) < (u_a - u_w)_{sa}$

if $(u_a - u_w) > (u_a - u_w)_{sa}$

$$\tan(\phi^b) = \tan(\phi') \left[\frac{(u_a - u_w)_{sa}}{(u_a - u_w)} + \left(\frac{(u_a - u_w)_{sa} + 1}{(u_a - u_w)} \right) \ln \frac{(u_a - u_w) + 1}{(u_a - u_w)_{sa} + 1} \right]$$

3.5.8 Guan *et al.*'s (2010) studies on shear strength

Guan *et al.* (2010) proposed an equation to predict the shear strength of unsaturated soils, very similar to Equation 3.30 developed by Fredlund and Rahardjo (1993). Guan *et al.* (2010) determined the frictional angle (ϕ') for the effective normal stress and friction angle (ϕ^b) for matric suction by different means. The achieved it by comparing the matric suction to the air entry value (AEV).

$$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) \quad [3.34]$$

$$\text{if } (u_a - u_w) < \text{AEV}, \phi' = \phi^b$$

$$\tau_u = c' + [(\sigma_n - u_a) + \text{AEV}] \tan(\phi') + [(u_a - u_w) - \text{AEV}] b \theta^k \tan(\phi') \quad [3.35]$$

$$\text{if } (u_a - u_w) \geq \text{AEV}$$

$$k = [\log(u_a - u_w) - \log(\text{AEV})]^y$$

Where:

$$(u_a - u_w) = \text{matric suction [kPa]}$$

$$\text{AEV} = \text{air entry value [kPa]}$$

$$\theta = \text{volumetric water content [\%]}$$

$$\text{PI} = \text{plasticity index [\%]}$$

$$k, y, b = \text{fitting parameters depending of the plasticity index}$$

3.5.9 Ahmadi and Toker's (2019) works on shear strength

Ahmadi and Toker (2019) proposed an exponential equation to predict the shear strength of unsaturated soils. The equation includes three parameters, two of which are conventional effective shear strength parameters, and the third of which is the maximum cohesion capillary (c''_{max}). The effective shear strength parameters (ϕ', c') can be evaluated utilising a typical saturated shear strength test. The third parameter (ψ_t) was obtained from the SWCC. The outcomes were validated by lab tests Ahmadi and Toker (2019) conducted and other published results. The results show that the proposed equation gives acceptable predictive values of the shear strength of unsaturated soils. However, the predictive equation is not suitable for unsaturated expansive soils because it does not consider the influence of the swelling properties on shear strength.

$$\tau_u = c' + (\sigma - u_a) \tan(\phi') + \left[1 - e^{\frac{-\psi}{\psi_t}} \right] \cdot \psi_t \cdot \tan(\phi') \quad [3.36]$$

$$c''_{max} = \tan(\phi') \cdot \psi_t$$

$$\chi = \frac{\psi}{\psi_t} \cdot \left[1 - e^{\frac{-\psi}{\psi_t}} \right]$$

Where:

ψ = suction [kPa]

ψ_t = transition suction [kPa]

ϕ' = friction angle of effective normal stress [°]

c''_{max} = maximum capillary cohesion [kPa]

χ = effective stress parameter

3.5.10 Bulolo and Leong's (2019) research on shear strength

Bulolo and Leong (2019) proposed a shear strength model for compacted soils. Direct shear tests were conducted on compacted samples under constant moisture content conditions. Before the shear test, the initial matric suction was estimated through the filter paper technique. The extended Mohr-Coulomb failure criterion with two-stress state variables was utilised. The results showed that the measured shear strength values are higher than the predicted shear strength values obtained from the proposed model. Thus, it provides a conservative estimate for unsaturated shear strength. The shear box test exhibits some limitations: the stress conditions are indeterminate during the test because stress conditions are not uniform, and the stress history cannot be measured. The shear strength distribution over the shear area of the specimen is not constant. The accuracy of results may be questioned because the soil specimen is not permitted to fail along the weakest plane, but rather along the split plane of the shear box. Further research is needed to develop an equation for predicting the shear strength of compacted expansive soils over a wide suction range and swelling properties.

$$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \chi S_r \tan(\phi') \quad [3.37]$$

Where:

τ_u = unsaturated shear strength [kPa]

c' = effective cohesion [kPa]

- σ_n = total normal stress [kPa]
 S_r = degree of saturation [kPa]
 u_a = pressure of pore air [kPa]
 $u_a - u_w$ = matric suction [kPa]
 ϕ' = effective angle of internal friction [°]
 χ = Bishop parameter
 χ_{max} = Bishop parameter of maximum average skeleton

3.5.11 Gao *et al.*'s (2020) works on shear strength

Gao *et al.* (2020) predicted the shear strength of unsaturated soils across a wide range of suction values. Equations 3.38 and 3.39 were developed to predict the shear strength of unsaturated soils. The proposed shear strength equation shows a good correlation between the measured and predicted shear strength for sand and silty soils. However, further investigations are necessary to derive a mathematical equation to predict the shear strength of compacted unsaturated expansive soils over a wide range of suction and swelling stress.

$$\tau_u = c' + (\sigma_n - u_a) \tan(\phi') + \chi S \tan(\phi') \quad S \leq S_{max} \quad [3.38]$$

$$\tau_u = \tau_{max} + (\chi S - \chi_{max} S_{max}) \tan(\phi_s) \quad S > S_{max} \quad [3.39]$$

Where:

- τ_u = unsaturated shear strength [kPa]
 τ_{max} = shear strength of maximum average skeleton [kPa]
 c' = effective cohesion [kPa]
 σ_n = total normal stress [kPa]
 S = soil suction [kPa]
 S_{max} = soil suction of maximum average skeleton [kPa]
 u_a = pressure of pore air [kPa]
 ϕ' = effective angle of internal friction [°]
 ϕ_s = effective angle of internal friction when
 soils becomes overconsolidated, where $0 \leq \phi_s \leq \phi'$ [°]
 χ = Bishop parameter
 χ_{max} = Bishop parameter of maximum average skeleton

3.6 Past studies on shear strength of expansive soils

Saile and Bucher (1984) examined the shear strength properties of tropical clays and presented their findings based on consolidated undrained triaxial tests. Saile and Bucher (1984) concluded that highly expansive soils display a high shear strength, and water content significantly impacts the shear strength of expansive soils. Zewdie (2004) investigated the shear strength properties of expansive soils; unsaturated and saturated soil specimens were subjected to UU triaxial shear strength tests collected from the Addis Ababa area. The findings indicated that for unsaturated soils, cohesion values of expansive soils range from 30 kPa to 150 kPa, and the friction angle ranges from 3° to 25°. In saturated soil specimens, cohesion values range from 55 kPa to 94 kPa. However, the investigation does not consider the impact of the swelling stress on the shear strength of expansive soils.

Zhan and Ng (2006) studied the shear strength parameters of unsaturated expansive soils from China by performing suction-controlled direct shear tests on compacted specimens and field samples. The tests were conducted using clay/silt soil material with medium swelling potential and intermediate plasticity. Zhan and Ng (2006) found that the dilatancy (expansion in volume due to shear deformation) of expansive soils increases with the augmentation of applied suction for both compacted specimens and field samples. Matric suction contributes to the shear strength of expansive soils through two unique mechanisms: the impact of suction on dilatancy and the capillary force on the inter-particle normal stress value. Due to the first mechanism, the suction's contribution to the peak shear strength for clay is more significant than the post-failure shear strength, especially at high suction values. The suction's contribution to shear strength post-failure is consistent with the compacted soil specimen. The suction's contribution to the shear strength for compacted expansive soils exceeds that of compacted kaolin at suction values under 100 kPa. However, the volume expansion is not induced solely by shear deformation; the swelling behaviour of expansive soils contributes to volume changes. Therefore, assessing the influence of swelling stress on shear strength properties is fundamental.

Bai and Liu (2012) conducted a study on the shear strength of unsaturated expansive soils. The filter paper technique was utilised to establish the soil material's SWCCs. Subsequently, a direct shear test was performed on soil specimens. The results

indicate that the unsaturated shear strength parameter (ϕ^b) is equivalent to the effective friction angle (ϕ') when the soil is near saturation, and (ϕ^b) decreases as the soil specimens become drier. This process offers a simplified method for assessing the shear strength of unsaturated soils. However, this approach did not account for the influence of swelling properties on the shear strength of soil material. An extended investigation into the effects of swelling stress on shear strength characteristics is necessary to gain a deeper understanding of the behaviour of expansive soils.

Chen *et al.* (2012) conducted a study on the impact of water content and dry unit weight on the shear strength parameters of expansive soils. Chen *et al.* (2012) performed shear box tests on soil specimens. Their results reveal that the friction angle and cohesion increase as the dry unit weight increases, but decrease as the initial water content increases. Predictive Equations 3.40 and 3.41 for the friction angle and soil cohesion were developed using multiple regression analysis.

$$\log \phi' = -(1.86w + 28) \times 10^{-2} \gamma_d + 1.41 \times 10^{-2} w - 1.15 \quad [3.40]$$

$$c' = (-20.5w + 690) \gamma_d + 24.6w - 820 \quad [3.41]$$

Where:

W = initial water content [%]

γ_d = dry density [kN.m^{-3}]

c' = cohesion [kPa]

ϕ' = friction angle with respect to effective normal stress [$^\circ$]

The study demonstrates that the shear strength characteristics of expansive soils can indeed be determined using common soil properties like dry density and water content. However, since expansive soils are generally in unsaturated conditions, the contribution of soil suction and swelling stress should also be considered as independent variables to realistically predict shear strength.

Chowdhury (2013) examined the shear strength parameters of expansive soils from Regina in western Canada. Shear box tests were conducted on field-compacted and

natural specimens to quantify the stress-strain behaviour and measure the shear strength characteristics. The soil specimens on the dry side of optimum moisture content display brittle stress-strain behaviour, while those on the wet side of optimum moisture content exhibit ductile behaviour. Cohesion (c') follows the compaction curve, reaching a maximum value of 66 kPa at optimum water content. The friction angle (ϕ') decreases from 44° on the dry side to 27° at optimum water content and remains constant on the wet side. The friction angle of suction (ϕ^b) was evaluated utilising the direct shear test on unsaturated soil specimens and estimated from the SWCC with a direct shear test on saturated soil specimens. The relation between the measured (ϕ^b) values and experimental values shows a linear correlation. Additionally, the SWCC displays a bimodal shape with an air entry value (AEV) ranging from 10 kPa to 100 kPa. The study revealed that the dry unit weight and consistency limits influence the shear strength of expansive soils. However, the influence of the swelling characteristics on the shear strength was overlooked.

3.7 Influence of swelling on shear strength

Sailie and Bucher (1984) conducted the CU test on undisturbed soil specimens. It was concluded that highly swelling clay exhibits high shear strength. However, these results contradict the findings of the investigation carried out by Calabresi and Scarpelli (1985) on the impact of swelling potential on shear strength. The CU test was conducted on slightly fissured Italian clay. The soil specimens were pre-swelled in a laboratory under lower confining pressure. The results indicate that an increase in swelling stress reduces the cohesion of clay with an increment in water content. In a similar study, Wiseman (1993) investigated the shear strength of expansive soils. The outcomes revealed that volumetric changes and shear resistance in expansive soils depend on soil wetting conditions. Volumetric changes are induced by variations in water content in the soil material, indicating an inseparable relationship between volumetric variation and shear strength in expansive soils. Wong (1998) investigated the impact of swelling on the shear strength of expansive soils through laboratory tests. Tests including free swell, one-dimensional oedometer swell, and triaxial tests were conducted on soil specimens at different swelling values. The results demonstrate that cohesion and Young's modulus decrease as swelling stress increases.

Al-Mhaidib and Al-Shamrani (2000) conducted an experimental study on the influence of swelling on the shear strength of shale samples. Soil specimens were subjected to different water contents and tested under confining pressures of 25 kPa, 50 kPa, 100 kPa, and 150 kPa. The shear strength test was performed when soil specimens reached predetermined swelling magnitudes of 25 %, 50 %, 75 %, and 100 % of the final vertical swell. The deviator stress increases as axial strain increases for swelling magnitudes of 25 %, 50 %, and 75 % of final swelling. However, once the soil specimen reaches the final swell value of 100 %, the deviator stress decreases, while axial strain continues to increase. Swelling diminishes the influence of deviator stress at similar confining pressure and water content. Finally, when the soil specimens reach the ultimate swelling of 100 %, the deviator stress is about 10 % of the shear strength value of non-swelled specimens. Figure 3.9 shows the deviator stress versus axial strain graph for the sample at 14 % of initial water content and under 100 kPa confining pressure.

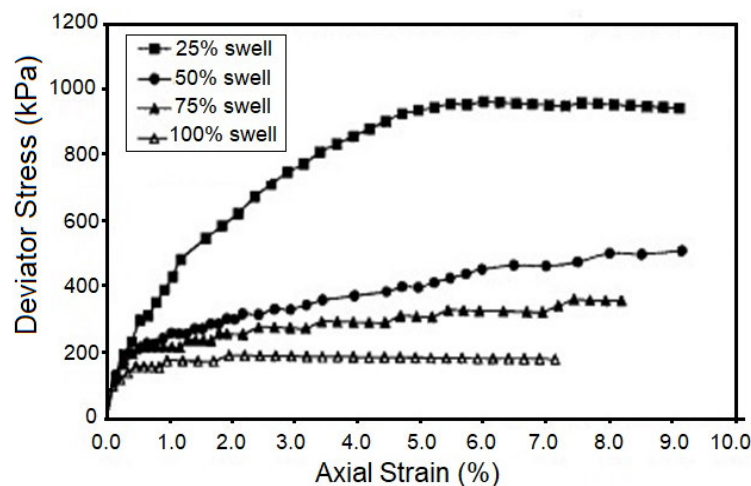


Figure 3.9: Deviator stress versus axial strain (Al-Mhaidib and Al-Shamrani, 2000)

Domitrović and Zelić (2013) examined the relationship between shear strength and swelling properties of bentonite. Swelling behaviour was measured through consolidometer testing under effective-stress conditions 50 kPa, 100 kPa, and 200 kPa. The results demonstrated that the specimen exhibits the highest swelling magnitude and relative vertical deformation under the lowest normal stress. A direct shear test was performed on granular bentonite to assess the influence of swelling on shear strength. The soil specimens were tested under various normal stress values. The results indicate that both peak and residual strength values decrease with reduced

normal stress and the increment of swelling potential. Figure 3.10 illustrates that both peak and residual shear strength decrease with increasing moisture content.

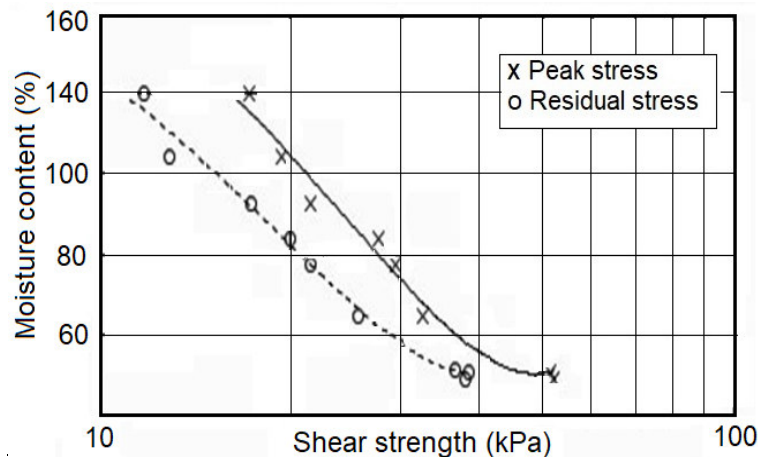


Figure 3.10: Final moisture content vs. shear strength (Domitrović and Zelić, 2013)

Deliktaş (2016) examined the influence of swelling on the shear strength of expansive soils. An artificial expansive soil was prepared by combining bentonite and kaolinite for initial testing. Swelling tests and unconfined compression tests were conducted to assess the rate of swell. Soil specimens were sheared without permitting the swelling to occur. Additionally, soil specimens were sheared when the vertical swell was completed. Shear strength values were estimated for 10 %, 15 %, 20 %, 25 %, and 75 % of swelling. The findings indicated that shear strength reduced by 90 % when the total vertical swell value reaches 100 %.

3.8 Shear strength and suction relationship

Escario (1980) and Escario *et al.* (1989) conducted direct shear tests on expansive soil specimens collected from various areas across Spain. Escario (1980) and Escario *et al.* (1989) investigated correlations between soil suction and compaction characteristics, including cohesion (c') and friction angle (ϕ'). Figure 3.11 illustrates the influence of soil suction on the cohesion of expansive soil. Cohesion of the soil increases with the increment of soil suction for the four soil specimens. The compacted expansive soil exhibits the highest cohesion value, while the clay-sand specimen displays the lowest cohesion value. Thus, soil compaction enhances soil cohesion. Figure 3.12 depicts the impact of soil suction on the friction angle of expansive soil. The friction angle of soil increases with the augmentation of soil suction for the four soil specimens. Similarly, the compacted specimen presented the smallest friction

angle value, while the compacted clay-sand soil exhibited the highest friction angle value. Soil compaction thus influenced the magnitude of the friction angle.

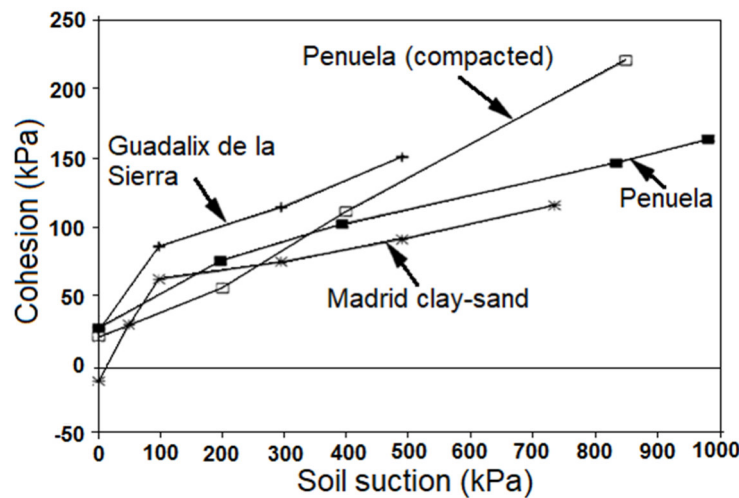


Figure 3.11: Relationship between soil cohesion and soil suction of unsaturated clay (Escario *et al.* 1989)

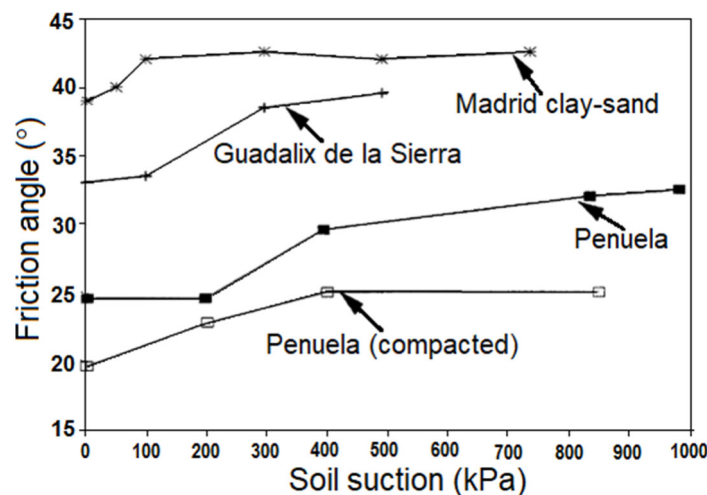


Figure 3.12: Relationship between friction angle and soil suction of unsaturated clay (Escario *et al.* 1989)

Escario *et al.* (1989), Gan *et al.* (1988), and Vanapalli *et al.* (1996) reported that many test outcomes demonstrated a non-linear variations of shear strength beyond the linear state. When suction is lower than the AEV, the soil is saturated, and the suction increment is comparable to an increase in effective normal stress. When soil suction surpassed the AEV, the shear strength envelope exhibited a non-linear behaviour. Figure 3.13 illustrates the influence of soil suction on the shear strength of fine-grained soils and sand within the linear state and non-linear state.

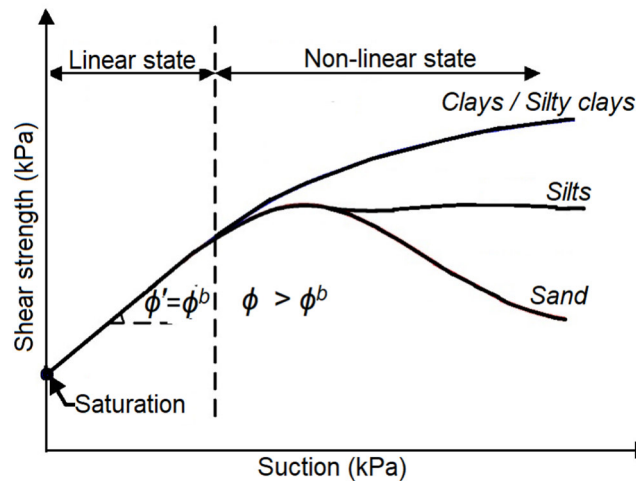


Figure 3.13: Types of shear strength for unsaturated soils (Gao *et al.* 2020)

Toll (1990) investigated the correlation between the shear strength of compacted unsaturated granular soil and soil suction. The results revealed that the precise nature of the relationship between shear strength and suction varies with soil properties such as soil initial fabric, degree of saturation, void ratio, and dry density. Thus, the type of soil significantly impacts the relationship between soil suction and shear strength. No generalised relationship exists between shear strength and matric suction. Another investigation conducted by Miao *et al.* (2007) explored the correlation between shear strength and suction through theoretical and experimental investigations of shear strength of unsaturated expansive soil. Results indicate that expansive soil shear strength increases with the increment of soil suction. The variation in soil suction primarily depends on water content within the soil and other physical soil properties. Tilgen (2003) performed direct shear tests on compacted and soaked clays at the dry side, optimum moisture content, and wet side. Soil suctions were estimated using the filter paper technique. Results show that as soil suction decreases, cohesion values increase up to optimum water content, while friction angle diminishes due to soil-granular fabric. Beyond optimum water content, both friction angle and cohesion values decrease. The friction angle and cohesion obtained from compacted soil specimens were reported higher than those from soaked specimens. Additionally, total suction and matric suction decrease as water content increases at the dry side of optimum moisture content, confirming a direct correlation between suction and shear strength. Similarly, Zhan and Ng (2006) reported that matric suction impact could lead to the hardening of unsaturated soils without modifying the internal angle of friction. Unsaturated shear strength parameters increase non-linearly with suction increment, indicating a lack of proportionality between shear strength and soil suction. These

findings coincide with the study by Kayadelen *et al.* (2007) on the relation between shear strength and soil suction in unsaturated soils. Triaxial tests were performed on soil specimens using the ATT to control pore air and pore water pressures via a transducer. The results describe the significant contribution of matric suction to the shear strength of unsaturated soils, with a non-linear relation between shear strength and suction.

Ye *et al.* (2010) studied expansive soil using soil-water property tests. A modified triaxial test with suction control was performed to obtain the shear strength characteristics and SWCC. Results reveal that unsaturated shear strength increases with controlled suction increment during the test. In a similar study, Çokça and Tilgen (2010) explored the correlation between soil suction and shear strength of compacted clay from Ankara at different water contents. Soil specimens were analysed on the dry side, optimum moisture content, and wet side. Direct shear tests were conducted, along with suction tests using the filter paper technique. Shear strength values at specific normal stress were measured. The outcomes indicated that shear strength increases with the increment of soil suction, and the correlation between moisture content and soil suction curves mirrors that of shear strength. Ye *et al.* (2006) studied shear strength properties on Shanghai clay through modified triaxial tests. At lower matric suction values, shear strength is proportional to matric suction. These results affirm the use of a double-variable stress theory for unsaturated shear strength of clay. Unsaturated shear strength increases as matric suction increases within the controlled suction range.

Finally, while there is no generalised relationship between shear strength and soil suction, shear strength does increase as matric suction increases within the controlled suction range. The precise nature of the relationship between shear strength and soil suction varies with soil properties, demonstrating the significant impact of soil type on the soil suction-shear strength relationship.

3.9 Impact of soil fabric on shear strength

Pells *et al.* (1975) reported that soil particles within the assemblage were held together by soil moisture deficiency and were acting as individual particles. The friction angle of larger grain sizes was higher than that of smaller grain sizes in the soil material.

Seed *et al.* (1961) stated that interparticle forces due to external forces applied during soil formation and stress history were related to the soil fabric of compacted cohesive soil. Toll (2000) emphasised that soil fabric played an essential role in the mechanical behaviour of compacted soils. Compacted clayey material at the dry side of the optimum moisture content displayed an aggregated fabric. The presence of aggregations caused the soil material to act more coarsely due to grading. Soils compacted to a saturation level of $\geq 90\%$ were usually non-aggregated. As the saturation level dropped, the quantity of aggregation rose rapidly, reaching a fully aggregated condition for a saturation level below 50 %. Toll (2000) mentioned that although compacted materials exhibited an aggregated structure on the dry side of optimum moisture content, there was no aggregation on the wet side and optimum moisture content. However, aggregation levels increased as the quantity of water decreased below the optimum moisture content. When the water content fell below 70 % of the optimum moisture content, the material was fully aggregated with no matrix material.

Warkentin and Yong (1962) investigated the shear strength of expansive soils and mentioned that the shear strength of expansive soil reduced when the void ratio increased within the soil material. In addition, Wang *et al.* (2014) conducted an experimental investigation on compacted clayey samples from Changsha, China. The shear strength parameters of soil specimens were obtained through a direct shear test. The results demonstrated that the relationship between the friction angle and void ratio follows a logarithmic correlation with a determination coefficient of 90 %. The friction angle decreased as the void ratio increased. Also, the cohesion decreased as the void ratio increased, and showed a strong correlation with a determination coefficient of 96 %.

3.10 Shear strength and moisture content relationship

Casagrande and Hirschfeld (1960) investigated the change of shear strength with varying initial degrees of saturation by performing undrained tests on clay specimens prepared at approximately the same initial dry density. Figure 3.14 illustrates that the shear strength of inorganic clay reduced with an increment in the degree of saturation and vice versa. Saile and Bucher (1984) examined the shear strength properties of tropical clays based on consolidated undrained triaxial tests. It was reported that in

highly expansive soils, water content significantly impacted the shear strength and led to its reduction as water content increased. The findings were in line with studies conducted by Toll (1990) on the influence of the degree of saturation on the shear strength of unsaturated soils, indicating that shear strength decreased as the degree of saturation increased.

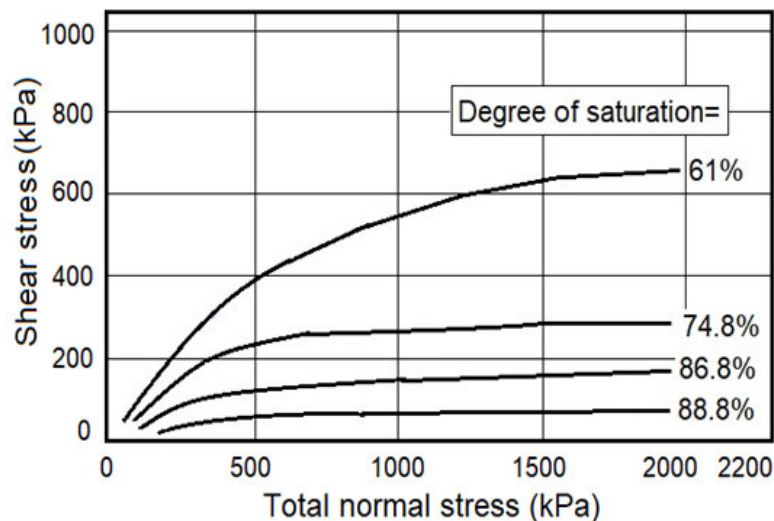


Figure 3.14: Shear strength variation with initial degree of saturation (Casagrande and Hirschfeld, 1960).

Stark and Duncan (1992) studied clay soil from San Luis Dam in California. The results showed a decrease in cohesion from 263 kPa to 0 and a decrease in friction angle from 39° to 25° upon water addition. Experimental studies by Tilgen (2003) demonstrated that shear strength increased as water content increased on the dry side of optimum moisture content and decreased with an increase in water content on the wet side. In another study, Kong and Tan (2000) investigated the shear strength characteristics of compacted unsaturated expansive soils collected across the Hubei area in China. The findings indicated that when the initial water content in compacted samples was less than the plasticity index, shear strength changed linearly with the water content. Additionally, the friction angle exhibited a critical point near the plastic limit. Beyond the plastic limit, the impact of water content on shear strength was marginal. In general, soil cohesion was reduced with the addition of water content. The friction angle decreased significantly when the water content was less than the plastic limit, while the friction angle tended toward a constant value when the water content surpassed the plastic limit.

Zewdie (2004) conducted an experimental study on the shear strength properties of expansive soils in Ethiopia. Triaxial tests were performed on unsaturated soil specimens. The outcomes revealed that the shear strength of unsaturated samples reduced with an increment in water content, emphasising the significant influence of water content on the shear strength of expansive soils. In a similar study, Wang *et al.* (2014) investigated compacted clayey samples from Changsha, China. The shear strength parameters of soil specimens were obtained through direct shear tests. The results showed that cohesion decreased logarithmically with an increment in water content, with a determination coefficient of 96 %. The friction angle also reduced logarithmically with an increment of water content, with a determination coefficient of 95 %.

3.11 Relationship shear strength and mineralogy of clay

Warkentin and Yong (1962) investigated the shear strength of expansive soils composed of sodium montmorillonite and calcium montmorillonite. The results demonstrated that the shear strength of calcium montmorillonite was smaller than that of sodium montmorillonite at an identical volume ratio. Escario (1980), and Escario and Sáez (1986) conducted an experimental study on the effect of clay mineralogy on the shear strength of expansive soils. Laboratory tests, including triaxial testing and X-ray analysis, were performed on expansive soil specimens. The results revealed that the presence of clay minerals such as smectite led to a reduction in shear strength. In another research, Morrow *et al.* (1984) reported that as the quantity of clay minerals increased, the shear strength of clay soils decreased. The outcome was justified by the fact that clay minerals were inherently weaker than rock-flour gouges made out of crushed granitic material.

3.12 Summary of parameters affecting the shear strength of soils

Tables 3.4a, 3.4b, 3.4c and 3.4d summarised some of the parameters that influenced the shear strength.

Table 3.4a: Summary of parameters affecting the shear strength of soils

Parameters	Influence of parameters on shear strength	References
Cohesion (c')	The shear strength increases with an increment of soil cohesion, and soil cohesion (c') portrays a significant effect on shear strength.	Terzaghi (1936), Bishop <i>et al.</i> (1960), Fredlund <i>et al.</i> (1978), Escario (1980) and Escario <i>et al.</i> (1989), Lamborn (1986), Fredlund and Rahardjo (1993), Fredlund <i>et al.</i> (1996), Öberg and Sällfors (1997), Sheng <i>et al.</i> (2008), Guan <i>et al.</i> (2010), Bulolo and Leong (2019), Ahmadi and Toker (2019).
Friction angle (ϕ')	The friction angle (ϕ') of soil associated with the normal stress exhibits a relevant influence on the shear strength.	Terzaghi (1936), Bishop <i>et al.</i> (1960), Fredlund <i>et al.</i> (1978), Fredlund and Rahardjo (1993), Lamborn (1986), Fredlund <i>et al.</i> (1996), Öberg and Sällfors (1997), Sheng <i>et al.</i> (2008), Guan <i>et al.</i> (2010), Bulolo and Leong (2019), Ahmadi and Toker (2019).
	The friction angle (ϕ') increases with the increase of the matric suction in the unsaturated expansive soils sample.	Escario (1980) and Escario <i>et al.</i> (1989).
Friction angle (ϕ^b)	The angle (ϕ^b), which signifies the rate of shear strength increase with changes in matric suction of the soil, significantly influences the shear strength of unsaturated soils.	Fredlund <i>et al.</i> (1978), Lamborn (1986), Fredlund and Rahardjo (1993), Sheng <i>et al.</i> (2008), Guan <i>et al.</i> (2010), Bulolo and Leong (2019), Ahmadi and Toker (2019).

Table 3.4b: Summary of parameters affecting the shear strength of soils

Parameters	Influence of parameters on shear strength	References
Water content	Water content has a significant influence on shear strength, with the shear strength decreasing as the water content in the expansive soil increases.	Saile and Bucher (1984), Calabresi and Scarpelli (1985), Toll (1990), Stark and Duncan (1992), Chen <i>et al.</i> (2012), Chowdhury (2013), Domitrović and Zelić (2013).
	The cohesion and friction angle reduce when the water content increases and exhibit a higher strength logarithmic correlation with a determination coefficient of $R^2 > 90\%$.	Wang <i>et al.</i> (2014).
Plasticity index (PI)	The plasticity index of soil mass exhibits an influence on the shear strength of unsaturated soils.	Vanapalli and Fredlund (2000), Garven and Vanapalli (2006), Chowdhury (2013).
Dry unit weight (γ_d)	The dry unit weight of soil influences the shear strength of expansive unsaturated soils, with the shear strength increasing as the dry unit weight increases.	Escario (1980), Escario <i>et al.</i> (1989), Chen <i>et al.</i> (2012), Chowdhury (2013).
Degree of saturation	The shear strength of unsaturated soil reduces when the degree of saturation increases. The degree of saturation significantly influences the shear strength of expansive soils.	Casagrande and Hirschfeld (1960), Zewdie (2004).

Table 3.4c: Summary of parameters affecting the shear strength of soils

Parameters	Influence of parameters on shear strength	References
Swelling stress (P_s)	The swelling stress significantly influences the shear strength of expansive soils. The shear strength reduces when the swelling stress increases upon the addition of water.	Calabresi and Scarpelli (1985), Wiseman (1993), Wong (1998), Al-Mhaidib and Al-Shamrani (2000), Domitrović and Zelić (2013), Deliktaş (2016).
	The swelling stress significantly influences the shear strength of expansive soils. When the specimens attained the ultimate swelling of 100%, the deviator stress was about 10% of the shear strength of the non-swelled soil specimens.	Al-Mhaidib and Al-Shamrani (2000).
	The shear strength and the swelling stress developed within expansive soils are not dissociable.	Wiseman (1993).
	The shear strength of expansive soils can be reduced by 90 % when the total vertical swell is 100 %.	Deliktaş (2016).
Void ratio (e)	As the void ratio increases, the shear strength of expansive soils reduces. The void ratio plays a significant role in influencing the shear strength of the soil material.	Warkentin and Yong (1962), Wang <i>et al.</i> (2014).
Air entry value (AEV)	The AEV influences the relationship between shear strength and matric suction. The relationship is linear when $U_a - U_w < \text{AEV}$ and becomes non-linear when $U_a - U_w > \text{AEV}$.	Khalili and Khabbaz (1998), Escario <i>et al.</i> (1989), Gan <i>et al.</i> (1988), Vanapalli <i>et al.</i> (1996), Tekinsoy <i>et al.</i> (2004), Guan <i>et al.</i> (2010).

Table 3.4d: Summary of parameters affecting the shear strength of soils

Parameters	Influence of parameters on shear strength	References
Matric suction ($u_a - u_w$)	The matric suction significantly influences the shear strength of unsaturated soils. The shear strength increases when the matric suction increases.	Bishop <i>et al.</i> (1960), Gan <i>et al.</i> (1988), Escario <i>et al.</i> (1989), Fredlund and Rahardjo (1993), Vanapalli <i>et al.</i> (1996), Ye <i>et al.</i> (2006), Zhan and Ng (2006), Miao <i>et al.</i> (2007), Kayadelen <i>et al.</i> (2007), Ye <i>et al.</i> (2010), Guan <i>et al.</i> (2010), Çokça and Tilgen (2010), Chowdhury (2013).
	The relationship between the shear strength and the matric suction is non-linear. Therefore, there is no proportionality between the shear strength and soil suction.	Toll (1990), Zhan and Ng (2006), Kayadelen <i>et al.</i> (2007), Bai and Liu (2012).
Clay mineralogy	The fraction and type of clay mineral significantly influence the shear strength of expansive soils. The shear strength reduces when the quantity of clay mineral increases. However, the calcium montmorillonite shear strength is smaller than the sodium shear strength of an identical volume ratio.	Warkentin and Yong (1962), Escario (1980), Morrow <i>et al.</i> (1984), Escario and Sáez (1986).

3.13 Conclusion

Bishop *et al.* (1960) were pioneers in developing a shear strength equation for unsaturated soils by incorporating the contribution of matric suction into the initial saturated shear strength equation proposed by Terzaghi (1936). Subsequent notable works by Fredlund *et al.* (1978), Fredlund and Rahardjo (1993), and Fredlund *et al.* (1996) were significant in advancing the unsaturated soil mechanics. Their works included adding the friction angle associated with matric suction to the equation developed by Bishop *et al.* (1960). Garven and Vanapalli (2006) developed a non-linear shear strength equation by integrating the volumetric water content (θ^k) into the unsaturated shear strength equation. In this equation, the volumetric water content was raised to the power of (k) and was dependent on the plasticity index. Guan *et al.* (2010) formulated a shear strength equation for unsaturated soils similar to the equation by Fredlund and Rahardjo (1993), introducing a novel approach to evaluate the (ϕ', ϕ^b) values. Other contributions involved incorporating the matric suction fitting parameters such as degree of saturation, AEV, volumetric water content, and plasticity index. Calabresi and Scarpelli (1985) examined the influence of swelling on shear strength, revealing that the swelling stress reduced the clay cohesion with increasing water content. Wiseman (1993) noted that both volumetric changes and shear strength of expansive soil were dependent on wetting conditions and showed that the two behaviours were not dissociable. Wong (1998) observed that cohesion and Young's modulus decreased as swelling stress increased. Al-Mhaidib and Al-Shamrani (2000) investigated the effect of swelling on shear strength. The findings showed that the shear strength decreased with the swelling of soil material. Specimens that attained the ultimate vertical swell exhibited a 10 % reduction in shear strength compared to non-swelled soil specimens. Deliktaş (2016) reported a substantial 90 % reduction in shear strength as expansive soil specimens reached ultimate swelling. In conclusion, the impact of swelling stress on the shear strength of unsaturated expansive soil is significant. Therefore, it is fundamental to consider the contribution of swelling stress in the constitutive relation of shear strength of unsaturated expansive soils.

CHAPTER 4: SHEAR STRENGTH AND SUCTION MEASUREMENT OF UNSATURATED SOILS

PART 1: SHEAR STRENGTH MEASUREMENT

4.1 Introduction

Shear strength is a fundamental characteristic of engineered soil material. While the underlying physics governing soil behaviour remain too complex to be fully described or modelled, several approaches have produced satisfactory results. These approaches focus on specific aspects of soil behaviour and provide highly accurate estimations of soil strength, compressibility, volume change behaviour, pore water movement and pressure, chemical interactions, and deformation behaviour over a range of strain magnitudes (Blight, 2013; Fondjo et al. 2023a; Fondjo et al. 2024). However, no single model can encompass all these aspects. The geotechnical engineer must ascertain the most critical behaviour pertinent to the problem at hand, and subsequently employ the most suitable model to address it. Estimating the shear strength of soils under unsaturated conditions is no exception. The models presented below offer suitable solutions for a variety of specific conditions, especially when considering ultimate shear strength influenced by negative pore pressures resulting from unsaturated conditions (Fredlund and Rahardjo, 1993; Fredlund et al. 2012; Blight, 2013). Some applications of shear strength include slope stability, bearing limit, soil stabilisation, and lateral pressure (Fredlund et al. 2012; Fondjo et al. 2023a; Fondjo et al. 2024).

Shear strength is usually defined as the stress state where large, often irreversible, strains occur. Since most soils exhibit relative incompressibility under hydrostatic stress ($\sigma_x = \sigma_y = \sigma_z$), the stress state of greater concern involves conditions of unequal stress magnitudes. In such conditions, shear stresses and even tensile stresses may occur. Soil's shear strength is considerably weaker, and hence, shear stress conditions generally govern soil strength limits. Most soil models establish a stress limit state (in 2 or 3-dimensional stress space) defined by a yield line (2D) or surface (3D) beyond which the soil undergoes plastic deformation. This plastic deformation or flow might be localised and potentially limited by neighbouring soils that remain below the limit state. However, if the plastic region expands to a point where it is unrestrained, collapse will follow. In design terminology, this represents the system's limit state.

Ultimate bearing capacity, slope failure, and retaining wall collapse, are examples. An exception to this form of plastic behaviour is observed in swelling-shrinking clays, where large strains occur volumetrically due to moisture influx in all directions. The plastic state may be reached, but it is generally in a different region of the defined yield line or surface. Alternatively, swelling conditions could alter the size and shape of the yield surface (usually reducing it), allowing shear stress conditions to exceed it, resulting in plastic strains similar to the aforementioned case.

The interaction between pore water availability and the yield surface is complex, leading to the application of simplified approximations, as discussed in the following sections. In situ methods includes shear vane tests or penetrometers. These methods indirectly evaluate shear strength through correlations with laboratory results or back-calculation from actual failures. Laboratory tests directly yield shear strength parameters. This section reviews various laboratory methods for measuring shear strength, their limitations, advantages, and the selection of appropriate apparatus for the research work.

4.2 Soil shear strength testing

The extension of the Mohr-Coulomb shear strength concept to unsaturated soils necessitate the initial estimation of classical shear strength parameters (c' , ϕ'). These parameters are estimated using conventional triaxial or direct shear apparatus. In unsaturated conditions, the testing equipment requires modification to produce matric suction conditions within the specimen. In a given test, the specimen is equilibrated with moisture (and suction) under isotropic (hydrostatic) confinement. Vertical (triaxial) or horizontal (direct shear) loads are then slowly applied. Further consideration must be given to the redistribution of moisture/suction during loading, similar to drained triaxial testing, albeit potentially at a slower rate due to the dependency of pore pressure redistribution time on hydraulic conductivity. Unsaturated specimens have significantly lower conductivity as initial available moisture diminishes.

4.3 Triaxial testing

4.3.1 Triaxial apparatus

The triaxial equipment was developed by Bishop and Henkel (1962), and subsequent enhancements included the incorporation of a mechanical framework, an electronic control system, and a data logging device. The triaxial apparatus serves the purpose of evaluating shear strength parameters, the characteristic stress factor (K_o), soil stiffness, and more. In the context of testing unsaturated soils, the triaxial cell is coupled with a suction control device such as the Axis Translation Technique (ATT). The use of the osmotic control framework in triaxial testing was documented by Cui and Delage (1996), while the vapour control method was described by Blatz and Graham (2000) and Chávez *et al.* (2009). Certain types of triaxial equipment have been developed with psychrometers (Tang *et al.* 2002 and Thom *et al.* 2008), as well as high-capacity tensiometers (Ridley, 2002, Meilani *et al.* 2002, and Jotisankasa *et al.* 2007). Figure 4.1 shows a schematic of test specimen connections when using GDS Method A for unsaturated triaxial testing of soils.

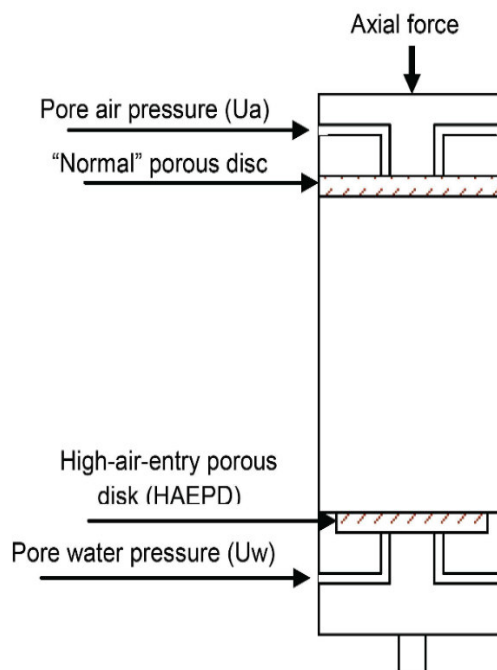


Figure 4.1: Schematic of test specimen connections when using GDS Method A (Courtesy of GDS Instruments, London.)

4.3.2 Triaxial test conditions

4.3.2.1 Triaxial test without suction controlled (Gueye, 1993)

Triaxial tests without suction control involve conducting tests on consolidated and directly sheared specimens without regulating saturation or suction. Soil suction is estimated based on the water content during shearing, employing soil-water characteristic curves (SWCC). These tests are conducted under undrained conditions using a conventional cell. Triaxial tests without controlled suction are undrained tests on unsaturated soil specimens, with no monitoring of suction at various saturation degrees. The porous stones are identical to those used in triaxial tests on saturated soils, and the soil specimen fails under undrained conditions. The unique difference is the absence of a saturation phase. In this scenario, suction varies in a manner similar to void ratio and degree of saturation during shearing. Initial and final suction magnitudes are determined utilising the SWCC. This approach is straightforward, efficient, and does not require the modification of the conventional triaxial cell.

4.3.2.2 Triaxial test with suction controlled (Fredlund *et al.* 1978)

A triaxial test involving controlled suction is conducted on saturated specimens that have been desiccated using air pressure equivalent to suction. These specimens are subjected to repeated shear loading tests. The tests are performed in modified cells, where porous stones are replaced by tensiometers. These cells incorporate fine porous stones that facilitate the passage of water, not air, and a valve allowing the injection of air under pressure and maintaining suction control. The apparatus utilised for these tests is slightly adapted from conventional equipment, featuring an additional external connection to the cell for injecting pressurised air. Water drainage at the sample's bottom is achieved through fine porous stones that become saturated by capillary action. The desaturation tube of the specimen is exposed to the open air, maintaining water at atmospheric pressure, while matric suction is achieved via air pressure. The fine porous stones employed exhibit air impermeability under specific pressures. Once the stabilisation is completed, the suction is assumed to be uniform in the specimen, corresponding to the AEV. The sample is sheared at a rate of deformation that makes it possible to keep the suction constant without excess pore pressure. The variation of specimen volume is measured during consolidation when suction changes. During the shearing and desaturation phases, conducted through controlled suction by imposing air pressure, the variation in volume is measured. The

loading of the sample starts after desaturation, with no further water drainage for the imposed air pressure. The specimen is loaded until it approaches failure, then it is unloaded. After, a higher suction value is achieved by increasing air pressure. The sample continues to consolidate as the air pressure expels water. After stabilisation, the specimen is reloaded until the deviator stress approaches the failure value.

4.3.3 Triaxial test for unsaturated soils

Several factors influence the results of unsaturated shear tests. Establishing the variation of shear strength with respect to matric suction requires a substantial number of tests. The use of advanced triaxial equipment to measure the shear strength for unsaturated soils is costly and complex (Blight, 2013). The duration of unsaturated triaxial tests exceeds that of equivalent tests on saturated soils. This extended period is necessary to attain equilibrium in soil tests before the commencement of the test itself. The process for testing the unsaturated soils proves to be more demanding due to the involvement of two liquid phases – water and air. The difference in test duration primarily stems from the reduction in the coefficient of permeability in unsaturated soils. A modification of the conventional triaxial setup, allowing for the consideration of pore air and pore water pressure estimation, becomes imperative to measure unsaturated shear strength parameters.

4.3.3.1 Consolidated Drained (CD) test (Fredlund et al. 2012)

In the Consolidated Drained (CD) test, the original stress state of the soil specimen is obtained by applying a recommended confining pressure (σ_c), along with pore-air stress (u_a), and pore-water stress (u_w). The application of confining pore-air and pore-water stresses is accomplished by opening the appropriate valves on the triaxial control board. Throughout the test, the remaining valves on the base platform remain closed, except during the flushing of diffused air beneath the high-air-entry disk. Radial deformation and the vertical failure allow determining the general volume change of the specimen. The inflow and outflow of water in the soil specimen is recorded using a marker. The water volume change indicator valve is opened during the consolidated drained test, except when diffused air is being flushed from the base plate. However, determining air volume change is not a common practice. The consolidation process

reaches an equilibrium state when either there is no water flowing through the specimen or the total volume fluctuates.

4.3.3.2 Consolidated Undrained (CU) test (Fredlund et al. 2012)

The soil specimen is tested following the method described for the consolidated drained test. Once equilibrium conditions are achieved under the applied stresses (σ_c , u_a , and u_w), undrained conditions during shearing are initiated by closing the valves on the pore-air and pore-water lines. The soil specimen is then tested under undrained conditions for both the air and water phases. Pore-water stress induced during shearing can be determined through the stress transducer mounted on the base plate. If possible, a pore-air pressure transducer should also be affixed to the loading cap when estimating changes in pore-air pressure. The diffused air volume is calculated during the steady water-content test. Nevertheless, it is feasible to correct the determination of pore-water pressures for air diffusion. Obstacles related to air diffusion may present a significant argument for a limited number of consolidated undrained tests involving pore-air and pore-water stress estimation.

4.3.3.3 Unconsolidated Undrained (UU) test (Fredlund et al. 2012)

The procedure for conducting undrained tests on unsaturated soil specimens is similar to that for saturated soil specimens. Unsaturated soil specimens fail at their original matric suction or water content. The initial matric suction within the soil specimen remains constant before conducting the shear strength test. No consolidation test is allowed since the confining pressure remains under undrained conditions for both pore-water and pore-air phases. The soil specimen is compressed under undrained conditions for both the water and air phases. Typically the undrained test is performed at a strain rate ranging from 0.017 to 0.03 % per second, and there is no need to measure pore-water and pore-air stresses. In cases where the conventional triaxial test is used to perform undrained tests on unsaturated specimens, porous disks are replaced by plastic or metal disks positioned on top and at the base of the soil specimen. During the test, the soil specimens are enclosed within a rubber covering layer. The results of the undrained triaxial test on unsaturated soil specimens can be described as the unsaturated soil shear strength approach.

4.3.3.4 Constant Water Content test (Fredlund *et al.* 2012)

The initial consolidation process is carried out in a manner similar to both the constant-water-content test and the consolidated drained test. Once equilibrium conditions are achieved under the applied stresses (σ_c , u_a and u_w), the soil specimen fails under undrained conditions for the pore-water phase and under drained conditions for pore-air phase. The pore-air stress is kept at the value applied during consolidation. The pore-air stress valve remains open during consolidation and shearing. Additionally, the pore-water stress valve remains closed during shearing to induce undrained pore-water conditions. Pore-water stress is estimated using a transducer positioned near the base plate of the triaxial cell.

4.4 Selection of equipment for shear strength measurement

Sailie and Bucher (1984) investigated the shear strength of expansive soils through the implementation of a consolidated undrained triaxial (CU) test. Calabresi and Scarpelli (1985) documented the influence of swell potential on shear strength using the consolidated undrained triaxial (CU) test. Wong (1998) mentioned the impact of swelling properties on the shear strength of expansive soils using the consolidated drained (CD) test. Zewdie (2004) examined shear parameters of expansive soils by conducting the unconsolidated undrained (UU) test. Fredlund *et al.* (2012) mentioned that the standard triaxial apparatus is also employed for testing unsaturated soil specimens. Lastly, the consolidated undrained triaxial test was selected to measure the soil stress-strain and shear strength characteristics of prepared, compacted soil specimens according to TR 2002/21 CSIR protocol for triaxial testing, which agrees with ASTM D4767-99 but uses different specimen dimensions measuring 300 mm in height and 150 mm in diameter with a diameter/height ratio of 1.2.

PART 2: SOIL SUCTION MEASUREMENT

4.5 Introduction

Soil suction constitutes a crucial stress-state variable of unsaturated soils. The magnitude of soil suction significantly affects shear strength, the volumetric variations, and hydraulic conductivity of unsaturated soil. The evaluation of suction is essential to characterise partially saturated soils. Suction refers to the free energy state of water within the soil material (Edlefsen and Anderson, 1943). Rao and Revanasiddappa (2000) noted that matric suction values for compacted clay soil specimens range between 50 kPa and 8 000 kPa at degrees of saturation ranging from 90 % to 35 %. In a similar study, Fondjo *et al.* (2021) observed that for compacted expansive soils, the matric suction values range from 40 kPa and 7 694 kPa, with total suction values ranging from 55 kPa to 9 926 kPa. Determining soil suction poses challenges in the field and in the laboratory (Delage *et al.* 2008, and Fredlund *et al.* 2012). This section reviews the working principles, identifies the advantages and limitations of various techniques used to directly or indirectly assess soil suction, and selects the appropriate suction measurement technique for the research work.

4.6 Direct suction measurement

Matric suction is measured through the direct assessment of negative pore-water pressure, employing a porous ceramic cup. Determining matric suction requires a separation of the water and air phases using this ceramic cup. The maximum measurable matric suction value is limited by the air entry value of the ceramic cup. The suite of devices for the direct calculation of matric suction discussed in this section includes the suction probe, tensiometers, axis translation, and suction plate.

4.6.1 Conventional tensiometer

The tensiometer technique enables a direct assessment of soil suction. The process involves water retention in a high air entry material contained in the tensiometer. This retained water will exhibit a similar negative pore-water pressure in the soil once equilibrium between the gauge device and soil material is achieved. Because there is no genuine semi-permeable layer for dissolvable salts, the impact of the osmotic component on total suction is not estimated. Thus, the estimated pore-water pressure reflects represents solely the matric suction component. However, due to of water

cavitation within the tensiometer, the ceramic cup with the higher air entry restricts the matric suction range assessment to 90 kPa. A standard tensiometer consists of a high air section and porous ceramic cup connected to a gauge pressure, as described in Figure 4.2. To ensure precision, it is essential to calibrate the tensiometers before utilisation and inspect the ceramic cup for any cracks. Additionally, any air pockets should be removed during setup. The tensiometer's response time is verified by subjecting it to an incremental negative water pressure of up to 90 kPa, induced by evaporation water from the submerged ceramic cup. Within 5 minutes of submersion, the negative water pressure within the tensiometer should equalise with atmospheric pressure. To prevent desaturation due to dissipation from the ceramic cup, it should remain submerged in water prior to installation. The prepared tensiometer can be positioned within a soil specimen within the laboratory or a pre-bored hole in situ. Ensuring optimal contact between the soil material and the ceramic cup is imperative to guarantee fluid continuity between the water in the tensiometer tube and the pore-water in the soil material. Equation 4.1 outlines the description of matric suction.

$$\Psi_m = \Psi_{gauge} + (Z_{gauge} - Z_{cup}) \quad [4.1]$$

The vertical distance from the gauge surface to the cup signifies the negative value to be added to the gauge-estimated suction (Ψ_{gauge}) to derive the matric suction at the cup's depth. This correction accounts for the positive head exerted by the water column within the tensiometer at the depth of the ceramic cup. This vertical elevation difference is applicable only when the soil potential is specified per unit of weight.

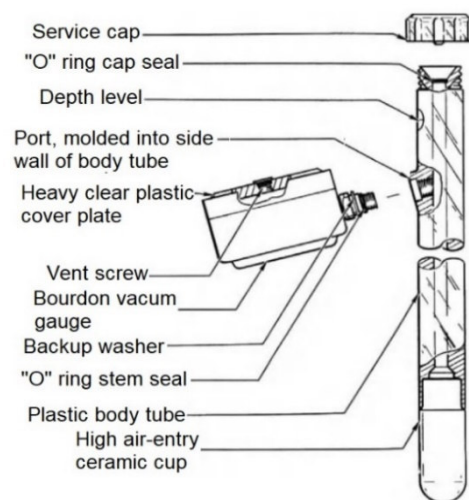


Figure 4.2: Conventional tensiometer (Fredlund et al. 2012)

Advantages of conventional tensiometer:

- Enables rapid measurement of matric suction values in both in laboratory and field settings.
- Easy to use with minimal disturbance to the soil.
- Operates based on a simple, purely physical working principle.
- Interchangeable porous ceramic cups.

Disadvantages of conventional tensiometer:

- Direct measurement of suction encounters challenges due to water cavitation (Fredlund, Rahardjo, and Fredlund, 2012).
- The standard tensiometer has a suction estimation limit of approximately 90 kPa due to air dissemination through the ceramic cup and air displacement as water pressure reduces. The suction limit is not sufficient to evaluate the suction range of compacted clayey soils.
- The air in the sensor induces an inaccurate estimation of pore water pressure when water vaporises as the soil water pressure reaches the vapour pressure at ambient temperature (Pan *et al.* 2010).
- Tensiometer operates optimally only in relatively wet soils and may exhibit slower response times in coarse sand.
- Regular maintenance is required, and it cannot measure total suction.

4.6.2 High suction tensiometer

In contrast to the conventional tensiometer, the high suction tensiometer is an advanced sensor capable of estimating pore-water tension beyond the water cavitation limit of 90 kPa, providing rapid response and enhanced precision. The original concept for the high suction tensiometer was put forth by Ridley and Burland (1993). Improvements were made to the tensiometer measurement technique to enable the direct estimation of a matric suction range of up to 1 500 kPa (Toker *et al.* 2004; Tarantino and Mongiovi, 2001; Toll *et al.* 2013; Mendes and Buzzi, 2014 and Bagheri *et al.* 2018). These tensiometers prevent cavitation by employing a restricted water container volume underneath the ceramic tip, which is additionally pre-pressurised.

4.6.3 Imperial college tensiometer (suction probe)

Ridley and Burland (1993) proposed a suction probe for direct matric suction estimation, shown in Figure 4.3. Originally developed by Imperial College of London,

these miniature tensiometers encompass a soil water potential measurement range of 1 500 kPa. A comparative type of suction probes was developed by Tarantino and Mongiovi (2001) as well as Guan and Fredlund (1997). Later, a smaller probe was designed by Rahardjo and Leong (2006) for the direct assessment of matric suction in the range of 100 kPa to 500 kPa. Meilani *et al.* (2002) introduced a smaller suction probe to evaluate the matric suction and the specimen height during triaxial tests on partially saturated soils. This probe stands out for its ability to directly assess soil suction value up to 1 500 kPa. Additionally, Toker *et al.* (2004) proposed the MIT tensiometer, featuring a width of 38 mm, for use in triaxial tests. The rule for performing suction assessment using the Imperial College tensiometer relies upon the equilibrium between the pore-water pressure in the cylinder and the soil's pore-water pressure. Before equilibrium is reached, water movement occurs either from the water cylinder into the soil or vice versa. The tensiometer is used to estimate pore-water pressure, and matric suction is then deduced based on the applied air tension. The suction probe consists of a high air entry ceramic cup (100 kPa) attached to the tip of the transducer device, the suction probe includes a sensor linked to the opposite end of diaphragm. In the space between the porous plate and the diaphragm lies a de-aired water container, pre-pressurised to a usable high pressure. The diaphragm of the stress transducer responds to the stress applied, and the volume of water reservoir under the ceramic cup decreases. The fundamental issue in suction estimation using the suction probe is the cavitation and air dispersion through the ceramic head. The measurement is conducted in a borehole with a depth of 0 to 5 metres or directly on soil specimens. Equilibrium requires a few minutes, and the suction probe test can be conducted both in the laboratory and the field.

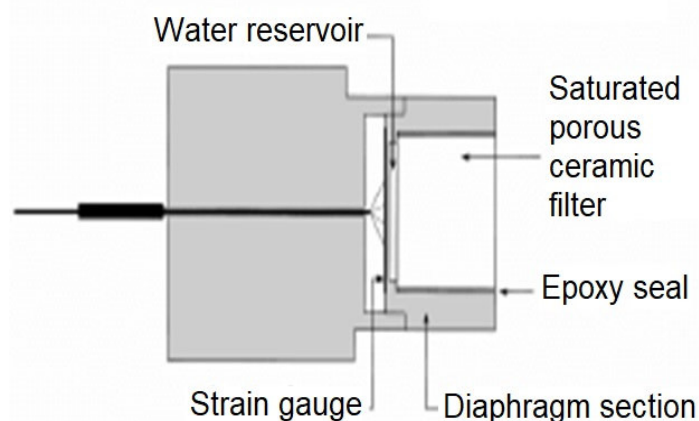


Figure 4.3: Imperial college tensiometer (Ridley and Burland, 1993)

4.6.4 Suction plate technique

The suction plate technique is a direct method employed to estimate the matric suction within the laboratory. The fundamental rule is that the soil specimen absorbs water from the reservoir through a permeable stone (Al-Khafaf and Hanks, 1974; Hamblin, 1981; Chandler *et al.* 1992). At a specific point, the decrease in pressure shown by the indicator is the matric suction. The suction plate apparatus consists of a saturated high air entry disk connected to a water reservoir. As depicted in Figure 4.4, a container is attached to a U-shaped cylinder, which is then linked to a vacuum indicator or manometer. This technique can evaluate matric suction within the range of 20 kPa to 85 kPa. When the matric suction value is higher than 85 kPa, air enters the framework and creates water cavitation. Equilibrium is typically reached in a few hours, depending on the level of suction and hydraulic conductivity of the soil. It is operated by placing a soil specimen on the saturated ceramic disk. As water flows from the reservoir into the specimen, the meniscus alters, and this change is read from the scale. A vacuum is then applied to restore the meniscus to its original position. Various suction values and suction versus moisture content curves for the soil specimen can be constructed by placing soil specimens at different moisture contents on the saturated ceramic plate.

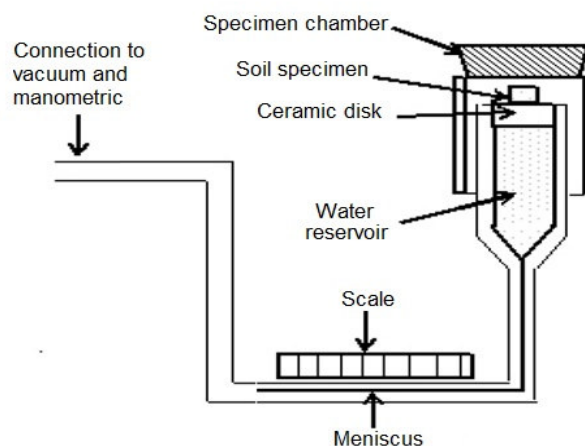


Figure 4.4: Sketch of a suction plate (Fondjo *et al.* 2020)

4.6.5 Pressure plate technique

A pressure plate is a direct method for measuring matric suction in the laboratory. The basic principle involves the controlled removal of soil moisture from soil specimens without disturbing their structure. The pressure plate device, as shown in Figure 4.5, consists of an air supply system, pressure chamber, high air ceramic disk, and air

supply equipment. Beneath the surface of the ceramic plate, two layers of plastic screen are connected to allow water to move between the ceramic disk and the neoprene film (Leong *et al.* 2003). The water reservoir is connected to a pressure transducer located outside the pressure chamber. Additionally, a drainage line is fixed to the water reservoir and vents outside the pressure chamber (indicated in blue in Figure 4.5). The high air ceramic cup separates the water state from the air state, preventing the passage of air through the water. Soil specimens are prepared to match water content and density, and then put in the ceramic disk. A marginal suction value is applied to the chamber so that water in the soil and water in the ceramic are fully connected to allow free movement between ceramic and soil. Subsequently, a consistent low, air pressure is applied to the chamber, compelling some of the pore water to exit through the ceramic and the drainage line. When the drainage has finished, the drain line is closed, pressure is released and the specimens are weighed, yielding an initial wet weight. The specimens are placed back into the chamber, and a higher pressure is applied, expelling more pore water through the ceramic. Equilibrium is reached when no further outflow occurs from the drainage line. At this point, the drain is closed, pressure is released, and further weighing is conducted. The process is repeated until the maximum desired pressure or the ceramic and cell's pressure limit is reached. For each specimen, a SWCC is derived based on wet weight and applied pressure. This method only yields a limited number of points on the SWCC and requires substantial time investment. The highest range of matric suction estimated on the pressure plate is around 1 500 kPa. Equilibrium typically takes a few hours to days. This test is conducted in a laboratory setting for matric suction measurement.

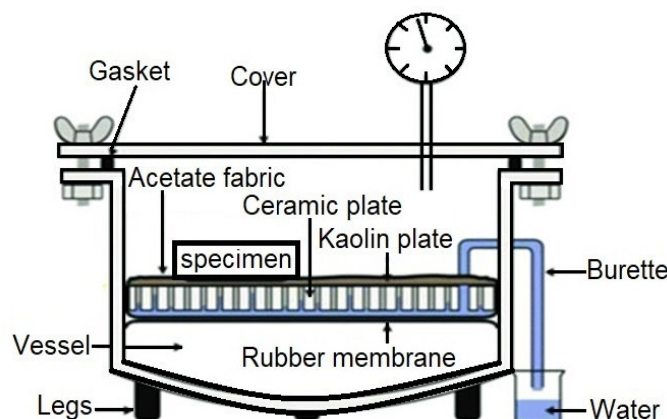


Figure 4.5: Pressure plate apparatus (ASTM C1699-09)

4.6.6 Axis-translation technique (ATT)

The axis-translation technique is a laboratory-based method used for direct matric suction assessment. The suction estimation is within the range of 0 to 1500 kPa, with an equilibrium time spanning from 1 to 16 hours. The working principle of the axis-translation technique consists of translating the reference origin for pore-water pressure (u_w) from its current value to a higher value, equivalent to the pressure of air applied to soil specimen (u_a). The matric suction in the soil specimen remains constant despite the translation of both pore-air and pore-water pressure. Hilf (1956) developed an axis-translation technique to resolve the cavitation issue that arises at low negative pore pressures, as shown in Figure 4.6.

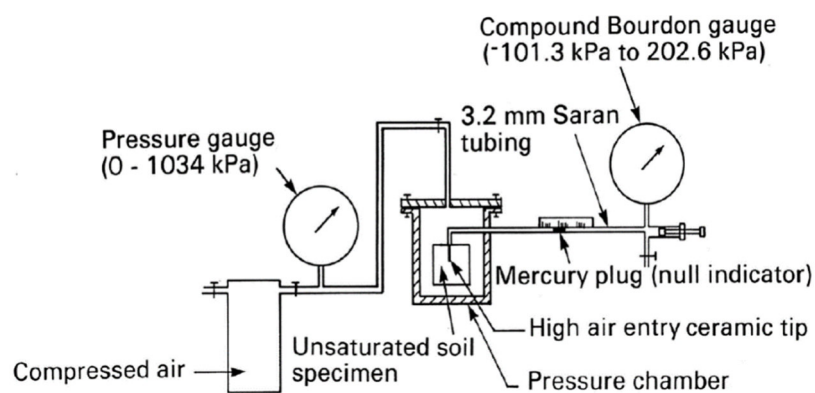


Figure 4.6: Null type, axis translation apparatus (Hilf, 1956)

To estimate the matric suction values through the axis-translation technique, continuous monitoring of both pore-air pressure and pore-water pressure, maintained at atmospheric pressure, is necessary. The axis-translation technique is performed by dividing water and air state in the soil using a saturated high air-entry (HAE) porous material, commonly a ceramic disk. The saturated HAE porous material permits water movement while effectively preventing the passage of air when suction is applied, ensuring that matric suction remains below the air-entry value of the ceramic disk. It is imperative to establish excellent contact between the soil specimen and saturated ceramic disk throughout all tests, guaranteeing alignment between the water state in the tested soil specimen and the pores of ceramic disk (Murray and Sivakumar, 2010). This approach, known as the null-type axis-translation (Fredlund, 1989), maintains the water pressure in the reservoir at zero. An illustration of high air-entry value stones is provided in Figure 4.7.

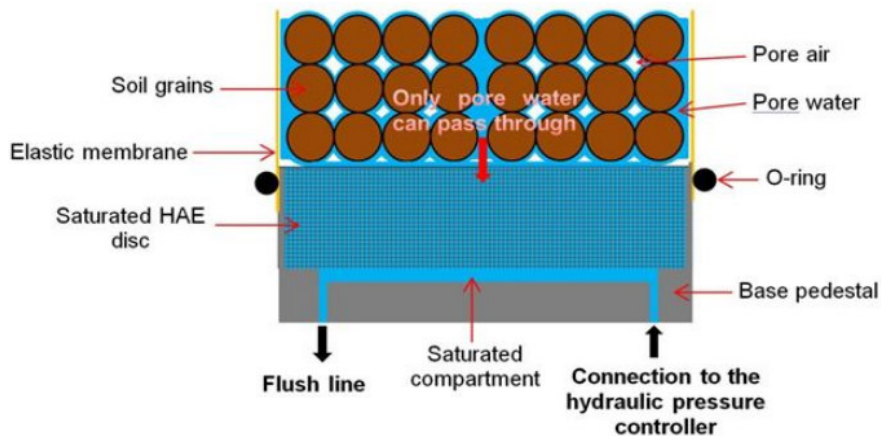


Figure 4.7: High air-entry value stones (VJ Tech, 2020)

Advantages of axis-translation technique

- The fundamental advantage of the axis-translation technique lies in its chemical-free process for regulating suction. Therefore, there is no risk of modifying the pore fluid chemical composition.
- This method is particularly suitable for testing unsaturated soils as it is easy to adjust existing equipment for partially saturated soils testing by including HAE channel and air pressure source.
- The axis-translation technique proves effective for evaluating volume changes and shear strength testing of unsaturated soils, including its application in triaxial cells (Matyas and Radhakrishna, 1968; Wheeler and Sivakumar, 1995) and direct shear boxes (Gan *et al.* 1988).

Disadvantages of axis-translation technique

- The axis-translation technique does not provide rapid results for matric suction estimation. It necessitates continuous water and air phases between the pore-water in the soil material and water in the measuring device to accurately portray the existing suction within the specimen (Romero, 2001).
- Generally, unsaturated soil measurement using the axis-translation technique requires an extended timeframe. Over the course of the test, pore-air diffuses through the water in the HAE disk, leading to the formation of air bubbles under the disk (Fredlund and Rahardjo, 1993). Thus, a flushing system is needed to address these challenges (Bishop and Donald, 1961; Fredlund, 1975).

4.6.7 SWCC by continuous extraction and infusion

An alternative approach to performing the axis-translation method is to control the moisture movement in and out of the soil while measuring the differential pressure (Znidarčić *et al.* 1991). This procedure is very efficient for soils with moderate conductivity, such as fine sands, silts, certain clayey sands (Ray and Morris, 1995). The equipment consists of a small pressure cell (50 mm diameter, 35 mm height) equipped with a high air-entry (HAE) stone at the base (Figure 4.8a). The test begins by saturating the specimen under low (gravity) pressure. Once saturated, the cell is closed, and air pressure (100-200 kPa) is applied to the top. When introduced gradually in stages, this air pressure maintains a zero net pressure between air and water. Once the pressure is attained, the actuator slowly extracts water from the base of the cell with precise flow control at approximately $1 \text{ mm}^3/3 \text{ seconds}$. The extraction rate can be adjusted to suit the test conditions. A slow extraction rate, ensures uniform suction equilibrium throughout the specimen. As the test progresses, matric suction occurs within the sample, lowering the water pressure at the base of the device. Since the air pressure remains constant, the difference ($u_w - u_a$) is interpreted as suction. The test continues to extract water and log pressure difference until a predetermined limit – either the maximum pressure difference (less than the AES blow-through limit) or the minimum moisture level. The test can be reversed, allowing infusion at any time to produce wetting-drying hysteresis (Figure 4.8). The advantage of such a test is the refined control to produce hysteresis and its capability to generate a large number of data points (up to 5 000-10 000, if desired) over a few days of testing.

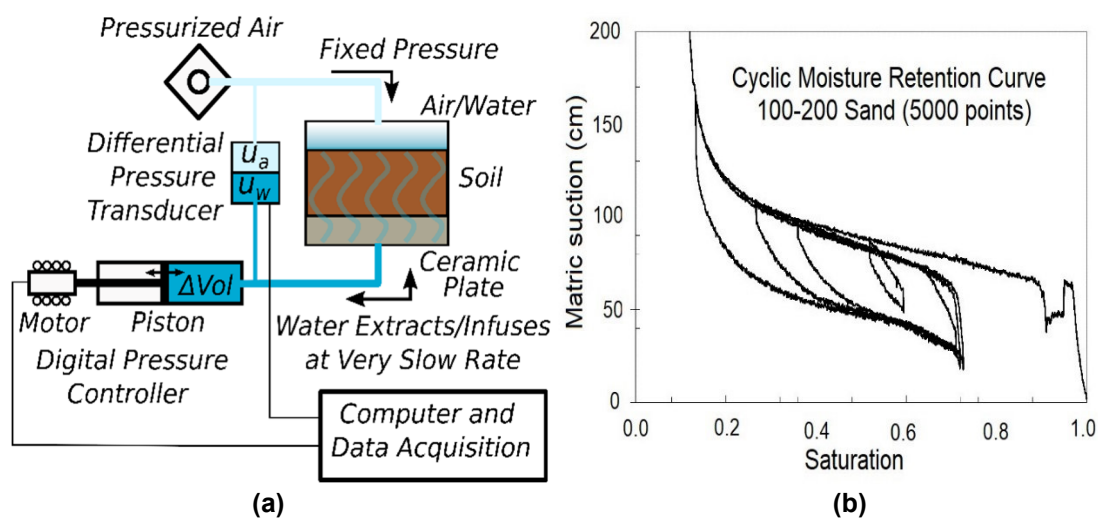


Figure 4.8: Continuous flux SWCC test by axis translation method
(a) test schematic, (b) saturation-suction curve showing hysteresis (Ray and Bene, 2017)

4.7 Indirect suction measurement

Indirect methods for measuring suction include a range of techniques such as the thermal conductivity sensor, electrical conductivity sensor, time-domain reflectometry, squeezing technique, filter paper technique, chilled-mirror hygrometer, and thermocouple psychrometer. These methods rely on achieving equilibrium between the negative pore pressure and the porous sensor embedded within the soil. This section outlines the standards and the fundamental qualities of these procedures.

4.7.1 Electrical conductivity sensor

Soil suction and soil resistivity depend on the rate of soil saturation, serving as parameters for evaluating changes in water content. The electrical conductivity sensor estimates the electrical resistance of a porous block that remains in contact with the soil specimen used for suction assessment. Figure 4.9 shows a conventional illustration of an electrical conductivity sensor. The electrical resistance of the porous block decreases as the water content increases. The device consists of two concentric electrical electrodes embedded in a gypsum-based porous block. A calibration graph establishes a relationship between water content, the electrical resistance of the block, and soil suction. Figure 4.10 shows that the change of electrical conductivity (σ') with matric suction relies upon the soil's nature.

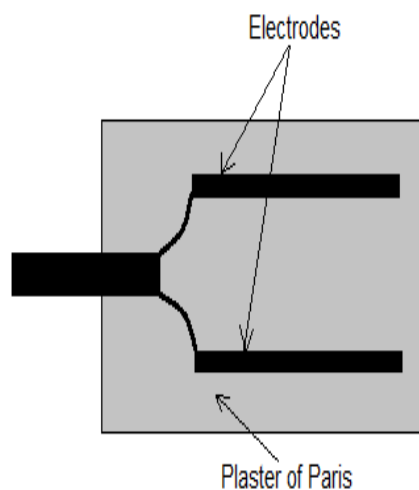


Figure 4.9: Typical electrical conductivity sensor (Fondjo et al. 2020)

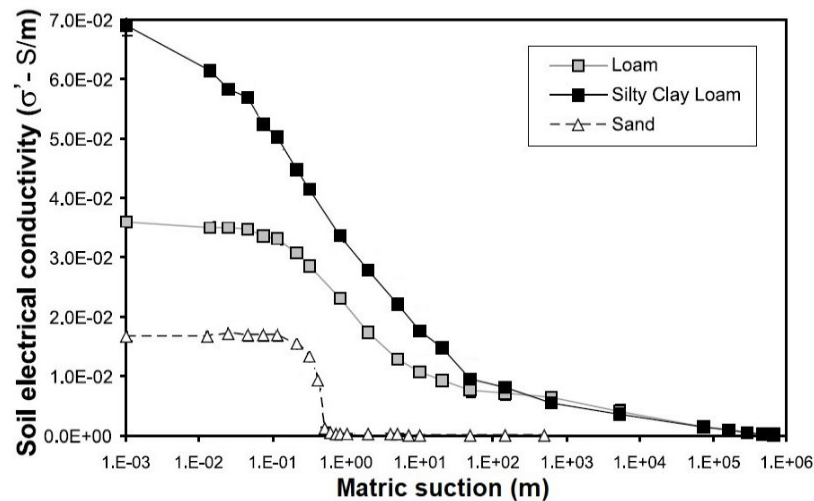


Figure 4.10: Variation of electrical conductivity with matric suction (Doussan and Ruy, 2009)

Advantages of electrical conductivity sensor

- Effective to estimate the suction in non-saline soil materials, with a suction value range typically spanning 30 kPa to 1 500 kPa.
- Convenient to handle, enabling field and laboratory soil suction assessments.

Disadvantages of electrical conductivity sensor

- Reduced sensitivity when the matric suction value is more than 300 kPa; potential dissolution of the block in water, leading to increased salt content in the soil material; and hysteresis during drying and wetting cycles.
- Impact of salt concentration in soils on block electrical resistance, potentially causing readings to deviate from precise water content values.
- Equilibrium time for the porous block is approximately 2 to 3 weeks, with variations depending on suction values – around six hours for a matric suction of 50 kPa and up to two days for a matric suction value of 1 500 kPa.

4.7.2 Thermal conductivity sensor

The thermal conductivity probe evaluates the negative pore pressure in the soil directly. It operates on the fundamental principle that a gradient of matric suction between the porous block and the soil material creates water movement until their suction values reach equilibrium. The water content inside the porous block depends upon the matric suction exerted by the surrounding soil. The equilibrium time depends on factors such as the hydraulic conductivity of the porous medium, properties of the

surrounding soil, and the temperature gradient. Shaw and Baver (1939) developed an apparatus formed with a temperature probe and heater to assess the thermal conductivity in soil materials. Richards (1955) designed an electrothermal device to estimate the dampness in a porous medium. This device consists of a resistance thermometer covered with a tinier heating coil. Richards suggested the use a sandy silt material as a porous block. Bloodworth and Page (1957) studied the materials suitable for a permeable cup in a thermal conductivity sensor, with their results endorsing capstone as an appropriate option. Phene *et al.* (1971) manufactured a thermal conductivity sensor utilising a Germanium P-N diode as a temperature probe. This probe is covered with a 40-gauge Teflon-coated copper wire serving as a heating coil. The thermal conductivity probe consists of a porous ceramic block containing a temperature detection element and the tinier heater. The thermal conductivity of a porous ceramic block varies in line with the dampness content within it. Figure 4.11 describes a cross-section of a standard thermal conductivity sensor.

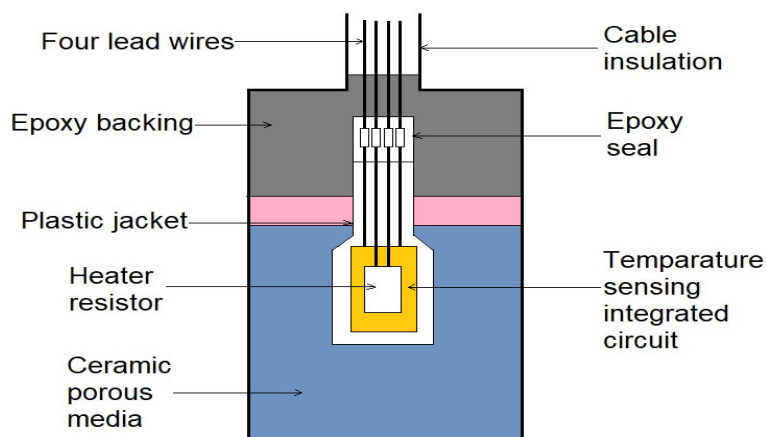


Figure 4.11: Cross section of thermal conductivity suction sensor (Phene *et al.* 1971)

The calibrated sensor estimates the matric suction using a sensor placed within the soil material, achieving equilibrium with the pore water stress state. The block should be large enough to contain the heat pulse without impedance from the surrounding soil, and the sensor's pore size distribution plays a significant role. Estimating thermal conductivity at equilibrium relies on the matric suction in the soil material, obtained from calibration curves. The thermal estimation conductivity process consists of assessing the heat diffusion within the porous block. Determining water content within the porous block entails heating the permeable block through a centrally inserted heater measuring the resulting temperature increase. Fredlund and Wong (1989) mentioned that a proficient calibration technique is necessary for practical design

applications. Due to the non-uniqueness of characteristics in the porous ceramic block, calibration becomes crucial for precise and reliable estimation. Calibration involves initial readings with sensors immersed in water, followed by further calibration readings after the sensors has dried. Additional calibration points are determined for each specific suction value.

Advantages of thermal conductivity sensor

- Reliable estimation of soil moisture deficiency across a wide range of suction values; measurement is not affected by the quantity of salt in water (Fredlund and Wong, 1989).
- Capacity to use a system of data loggers for remote and continuous monitoring.
- Applicable for suction measurement in both laboratory and field settings.

Disadvantages of thermal conductivity sensor

- Inherent variability in the consistency of porous stones between different sensors poses a challenge.
- An alternate calibration curve is applied for each thermal conductivity sensor before suction assessment.
- The thermal conductivity sensor shows hysteresis during drying and wetting cycles.
- The maximum matric suction range is 1 500 kPa; precision diminishes within the higher range.

4.7.3 Filter paper technique

The filter paper technique is an indirect method to estimate the negative pore pressure within the soil. This method is straightforward and efficient for gauging soil matric suction. The filter paper technique can be traced back to Hansen (1926) in Denmark, while Gardner (1937), Fawcett and Collis-George (1967), McQueen and Miller (1968), Al-Khafar and Hanks (1974) have contributed significantly to its development. Efforts have been made to utilise the filter paper technique in engineering practice (Gallen, 1985; McKeen, 1985; Chandler and Gutierrez, 1986). The filter paper technique was utilised, for instance, to estimate the profile of swelling potential in expansive soils and pavement subgrade (McKeen, 1985). In recent years, the filter paper technique has been used regularly to assess soil moisture deficiency, particularly following the U.S

protocol. The filter paper technique has been available and has gone through restoration with the appearance of low-cost precise digital balance (Abedi-Koupai and Mehdizadeh, 2008). The filter paper technique depends on the water absorption properties of filter paper. When the filter paper is in contact with the soil material, it will either desorb or retain dampness until the suction value inside the soil material and filter paper are equal. The filter paper accomplishes the equilibrium with the soil through vapour (total suction evaluation) or fluid flow (matric suction evaluation). The filter paper serves as a moisture sensor and offers versatility in soil suction estimation because of its benefits over other suction estimation techniques. Figure 4.12 shows a standard setup for both non-contact and in-contact filter paper techniques, for total and matric suction estimation. Filter paper technique ranges from a few kilopascals to several hundred thousand kilopascals (Fawcett and Collis-George, 1967; McQueen and Miller, 1968). However, meticulous execution is necessary to ensure reliable results. The non-contact filter paper technique evaluates total suction, while the contact version gauges matric suction.

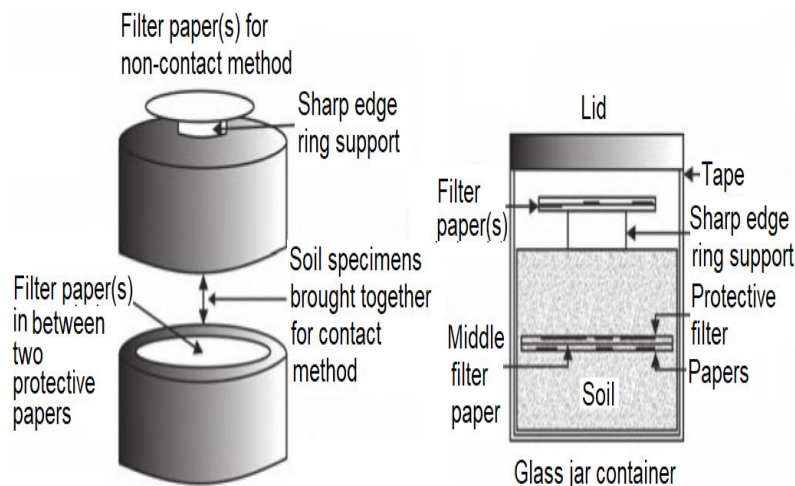


Figure 4.12: Non-contact and in-contact FPT for total and matric suction measurements (Fondjo et al. 2020)

4.7.3.1 In-contact filter paper technique

The filter paper technique depends on the water absorption properties of the filter paper. Soil suction is derived from the moisture content inside the filter paper, utilising an appropriately calibrated curve. Generally, two types of filter papers are used for suction assessment: Schleicher and Schuell No. 589, WH and Whatman No. 42. The in-contact filter paper technique provides direct estimation of matric suction. A filter

paper with a known mass is placed in direct contact with the soil specimen, sealed in a glass jar with a lid and electrical tape, and then placed in a temperature-controlled apparatus. Wet soil specimens take longer to attain equilibrium, approximately a week. Equilibrium is typically reached within 4 weeks with a 1 % error margin (Swarbrick, 1995). However, several researchers have used different equilibrium periods of filter paper with the suction of soil specimen. A period of at least seven days is necessary, with five days being a minimum requirement according to Chandler and Guitierrez, 1986. ASTM 5298-16 suggested an equilibrium period of one week. In addition, several filter paper measurements conducted by Ling and Toll (2000) show that equilibrium is achieved by approximately 97 % within a week. The estimation of filter paper water content takes place after the equilibrium state.

Table 4.1: Equilibration time for filter paper method (Leong *et al.* 2002)

References	Equilibration Time	Filter Paper Method
Fawcett and Collis-Georges (1967)	6 to~7 days	Contact
McQueen and Miller (1968)	7 days	Contact
Al-Khafaf and Hanks (1974)	2 days	Contact and uncertain contact
Hamblin (1981)	Minutes-36 days	Contact
Chandler and Gutierrez (1986)	5 days	Contact
Duran (1986)	7 days	Non-contact
Greacen <i>et al.</i> (1987)	7 days	Contact
Sibley and Williams (1990)	3 days	Contact
	10 days	Non-contact
Ridley and wray (1996)	14 days	Contact and non-contact
Houston <i>et al.</i> (1994)	7 days	Contact and non-contact
Harrison and Blight (1998)	7 to~10 days	Wetting and non-contact
	21 days	Drying and non-contact
	10 days	Wetting and contact
	25 to~30 days	Drying and contact

The accuracy of estimation depends on the type of filter paper, the calibration curves, and the sensitivity of the electronic scale. The calibration curves are obtained by equilibrating the filter paper on a suction plate at various applied suction values and water content in the filter paper. Additionally, the filter paper calibration curve is established using a salt solution as an osmotic-potential source for suction. Researchers such as Hamblim (1981), Chandler and Gutierrez (1986), Greacen *et al.* (1987), and Chandler *et al.* (1992) have contributed to this field, other researchers like McQueen and Miller (1968), Al-Khafaf and Hanks (1974), McKeen (1980), and Greacen *et al.* (1987) have conducted calibrations of Schleicher and Schuell No. 589 filter paper for both contact and non-contact filter paper situations.

Table 4.2: Calibration curves for Whatman No.42, Schleicher and Schuell No. 589 filter papers

Whatman No.42 Filter Paper		
References	Calibration curves $\psi =$ suction (kPa)	Filter paper water content w_f (%)
Hamblim (1981)	$\log \psi = 8.022 - 3.683 \log w_f$	-
Chandler and Gutierrez (1986)	$\log \psi = 4.84 - 0.0622 \log w_f$	$w_f < 47$
Chandler <i>et al.</i> (1992)	$\log \psi = 6.05 - 2.48 \log w_f$	$w_f \geq 47$
Greacen <i>et al.</i> (1987)	$\log \psi = 5.327 - 0.0779 \log w_f$	$w_f < 45.3$
ASTM E832-81	$\log \psi = 2.413 - 0.0135 \log w_f$	$w_f \geq 45.3$
Schleicher and Schuell No. 589 Filter Paper		
McQueen and Miller (1968)	$\log \psi = 5.238 - 0.0723 \log w_f$	$w_f < 54$
	$\log \psi = 1.8966 - 0.01025 \log w_f$	$w_f \geq 54$
Al-Khafaf and Hanks (1974)	$\log \psi = 4.136 - 0.0337 \log w_f$	$w_f < 85$
	$\log \psi = 2.0021 - 0.009 \log w_f$	$w_f \geq 85$
McKeen (1980)	$\log \psi = 4.9 - 0.0624 \log w_f$	$w_f < 66$
	$\log \psi = 1.25 - 0.0069 \log w_f$	$w_f \geq 66$
Greacen <i>et al.</i> (1987)	$\log \psi = 5.056 - 0.0688 \log w_f$	$w_f < 54$
ASTM E832-81	$\log \psi = 1.882 - 0.0102 \log w_f$	$w_f \geq 54$

4.7.3.2 Non-contact filter paper

The non-contact filter paper method estimates the total negative pore pressure. In this process, the vapour phase isolates the filter paper and the soil, acting as a barrier that prevents the transfer of solutes. A dry filter paper is placed on a soil specimen within a sealed glass container, allowing water vapour equilibrium to occur between the soil specimen and the filter paper at a constant temperature. The vapour space over the soil specimen functions as an authentic semi-permeable layer for water vapour but not for ions in pore water. Consequently, only the measurement of total suction is possible. After achieving equilibrium, the moisture content of the filter paper is rapidly evaluated. Prior to estimating the total suction, calibration curves depicting the relationship between suction and moisture content in the filter paper are either developed or adopted. Subsequently, the suction values within the soil specimen are estimated using the equations derived from the calibration curves. Fredlund (1989) has noted that the non-contact filter paper method is applicable for estimating total suction both in the field and in the laboratory. Chandler and Gutierrez (1986), and Hamblin (1981) have reported that the filter paper technique is an inexpensive, simple, and reliable method. Leong *et al.* (2002) have identified several factors influencing the estimation of matric suction using the filter paper technique, including the nature of the filter paper, hysteresis, the suction source, and the equilibrium period.

Advantages of filter paper technique

- Simplicity, low cost, effectiveness, relatively high precision, and no requirement for specific equipment.
- Suction value estimation covers a wide range, and measurements can be conducted in both laboratory and field settings.

Disadvantages of filter paper technique

- The test process demands extreme care, and the use of appropriate calibration curves is crucial.
- The precision of suction measurement values relies on the accuracy of calibration curves (Lu and Likos, 2006).
- The method is time-consuming, and several weeks are necessary for the filter paper to achieve an equilibrium state.

4.7.4 Time domain reflectometry (TDR)

Time domain reflectometry (TDR) is an application that indirectly measures matric suction both in laboratory and field settings. Initially, TDR was developed to estimate soil volumetric water content by Topp *et al.* (1980). Subsequently, others have utilised this technique, Kalinski and Kelly (1993), Benson and Bosscher (1999), Amente *et al.* (2000), and Yu and Drnevich (2004). The time domain reflectometry approach involves assessing the apparent dielectric constant of the material. The dielectric constant is connected to the water content within the soil material (Topp *et al.* 1980). The bulk pore-water, retained in the pores between clay clusters, induces capillary phenomena (i.e. matric pull) without a true semi-porous layer. As a result, the time domain reflectometry method primarily quantifies matric suction rather than total suction. Time domain reflectometry procedure requires embedding at least two metal rods into the ground to function as a waveguide or parallel transmission line. An electromagnetic signal is transmitted through the soil specimen along the conductor at the waveguide's starting point. Upon reaching the end of the conductors, the signal is reflected back to the transmitter. The oscilloscope records the return time and wavelength. The electromagnetic wave velocity in the soil is denoted as (V) and is provided by Equation 4.2.

$$V = \frac{1}{\sqrt{\epsilon\mu}} \frac{C_0}{\mu} \quad [4.2]$$

Where:

V = electromagnetic wave velocity in the soil [$\text{m} \cdot \text{s}^{-1}$]

ϵ = dielectric number relative to a vacuum ($8.85 \cdot 10^{-12}$) [$\text{F} \cdot \text{m}^{-1}$]

μ = magnetic permeability relative to a vacuum [$\text{H} \cdot \text{m}^{-1}$]

C_0 = velocity of light in vacuum ($3.0 \cdot 10^8$) [$\text{m} \cdot \text{s}^{-1}$]

The relative magnetic permeability of soils is generally unity, as it was rarely contain significant amounts of ferromagnetic minerals. The dielectric constant of the soil material decreases, denoted as $\epsilon = \epsilon_s$ in Equation 4.3.

$$\epsilon_s = \left[\frac{C_0^2}{V^2 \mu} \right] = \left[\frac{t^2}{\mu} \right] \times \frac{C_0^2}{4l^2} \quad [4.3]$$

Where:

l = length of the conductors or wave guides [m]

t = the return time [s]

V = velocity $c = 2l/t$ [m.s⁻¹]

The assessment involves duplicating the time by water at a comparable temperature to that of the soil material. If L represents the travel time within the soil material and l_w is the travel time within water, Equation 4.4 describes the dielectric constant of water (ϵ_w).

$$\epsilon_w = \epsilon_s \left[\frac{l_w^2}{l^2} \right] \quad [4.4]$$

The calibration of the ratio (l_w^2/L^2) depends on either the volumetric (θ) or gravimetric water content (ω). It is important to note that the relationship between volumetric water content (θ) and gravimetric water content (ω) is described in Equation 4.5. The relationship involves the specific gravity (G), the void ratio (e), and the degree of saturation (S).

$$\theta = \frac{\omega G_s}{1 + \omega G_s} = \frac{eS}{1 + eS} \quad [4.5]$$

Figure 4.13 shows the fundamental components of a time domain reflectometry water content estimating system, including water content and SWCC. This system can be calibrated to determine matric suction (Leong, *et al.* 2002).

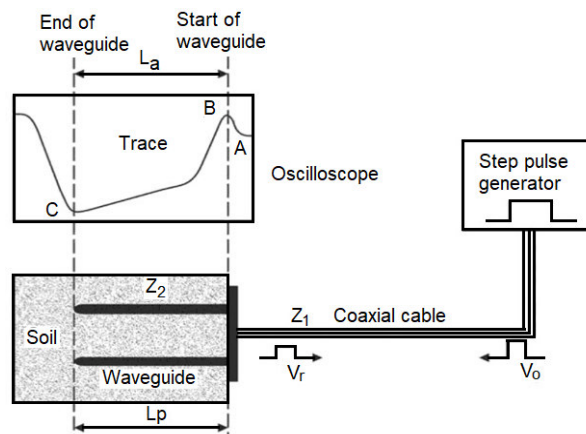


Figure 4.13: Four main components of a typical time domain reflectometry system (modified from Jones *et al.* 2001)

Figure 4.14 displays a calibration curve between volumetric water content (θ) and the ratio (I_w^2/L^2) derived from in situ measurement on several soils with varying particle size compositions. These compositions range from 54 % clay, clay 41 % silt, and 5 % sand to 9 % clay, 10 % silt, and 81 % sand, and 2 % clay and 98 % organic matter (Roth *et al.* 1990).

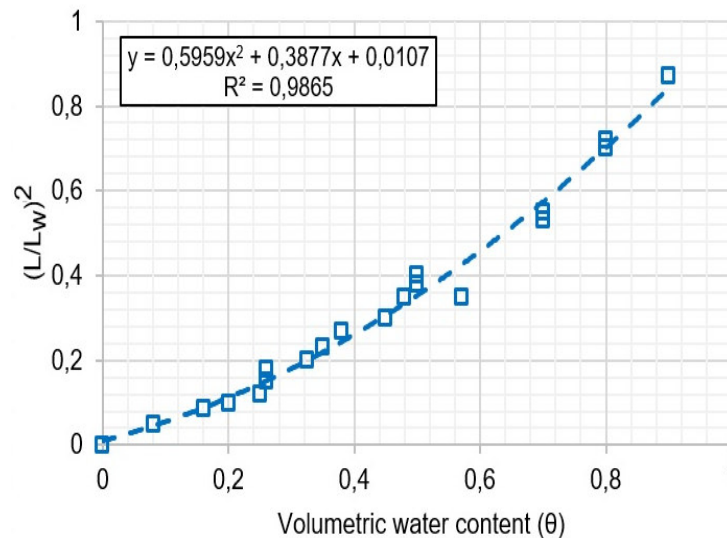


Figure 4.14: Time domain reflectometry (Roth *et al.* 1990)

Figure 4.15 shows the calibration of the apparent dielectric constant ($K_a = \epsilon_s/\epsilon_w$) against the matric suction. However, the length of insertion of the waveguide, between $0.4L$ and L , has a negligible effect on the calibration (Leong *et al.* 2002).

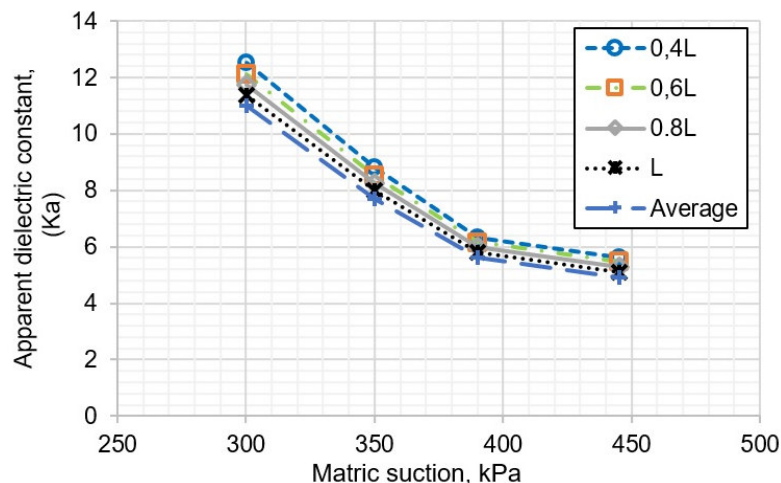


Figure 4.15: Relationship of apparent dielectric constant versus suction at different insertion lengths (Roth *et al.* 1990)

Advantages of the TDR technique

- One of the benefits of this method is its ability to provide reliable estimations of volumetric water content over a short duration (Benson and Bosscher, 1999).
- The TDR technique exhibits a narrow response zone perpendicular to the

waveguides. This enables accurate estimation of volumetric moisture content with high degrees of vertical resolution. Continuous investigation of soil moisture can also be achieved through automation and multiplexing.

- TDR is efficient in assessing volumetric water content with a precision of 1-2 %.

Disadvantages of the TDR technique

- The TDR technique requires sophisticated equipment. The precision of the TDR approach for matric suction evaluation depends on the accuracy of the SWCC of the tested soil specimen.
- The wave induced by the input signal can produce errors between actual and measured water content, reaching up to 65 % at low water contents ($\theta = 0.1$ and lower), decreasing to 15 % at high water content ($\theta = 0.4$ and above).
- There is a high likelihood of signal attenuation in saline soil materials, and it is not recommended for waterlogged soils or soils with high organic content.

4.7.5 Squeezing technique

Researchers have focused their studies towards understanding the relationship between water content and matric suction. However, the role of osmotic suction in the hydro-mechanical characteristics of clayey soil materials may also be significant (Pan *et al.* 2010). Osmotic suction is influenced by the concentration of ions dissolved in water-filled pores. The squeezing approach involves directly evaluating osmotic suction within the soil material. Osmotic suction values obtained through the squeezing method typically range from 0 to 1 500 kPa. Equilibrium times of several days are generally appropriate. Figure 4.16 illustrates a standard pore liquid squeezer. The pore fluid extraction process employs a pore-fluid squeezing technique. This involves squeezing a specimen to remove larger macropores of water and then assessing its electrical conductivity. The process is linked to the total concentration of dissolved salts, which is also indicative of soil osmotic suction. The pore fluid squeezer approach covers the entire range of osmotic suction and provides reliable suction values (Brackley, 1971; Fredlund and Morgenstern, 1977). The method isolates pore fluid from soil specimens placed between a squeezing cylinder and a set of disks at one side and a disk, filter paper, and cylindrical base on the other side. A syringe is used to extract pore liquid from the cylindrical base. When analysing the conductivity curve in relation to osmotic suction, the electrical pulse transforms into osmotic suction

(Fredlund and Morgenstern, 1977). Krahn and Fredlund (1972) conducted an experimental study on the squeezing technique, revealing that extraction energy affects the process. Osmotic suction is calculated as the difference between the total suction values obtained through a psychrometer minus the matric suction obtained from the pressure plate. The pore liquid squeezer method exhibits a reliable estimation of osmotic suction (Krahn and Fredlund, 1972; Wan et al. 1995).

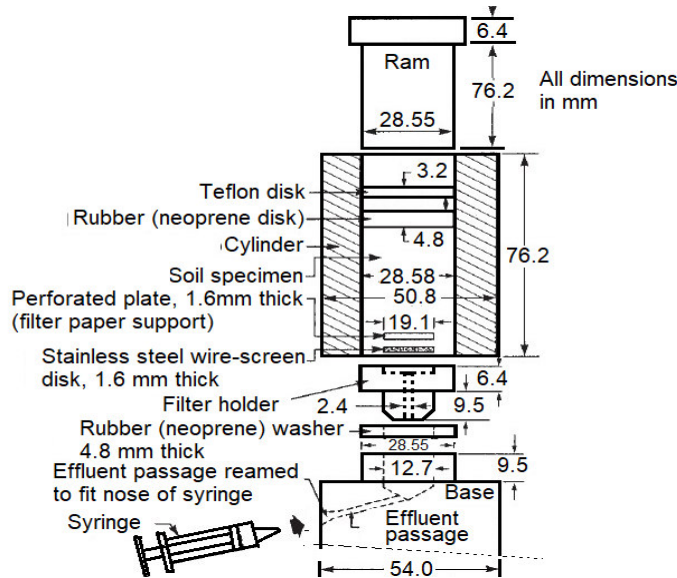


Figure 4.16: Design of pore fluid squeezer (Manheim, 1966)

4.7.6 Thermocouple psychrometer technique

The thermocouple psychrometer is a method that deduces the water potential of the fluid phase within a soil specimen by estimating it within the vapour phase in equilibrium with the soil specimen. Total soil suction is estimated using the relative humidity of the air phase in voids. In theory, the correlation between the relative humidity of the vapour phase and water potential of the liquid phase is described by the Kelvin relation (Fredlund and Rahardjo, 1993), as given in Equation 4.6.

$$\Psi_t = \frac{RT}{V_w} \ln \left(\frac{P}{P_0} \right) \quad [4.6]$$

Where:

Ψ_t = total suction [kPa]

R = universal gas constant (8.314×10^{-6}) [MJmol⁻¹K⁻¹]

T = temperature [K]

$V_w =$ Molar volume of water $(1.8 \times 10^{-5})[\text{m}^3 \text{mol}^{-1}]$

$P =$ actual vapour pressure of air in
equilibrium with the liquid phase [kPa]

$P_0 =$ saturation vapour pressure [kPa]

The Peltier psychrometer measures total suction within the range of 100 kPa to 8000 kPa, both in the field and laboratory. The Peltier type, introduced by Spanner (1951), and the wet loop-type, as developed by Richards and Ogata (1958), are the two essential thermocouple psychrometers. Peltier-type psychrometers and wet-loop ones differ in how the evaporating junction is wetted to initiate dissipation. Figure 4.17 displays a psychrometer sensor. The thermocouple comprises 0.025 mm diameter wires of constantan and chrome. On the other end, the wires are attached to 26 American Wire Gauge (AWG) copper lead wires to form a reference junction. The conductive copper wires with greater width function as heat sinks, maintaining a constant temperature at the reference junction. These heat sinks absorb the heat generated near the reference junction as the estimating junction device cools. The increased cooling produced by the chrome-constantan thermocouple is approximately 0.6 °C below the ambient temperature (Brown and Bartos, 1982). This heightened cooling signifies the minimum relative humidity or the uppermost limit of total suction, and it can be estimated using the thermocouple psychrometer. The estimation of total suction involves placing a psychrometer close to the soil specimen in an enclosed environment. The evaluation of the relative humidity occurs when an equilibrium state is reached. Prior to taking the psychrometer's reading, the isothermal equilibrium between the soil and air is mandatory. A precision of ± 0.001 °C in the temperature regulator is necessary to estimate total suction with an accuracy of ± 10 kPa (Krahn and Fredlund, 1972). The thermal equilibrium in the psychrometer occurs by setting the microvoltmeter reading to zero.

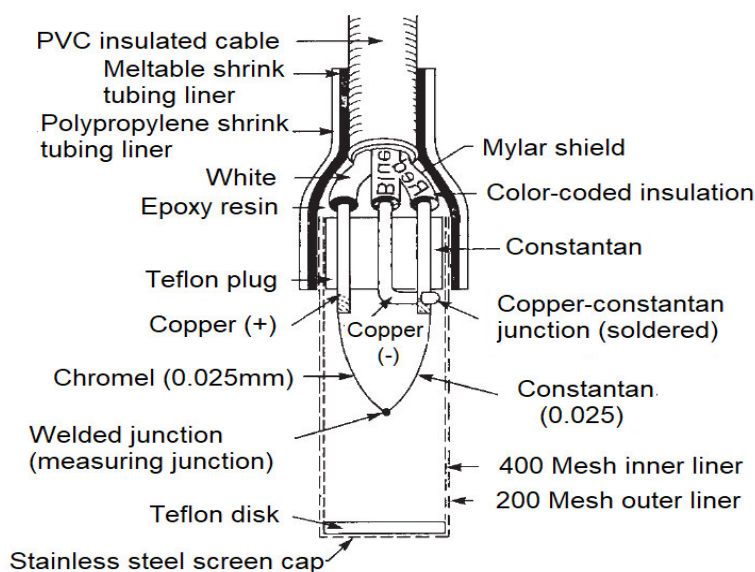


Figure 4.17: Peltier type thermocouple psychrometer (Brown and Collins, 1980)

The calibration of the psychrometer involves establishing a correlation between microvolt outputs of the thermocouple and an acceptable total suction value using a salt solution. Filter paper is saturated with NaCl or KCl and positioned at the chamber base. The osmotic suction for NaCl and KCl solutions at various modalities and temperatures is determined using the graphs shown in Figures 4.18 and 4.19.

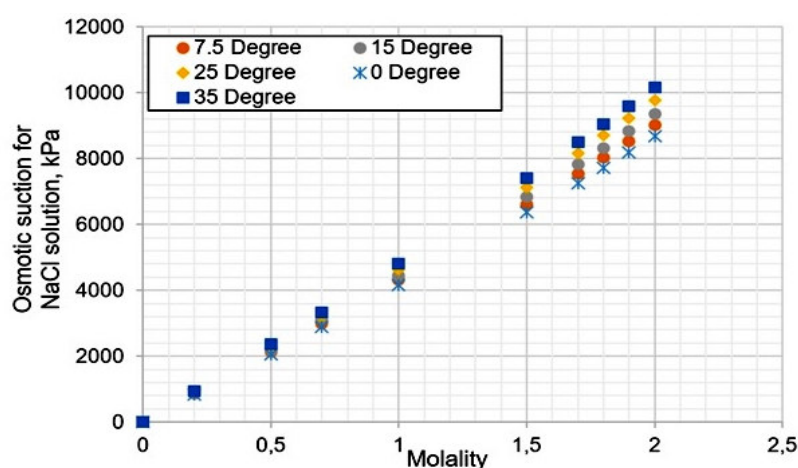


Figure 4.18: Calibration of Peltier-type psychrometers with NaCl solution (Lange, 1967)

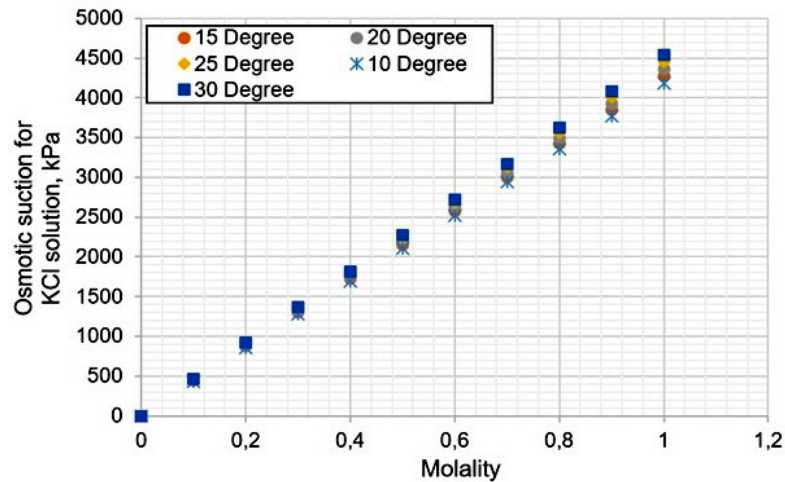


Figure 4.19: Calibration of Peltier-type psychrometer with KCl Solution (Campbell and Gardner, 1971)

The utilisation of thermocouple psychrometers in the field involves the following steps. The soil temperature near the psychrometer should be estimated using another calibrated thermocouple. This ensures that the calibration of the psychrometer remains valid at the actual soil temperature. Figure 4.20 illustrates the calibration lines of thermocouple psychrometer obtained from measurements and compared with suction values of NaCl solution concentration (Blight, 1992).

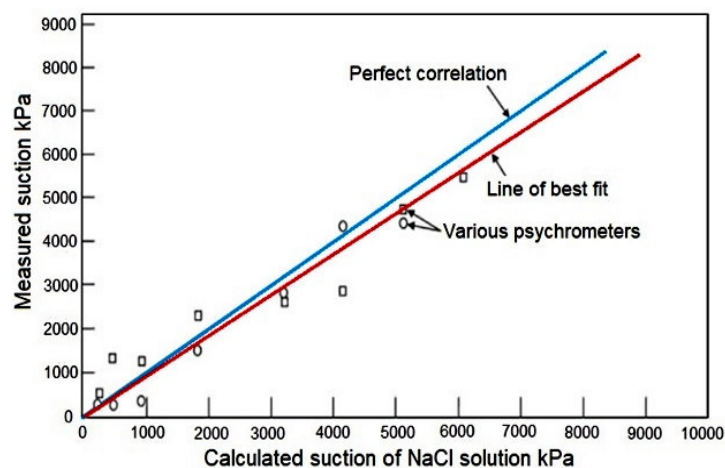


Figure 4.20: Calibration of psychrometers vs sodium chloride solution (Blight, 1992)

Advantages of thermocouple psychrometer

- A thermocouple psychrometer is a precise and reliable technique for estimating total suction, given proper procedures and precautions.
- The technique covers wide range of suction estimations relevant to engineering practice and is especially suitable for drier soils.

- Since the estimation occurs in the vapour phase, a constant fluid phase is unnecessary, and only a minute amount of water is required.
- The technique is flexible, and commercially available devices can be used for field and laboratory tests.
- Estimations can be conducted relatively quickly, often with the aid of an automated data acquisition device.
- Laboratory estimations performed with a sample chamber require smaller soil specimens, which is advantageous for continuous testing.

Disadvantages of thermocouple psychrometer

- Adequate time must be allocated to ensure total vapour-pressure equilibration, and steps are necessary to identify and prevent errors arising from temperature gradients.
- When working with small specimens, precautionary measures are necessary to avoid errors caused by evaporative loss during specimen handling.
- The operational lifespan of these instruments is generally short. In-situ sensors typically last around 2 to 3 years.
- Calibration dependency, susceptibility to environmental errors due to rapid temperature changes, vulnerability to corrosion in acidic environments, the need for sensitive measuring equipment, and the complexity of application are challenges associated with this technique.

4.7.7 Chilled mirror hygrometer

The chilled mirror hygrometer is an indirect method used to measure the total suction of soil. In geotechnical engineering applications, this technique involves measuring the negative pore pressure of soil (Leong *et al.* 2003; Albrecht *et al.* 2003; and Schanz *et al.* 2004). The chilled mirror hygrometer uses the chilled mirror dew point technique to evaluate total suction under isothermal conditions within a sealed container. The estimation of total suction using the chilled mirror hygrometer relies on achieving equilibrium between the fluid phase of water in a soil specimen and the water vapour phase in the airspace over the soil specimen, all within an airtight chamber (Bulut and Leong, 2008). The chilled mirror hygrometer apparatus is similar to the thermocouple psychrometer, and its total suction estimation range spans from 150 kPa to 30 000 kPa. The equilibration process takes approximately 10 minutes. The airtight container

includes components like a fan, optical sensor, infrared sensor, thermocouple, mirror, and soil specimen. Figure 4.21 shows a schematic drawing of a chilled mirror hygrometer.

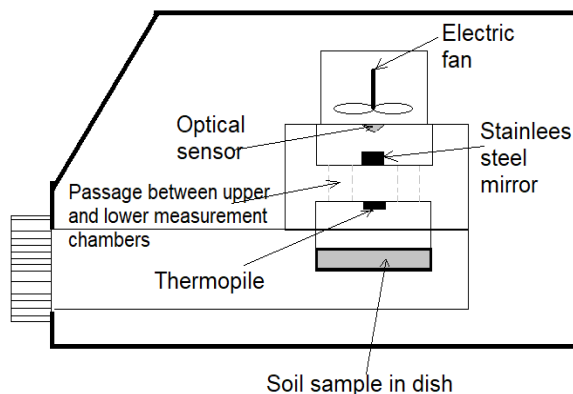


Figure 4.21: Chilled-mirror hygrometer (Bulut and Leong, 2008)

The chilled mirror hygrometer measures both the dew point and temperature of the headspace above the specimen. The specimen is enclosed within an appropriate sealed chamber to prevent excessive drying. Water vapour from the soil specimen is allowed to condense on a mirror surface. A photoelectric cell detects the point at which condensation first becomes visible in the mirror. The temperature of the soil specimen aligns with the vapour space temperature and is estimated using an infrared thermocouple. The relative humidity or water movement of the specimen comes from the estimated dew point and temperature. A small fan circulates air within the sensing chamber, promoting rapid vapour equilibrium. To ensure temperature equilibrium, both the soil specimens and equipment are kept in the same environment for at least several hours before testing. The chilled mirror hygrometer provides an essential characterisation of moisture in terms of the temperature at which vapour condenses. Precise temperature control is vital, and the difference between the dew point and specimen temperatures must be kept low. The calibration curve process involves employing standard salt solutions with known concentrations against their osmotic suctions. Figure 4.22 illustrates the calibration curve developed by Leong *et al.* (2003).

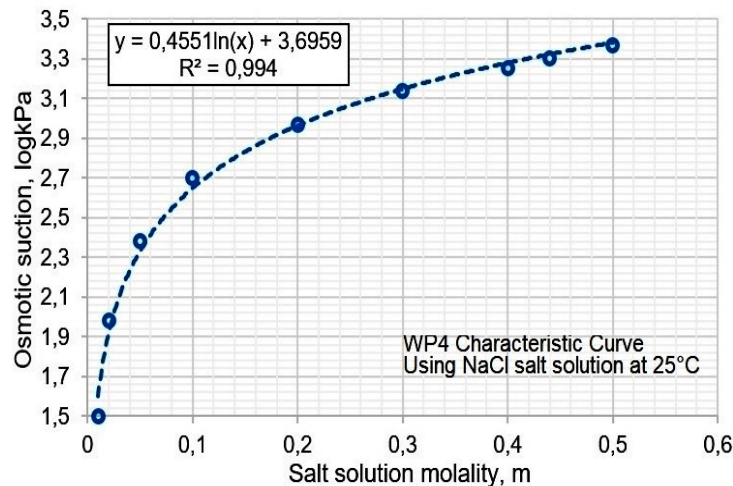


Figure 4.22: Calibration of WP4 chilled mirror hygrometer (Leong *et al.* 2003)

Leong *et al.* (2003) conducted an investigation into the accuracy of a chilled mirror dew point technique for compacted soil specimens. The calibration of the instrument involved utilising standard solutions of various salts. Equilibration times during both total suction estimation and calibration is less than 15 minutes. For compacted specimens, the total suction values were compared with the combined matric and osmotic suction of similar soils, estimated independently. Matric suction estimation relied on the axis-translation device, while the osmotic suction evaluation depends on electrical conductivity estimations of the soil water solution derived from a pore water squeezer device. The test outcomes indicated that the total suction measured using the chilled mirror dew point device is usually higher than the sum of matric and osmotic suction obtained. Nevertheless, taking into account the discrepancies of total suction assessment and the process of maintaining isothermal equilibrium between the specimen and vapour space, the chilled mirror hygrometer technique emerged as the most accurate approach for estimating total suction. Another hygrometer employed for total suction measurement is the polymer capacitance sensor. This sensor comprises two electrodes separated by a thermoset polymer film that releases or absorbs moisture based on the relative changes in the surrounding air (Albrecht *et al.* 2003).

4.7.8. Relative humidity sensor (Transistor psychrometer)

The relative humidity sensor evaluates the total negative pore pressure in the soil indirectly. The enhancement is due to the calibration procedure and innovation in micro-chip applied science (Woodburn *et al.* 1993). The performance enhancement allows the instrument to estimate a wide range of negative pore pressures ranging

from 100 kPa to 10 000 kPa. The range and precision in estimation are also influenced by the sensitivity of the transistor to changes in temperature. Woodburn *et al.* (1993) designed two types of thermally protected containers for the transistor probes: 12 and 8-probe units. The 8-probes psychrometer equipment includes an insulated cover for better temperature control. Each probe device can measure total suction within 1 hour. With the 12 and 8-probe units, total negative pore pressure estimation for twelve and eight specimens can be achieved in 1 hour. Bulut *et al.* (2000) reported that the relative humidity sensor demonstrates a superior ability to estimate total suction at lower moisture content compared to other psychrometric techniques. It allows measurement of both the relative humidity and temperature of the vapour space of the soil. Fredlund and Rahardjo (1993) mentioned that total suction estimation involves the application of Kelvin's law. The relative humidity sensor comprises a thermally insulated reservoir that holds the psychrometer probes and a data logger for recording the outputs. The instrument operates similarly to the thermistor psychrometer (Woodburn *et al.* 1993). The relative humidity sensor functions as a wet and dry bulb thermometer, wherein a wet/dry transistor probe is used instead of wet/dry thermometer bulbs as in thermistor psychrometers. This sensor deduces the relative humidity of the air space in equilibrium with a soil specimen. Figure 4.23 illustrates a schematic representation of wet and dry transistors psychrometer probe. The potential difference output of the probe is used to deduce the total negative pore pressure.

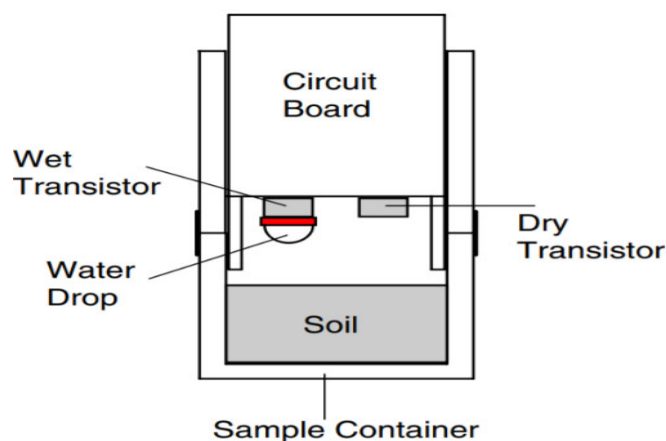


Figure 4.23: Schematic drawing of a transistor psychrometer probe (Woodburn *et al.* 1993)

Figure 4.24 describes a conventional calibration curve for a transistor psychrometer probe. The calibration curve is affected by the following factors: temperature variations, depth of the water drop, and hysteresis. The equilibrium process for transistor probes involves subjecting the semiconductor tests to at least 4 hours of

zero total soil potential over distilled water. The results are then adjusted to the initial zero reading before the calibration process for soil suction evaluation. Woodburn *et al.* (1993) mentioned that the data logger records the potential voltage difference after an equilibration period of 1 hour for the soil suction value. The insulated container housing the suction probes maintains a consistent temperature during the test. Precise outcomes are achieved in a chamber where the temperature fluctuates within approximately ± 0.5 °C.

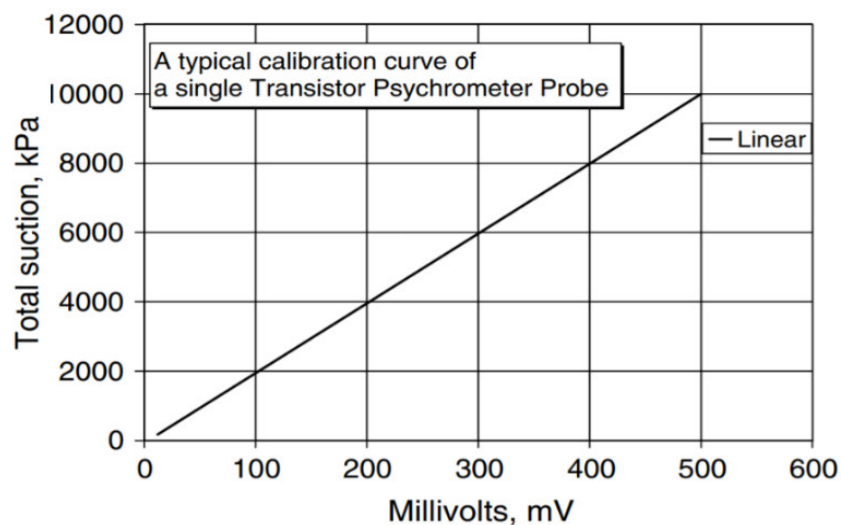


Figure 4.24: A typical calibration curve (Woodburn *et al.* 1993)

4.8 Summary of suction measurement methods

The methods commonly used to directly or indirectly measure soil suction are summarised in Table 4.3, categorised by method, suction range, equilibrium time, and application (Lab/Field).

Table 4.3: Summary of suction measurement methods

		Method / Technique	Suction Range (kPa)	Equilibrium time	Laboratory (L) or field application (F)
Direct method	Matric suction	Conventional tensiometer	0 - 90	Minutes	L & F
		Suction plate	20 to ~ 85	Hours	L
		High suction tensiometer : Imperial college tensiometer or Suction probe	0 – 1 500	Minutes	L & F
		Pressure plate	0 – 1 500	Hours-day	L
		Null-type axis-translation	0 – 1 500	1 to ~ 16 hours	L
Indirect Method	Matric suction	Electrical conductivity sensor	50 to ~ 1 500	6 to ~ 50 hours	L & F
		Thermal conductivity sensor	10 to ~ 1 500	Hours-day	L & F
		In-contact filter paper	Entire range	7 to ~ 14 days	L & F
		Time Domain Reflectometry (TDR)	0 – 1 500	Hours	L & F
	Osmotic suction	Squeezing technique	0 – 1 500	days	L
	Total suction	Thermocouple psychrometer	100 to ~ 8 000	1 Hours	L & F
		Relative Humidity Sensor (Transistor Psychrometer)	100 to ~ 10 000	Hours-day	L
		Chilled-mirror hygrometer	150 to ~ 30 000	10 minutes	L
		Non-contact filter paper	Entire range	7 to ~ 14 days	L & F

4.9 Selection of equipment for suction measurement

In summary, soil suction measurement involves both direct and indirect methods. The direct suction measurement apparatus includes conventional tensiometers, high suction tensiometers, suction plates, imperial college tensiometers (suction probes), pressure plates, and null axis translation. The indirect measurement method is divided into three suction groups: total suction, osmotic suction, and matric suction. The total suction measurement apparatus includes the thermocouple psychrometer, transistor psychrometer, chilled-mirror hygrometer, and non-contact filter paper. Osmotic suction estimation includes the squeezing technique, while matric suction estimation equipment utilises electrical conductivity sensors, thermal conductivity sensors, time-domain reflectometry, and in-contact filter papers. The review of various equipment used to estimate both indirectly and directly the matric suction in soil specimens corroborates a maximum suction value of 1 500 kPa. Thermocouple psychrometer and relative humidity sensors can estimate a maximum total suction value of 8 000 kPa. Chilled mirror hygrometer can assess a maximum total suction value of 30 000 kPa. Lastly, the chilled-mirror hygrometer and filter paper technique are cost-effective and efficient methods. However, the filter paper technique is likely the most cost-effective and simple procedure to estimate total suction and matric suction across the entire suction range, demanding careful attention to the test process. In this research work, the soil suction evaluation employs the filter paper technique according to ASTM D5298-16.

CHAPTER 5: EXPERIMENTAL RESEARCH METHODS

5.1 Overview

This chapter outlines the sampling technique, testing programmes and geotechnical modelling process. The laboratory tests carried out on soil samples and geotechnical modelling helped to achieve the research objectives. These laboratory tests were performed at the Geotechnical and Materials Laboratory within the Department of Civil Engineering at the Central University of Technology (CUT), as well as at the Advanced Material Testing Laboratories located at the Centre for Scientific Innovation and Research (CSIR) in Pretoria. The testing programme was employed to investigate the physical properties and hydro-mechanical characteristics of soil samples. The tests conducted included:

1. Gathering samples, field moisture: Standard practice for preserving and transporting soil samples (ASTM D4220/D4220-M14); Moisture content determination (SANS 3001-GR20)
2. Particle size: Sieve analysis (ATSM D6913-04); Sedimentation analysis (ASTM D7928-16)
3. Index and Plasticity: Specific gravity (ASTM D854-14); Consistency limits test (ASTM D4318-05)
4. Shrink-swell: Linear shrinkage (IS 2720 Part 20, 1992); Free swell index (FSI)(IS 2720 Part 40, 2000); Free swell ratio (FSR) (Sridharan and Prakash, 2000); Zero swell test (ZST); Swelling pressure test (ASTM D4546-14)
5. Moisture-density: Standard and Modified Proctor compaction tests (SANS 3001-GR30)
6. Mineralogy: X-ray diffraction test (Brindley and Brown, 1984)
7. Unsaturated pore pressures: Soil suction by filter paper technique (ASTM D5298-16); Soil-water characteristic curve modelling (SWCC) (Van Genuchten, 1980; Kosugi, 1996; Fredlund and Xing, 1994; Durner, 1994 and Seki, 2007)
8. Soil stress-strain, strength: Triaxial Consolidated-Undrained with pore pressure measurement; Hybrid Axis-translation/Triaxial Undrained tests (TR 2002/21 CSIR).

The laboratory data related to the research work's objectives were recorded and analysed. Software packages were utilised streamline and interpret the laboratory data, including:

1. General Laboratory Data Acquisition and Test Control: To acquire performance data over time and log it digitally for subsequent processing.
2. SWRC Fit: Process the SWCC data and obtain empirical moisture-suction fitting parameters.
3. NCSS 12: Design geotechnical predictive models (GPM) based on laboratory results.

5.2 Materials

The rocks of the Beaufort and Ecca groups underlie the Free State province in South Africa, forming a part of the Karoo Sequence. These 250-million-year-old siltstone and mudstone layers disintegrate near the surface, creating a deeper profile of expansive soil. Generally, these soil materials originate from either argillaceous sedimentary rocks or igneous rocks. For this research work, soil samples were collected from four locations across the Free State province. Workers excavated the pits manually using shovels and pickaxes and retrieved samples within the active zone depth. Figure 5.1 depicts the site locations along with their global positioning system (GPS) coordinates.

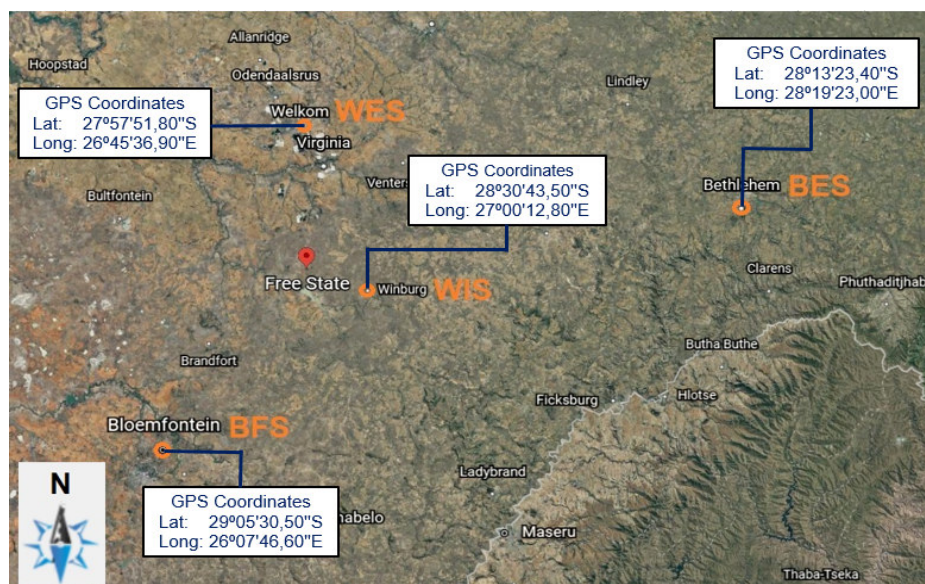


Figure 5.1: GPS coordinates of sampling sites

Sampling locations reflected different gradations and consistency limit values, encompassing a wide range of swelling characteristics. The soil samples are labelled as follows: Bloemfontein (BFS), Welkom (WES), Winburg (WIS), and Bethlehem (BES) soils. Field crews adhered to standard sampling practices to preserve and transport soil samples. Workers utilised sealed bags to maintain the field water content. Subsequently, the samples were transported to the Geotechnical and Materials Laboratory at the Department of Civil Engineering, Central University of Technology (CUT), and the Advanced Material Testing Laboratories at CSIR. For the initial soil classification, approximately 35 kg of each soil sample was oven-dried at 105 °C for preliminary geotechnical tests, including particle size distribution (PSD) test, consistency limit test, specific gravity test, XRD test, and modified compaction test. Additionally, oven-drying another 35 g at 105 °C allowed the determination of the natural water content for each soil sample. Figure 5.2 presents a photograph captured during sampling activities.

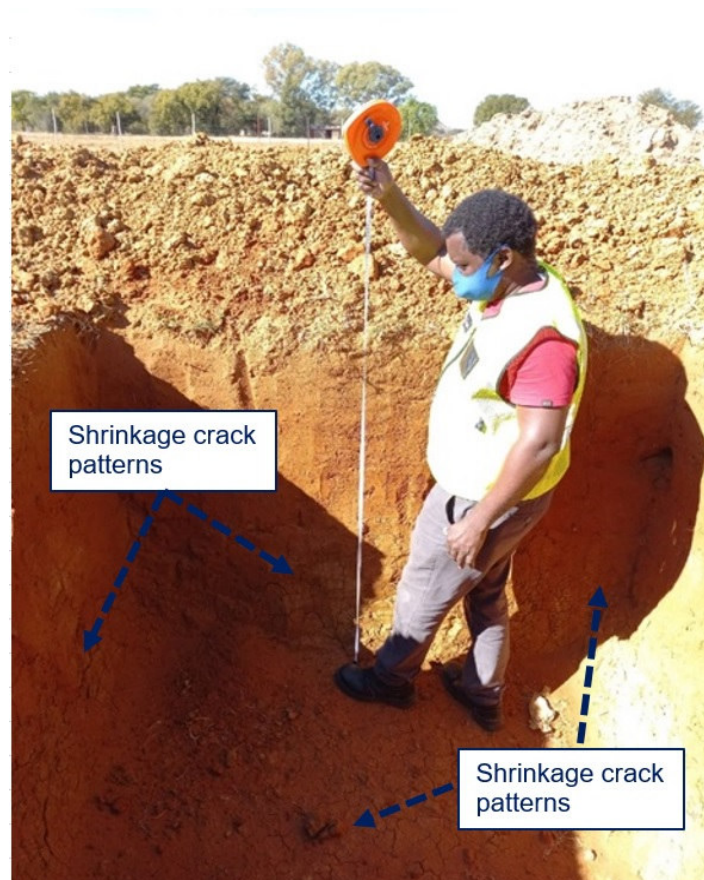


Figure 5.2: Typical borrow pit

5.3 Testing programmes

The established procedures and standards mentioned above were employed to evaluate the physical, mineralogical, and hydro-mechanical properties of the soil in this research. The following sections will detail the specific test procedures.

5.4 Specific gravity test

The specific gravity of a material is a ratio of the mass of a unit volume of the material to the mass density of free-gas distilled water at a specified temperature. Soil specimens were oven-dried at 105 °C for 16 to 24 hours. The test began by weighing an empty pycnometer, adding approximately 50 g of the oven-dried soil, and then weighing it again. Around 400-450 ml of tap water was added to the soil and allowed to soak for about 12 hours. Heating the water for 10 minutes helped to eliminate any air bubbles trapped within the soil, with agitation aiding in the air removal process. Finally, the pycnometer was carefully filled to the 500 ml level mark. The weight of the pycnometer, water, and soil was then measured. Equation 5.1 calculates the specific gravity of the soil within the pycnometer, with an additional term correcting the measurements for any temperature deviations.

$$Gs = \frac{M_2 - M_1}{[(M_4 - M_1) - (M_3 - M_2)]} \quad [5.1]$$

Where:

G_s = specific gravity

M_1 = empty mass of volumetric flask [g]

M_2 = mass of pycnometer + oven dried soil [g]

M_3 = mass of pycnometer + oven dried soil + filled water [g]

M_4 = mass of pycnometer + filled water only [g]

5.5 X-ray diffraction test

The X-ray diffraction (XRD) test identifies and quantifies the mineralogical composition of the soil. XRD utilises the crystallographic structure of lattices in stacked clay mineral sheets or other components such as quartz and feldspar. When these lattices are approached at the correct angle, diffracted X-rays create constructive interference, which is sensed by a goniometer. Different clay minerals have specific "signature"

angles that produce a stronger signal to the sensor. Quartz, feldspar, and clay minerals exhibit unique angles that can be deduced from the sensor output (Brindley and Brown, 1984). The diffractometer analysed clays from BFS, WES, WIS, and BES soil samples. The peak positions and relative intensities in each diffractogram matched reference standards, revealing the significant minerals present in the soils (Sachan and Penumadu, 2007). To minimise fabric orientation, the samples were ground to a fine powder, producing dispersed (flat, parallel) mineral sheets when suspended in water and allowed to dry on the glass base plate. Sample holders were front-loaded using a larger well holder, as shown in Figure 5.3.



Figure 5.3: Samples preparation by front loading for XRD test

The sample holder was maintained at a 45° angle with the horizontal to ensure proper alignment, preventing the specimen inside from breaking into small, disoriented pieces. Figure 5.4 shows the locations of the primary components of the diffractometer. Diffrac Plus XRD was used to control the device and record output to a raw data file. The samples were labelled BFS, WES, WIS, and BES.

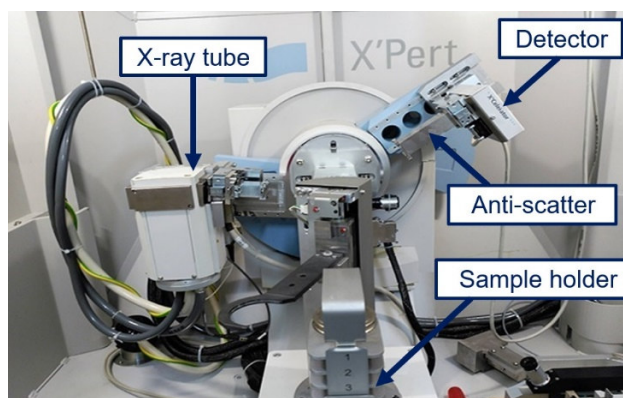


Figure 5.4: X-ray diffractometer setup

5.6 Free swell index test

The free swell index (FSI) reflects the potential for soil to expand when no confining force acts from above. Oven-dried soil samples passing through a 425 µm sieve served as the primary soil ingredient in compliance with the Standard Method IS 2720 Part 40. The test begins with 10 g of oven-dried soil poured into a 100 ml graduated cylinder, followed by kerosene. In a second cylinder, 10 g of oven-dried soil is placed and then filled with distilled water up to 100 ml. Shaking and stirring the cylinder allows trapped air bubbles to escape to the surface. The cylinders are left to stand for 48 hours to reach equilibrium, allowing the soil to expand freely within the cylinder. The final volumes of soil in kerosene and water are used for comparison. The free-swell index is calculated by dividing the difference in volumes by the soil volume in kerosene and expressing it as a percentage (Equation 5.2). Table 5.1 lists the qualitative degree of expansion as compared to FSI. Examples of the test can be seen in Figure 5.5.

Table 5.1: Swelling potential classification (IS 2720 Part 40, 2000)

FSI (%)	Degree of expansion	Degree of severity
< 50	Low	Non critical
50-100	Medium	Marginal
100-200	High	Critical
> 200	Very high	Severe

$$FSI = \frac{V_d - V_K}{V_K} \times 100 \quad [5.2]$$

Where:

FSI = free swell index [%]

V_d = volume of soil specimen in water [ml]

V_k = volume of soil specimen in kerosene [ml]



Figure 5.5: Free swelling index test setup

5.7 Free swell ratio test

The free swell ratio (FSR) is the ratio of the equilibrium sediment volume of 10 g oven-dried soil passing through a 425 μm sieve in distilled water versus kerosene, as described in Equation 5.3. Sridharan and Prakash (2000) proposed a method for performing the free swell ratio test. This test provides a realistic prediction through a simple methodology that examines the clay mineralogy of swelling soil. Table 5.2 provides information on the swelling potential and soil mineralogy.

$$FSR = \frac{V_d}{V_k} \quad [5.3]$$

Where:

FSR = free swell ratio

V_d = volume of soil specimen in water [ml]

V_k = volume of soil specimen in kerosene [ml]

Table 5.2: Swelling potential classification (Sridharan and Prakash, 2000)

FSR	Clay type	Soil expansivity	Dominant clay mineral type
=1	Non-swelling	Negligible	Kaolinite
1.0-1.5	Mixture of swelling and non-swelling	Low	Mixture of Kaolinitic and Montmorillonitic
1.5-2.0	Swelling	Moderate	Montmorillonitic
2.0-4.0	Swelling	High	Montmorillonitic
> 4.0	Swelling	Very High	Montmorillonitic

5.8 Modified Proctor compaction test

Compaction is the process of densifying and rearranging soil grains by eliminating air voids using mechanical equipment, such as a compaction machine. Compacting soil with carefully controlled moisture content improves soil shear strength, stiffness, slope stability, and bearing capacity. Engineers utilise soil compaction to enhance the support of buildings, roads, and other engineering structures. Laboratory compaction tests provide essential parameters for field applications. Applying constant energy to a specific soil specimen at its optimum water content (OMC) results in the maximum dry unit weight (γ_{dmax}). OMC represents the best conditions for the soil to contract without inducing plastic flow. OMC and γ_{dmax} are fundamental parameters used in analysing of the behaviour of compacted soil.

5.8.1 Compaction test procedure

The mould assembly consisted of a detachable collar and a base plate with a thick spacer. The next step involved filling the mould with water and placing a glass lid over it. Weighing the entire assembly and then subtracting the weight of the mould and glass lid yielded an accurate measurement of water weight and mould volume. Temperature adjustment factors could slightly influence the results, as illustrated in Figure 5.6. Equation 5.4 shows the final formula for calculating the mould volume. Soil samples that passed through a 4750-micron sieve were used as the primary soil source. Approximately 35 kg of each sample, oven-dried at 105 °C for 16 to 24 hours, was divided into five identical portions for compaction. Each portion was mixed with a slightly varying quantity of moisture, thoroughly integrated into its matrix. The compaction process involved five layers for each portion. Each layer received an equal number of tamps distributed evenly over its surface. After tamping, the depth to the tamped surface indicated the provisional specimen density. The same treatment was applied to four more layers. Following compaction, a moisture content specimen was extracted from the material to determine the final moisture content of the portion. Weighing precision was maintained to the nearest 0.1 gram, and the drying temperature was consistently set at 105 °C. The other sample portions, prepared in the same manner but with different initial moisture contents, produced slightly different OMC and γ_{dmax} values. Appendix 1, Table 5.3, displays a data sheet for the modified Proctor compaction test.

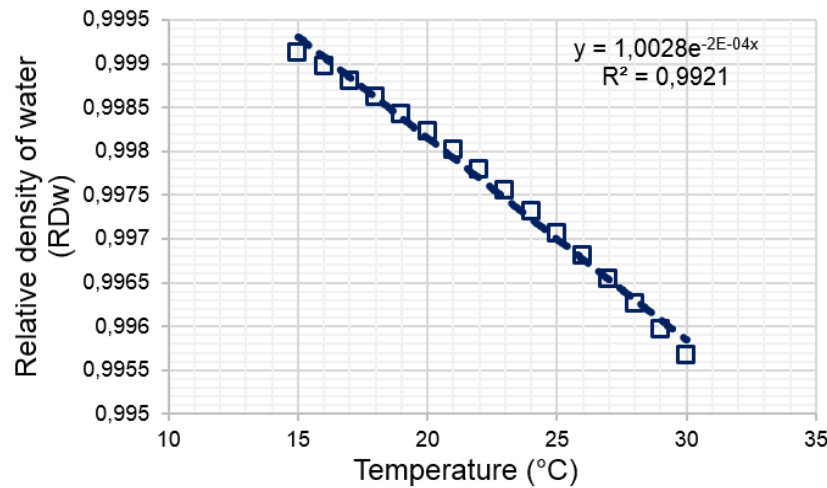


Figure 5.6: Relative density of water versus temperature

$$V = \frac{\text{Mass of water}}{RDW} \quad [5.4]$$

Where:

V_m = volume of the mould [ml]

RDW = relative density of water

5.8.2 Sample preparation for other tests

Each sample block (BFS, WES, WIS, and BES) yielded compacted specimens with varying moisture contents and dry densities. Dividing the compacted specimens into sub-specimens supplied the zero swelling, suction, and consolidated undrained test parameters. The laboratory estimated the most consistent results by averaging two outcomes for each tested specimen. Appendix 3, Figure 5.7, presents a diagram of the specimen and sub-specimen setup for zero swelling, suction, and triaxial tests.

5.8.3 Calculation of compaction test features

The final moisture content of the compacted soil was calculated as the average of all sub-specimens, rounded to the nearest 0.1 % (Equation 5.5).

$$W = \frac{M_1 - M_2}{M_2 - M_c} \times 100 \quad [5.5]$$

Where:

M_1 = mass of container + wet soil [g]

M_2 = mass of container + dry soil [g]

$$M_1 - M_2 = \text{mass of water [g]}$$

$$M_2 - M_c = \text{mass of the oven dried soil [g]}$$

$$W = \text{moisture content [\%]}$$

The bulk moist unit weight (γ) of the compacted soil sample was determined by dividing the wet mass by the volume of the mould used for each point corresponding to a specific moisture content, as described in Equation 5.6.

$$\gamma = \frac{M_t - M_m}{V} \quad [5.6]$$

Where:

$$M_t = \text{mass of mold, base plate, and wet soil [kg]}$$

$$M_m = \text{mass of mold and base plate [kg]}$$

$$V = \text{volume of the mold [m}^3\text{]}$$

$$\gamma = \text{bulk dry unit weight [kg. m}^{-3}\text{]}$$

Equation 5.7 provides the dry unit weight of the material at each moisture content.

$$\gamma_d = \frac{\gamma}{1 + \frac{W}{100}} \quad [5.7]$$

Where:

$$\gamma = \text{bulk dry unit weight [kg. m}^{-3}\text{]}$$

$$\gamma_d = \text{dry unit weight [kg. m}^{-3}\text{]}$$

$$W = \text{moisture content [\%]}$$

5.8.4 Estimation of compaction features using a mathematical function

Fondjo and Theron (2021) proposed an empirical curve-fitting approach to estimate the location of OMC and (γ_d) when test results do not precisely align with that point. Microsoft Excel spreadsheet was used to record the compaction test results. The compaction curve relies on two primary parameters: the dry unit weight (γ_d) and moisture content (W). Consequently, in the mathematical concept, (γ_d) can be expressed as a function of (W), $\gamma_d = F(W)$. The OMC represents the moisture

content at which the dry density is at its maximum, indicating that the slope of $dF(W)/dW$ should be zero at that point. Therefore, a helpful approach would be to estimate the relationship between (W) and (γ_d) through an empirical function. Various equations were tested to replicate the curve with different levels of success, including linear, logarithmic, inverse, quadratic, cubic, power, compound growth, and exponential. The cubic equation provided the best fit with R-squared value ($R^2 = 1$). Therefore, the mathematical function $\gamma_d = F(w)$ takes the form of a third-order equation (Equation 5.8) with constant real number values (a_1, a_2, a_3, a_4).

$$\gamma_d = a_1 W^3 + a_2 W^2 + a_3 W + a_4 \quad [5.8]$$

$$a_1, a_2, a_3, a_4 \in \mathbb{R}$$

Equation 5.8 of the compaction curve is differentiated with respect to the moisture content (W):

$$\frac{d(\gamma_d)}{dW} = \frac{d}{dW} (a_1 W^3 + a_2 W^2 + a_3 W + a_4) \quad [5.9]$$

$$\frac{d(\gamma_d)}{dW} = \frac{d(a_1 W^3)}{dW} + \frac{d(a_2 W^2)}{dW} + \frac{d(a_3 W)}{dW} + \frac{d(a_4)}{dW} \quad [5.10]$$

$$\frac{d(\gamma_d)}{dW} = a_1 \frac{d(W^3)}{dW} + a_2 \frac{d(W^2)}{dW} + a_3 \frac{dW}{dW} \quad [5.11]$$

$$\frac{d(\gamma_d)}{dW} = 3a_1 W^2 + 2a_2 W + a_3 \quad [5.12]$$

Solving Equation 5.14 below finds the function's local maxima and optimum moisture content (OMC).

$$\frac{d(\gamma_d)}{dW} = 0 \quad [5.13]$$

$$3a_1 OMC^2 + 2a_2 OMC + a_3 = 0 \quad [5.14]$$

The final OMC value satisfied Equation 5.14 and occupies the correct location on the compaction curve. Additionally, Equation 5.15 for the (γ_{dmax}) was obtained by substituting the OMC in Equation 5.8 of the compaction curve.

$$\gamma_{dmax} = a_1 OMC^3 + a_2 OMC^2 + a_3 OMC + a_4 \quad [5.15]$$

5.9 Filter paper test

Elgabu (2013), Mabirizi and Bulut (2009), and Houston *et al.* (1994) investigated the various techniques for soil suction measurement. This research conducted a comparative study on indirect and direct methods for soil suction assessment. For expansive soil, the chilled-mirror hygrometer and the filter paper methods qualified as cost-effective techniques. A more detailed review of different soil suction measurement methods is presented in Chapter 4, Part 2. In this research, Whatman No. 42 type filter paper provided the appropriate conditions for suction measurement (Ashless circles 70 mm diameter, Cat No. 1442-070).

5.9.1 Filter paper calibration

The accuracy of the soil suction measurement using filter paper is highly dependent upon the paper calibration curve. The calibration curve can be adopted or obtained through the calibration process. Figure 5.8 shows the calibration curves of Whatman No. 42 filter paper and Schleicher and Schuell No. 589 White Ribbon filter paper as given by ASTM D5298-16. Other calibration curve equations for Whatman No. 42 and Schleicher and Schuell No. 589 filter papers appear in Chapter 4, Table 4.2. In-house calibrations are discussed in the following paragraphs.

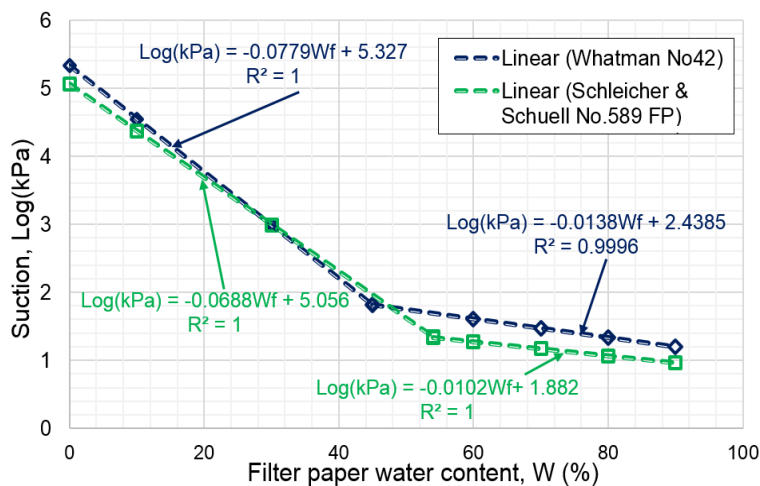


Figure 5.8: Filter paper calibration curves (ASTM D5298-16)

The filter paper calibration used a salt solution. Oven-drying the paper removed any residual moisture before testing and ensured an identical wetting path for each test (Swarbrick, 1995). Two laboratory technicians conducting the moisture content

estimation of the filter paper reduced its exposure time and moisture gain/loss during measurement. Tweezers and latex gloves were used to handle the materials during the calibration process. A salt (NaCl) solution was used to create an osmotic potential above 32 kPa. The solution was prepared with molality ranging from 0 to 2.7 to produce different magnitudes of total suction. Figure 5.9 presents the suction values produced by different NaCl molality.

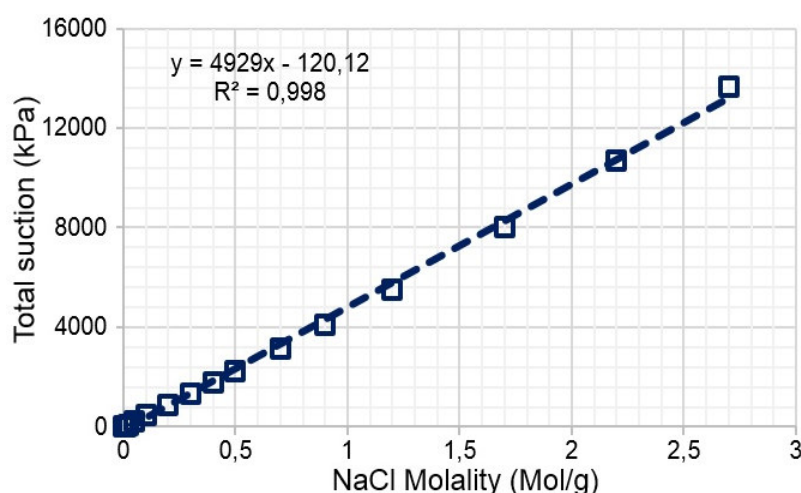


Figure 5.9: Total suction of NaCl at 20 °C (Lang, 1967)

Technicians placed approximately 200 ml of NaCl solution with a known molality into a 300 ml glass jar. A plastic platform positioned above the liquid level held two filter papers during the test. Duplicate specimens were used as a precaution against mishandling the paper. The tightly sealed jars were kept closed in a temperature-controlled chamber for an equilibrium period of 3 weeks. Figure 5.10 illustrates the assembly for the calibration of Whatman No. 42 filter paper.

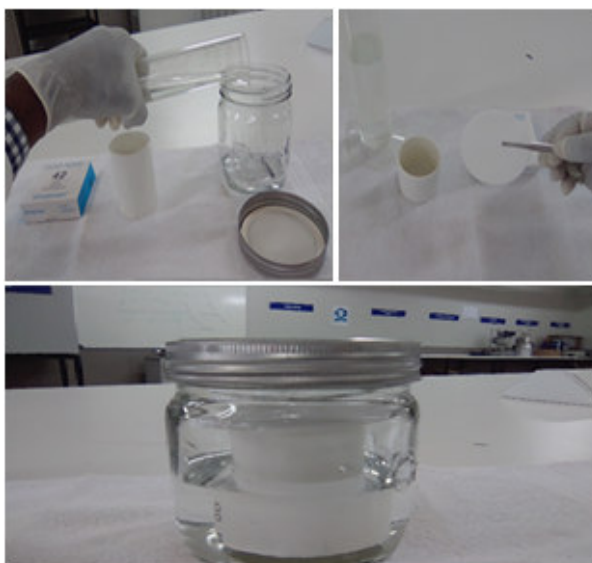


Figure 5.10: Whatman No. 42 Filter paper calibration setup

After the equilibrium phase, technicians weighed the moist filter paper. The weighing scales achieved an accuracy of 0.0001 g. To minimise exposure to the air, two technicians coordinated their efforts. While one opened the sealed glass jar, the second quickly placed the filter paper into the moisture tin (<5 seconds) using tweezers. Afterward, the technicians weighed each moisture tin with the wet filter paper and recorded their results. Finally, the technicians placed all the moisture tins into the drying oven, maintained at 105 ± 5 °C, for 24 hours with the lids half-open to allow evaporation. The final weighing process began by closing the lids and allowing them to equilibrate in the drying oven for five minutes. Technicians removed the moisture tins from the drying oven and placed them on a metal block to cool for about 20 seconds. The technicians then quickly weighed the moisture tin and filter paper inside. Finally, the technicians removed the filter paper and weighed the cooled, empty tins within a few seconds. The calibration curve consisted of values of NaCl-derived suction (y-axis) plotted against the moisture content of the filter paper (x-axis). Figure 5.11 presents the Whatman No. 42 filter paper calibration curves designated as the author calibration curve with Equations 5.16 and 5.17.

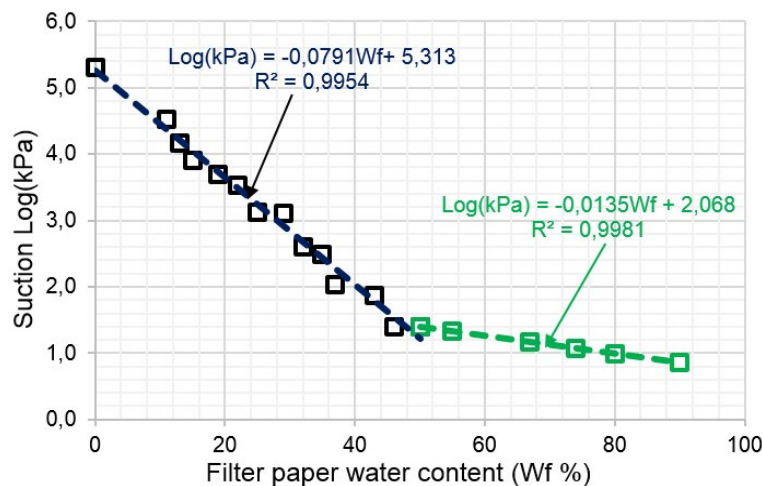


Figure 5.11: Whatman No. 42 filter paper calibration curve (Present work)

$$\text{Log}(\psi) = -0.0791W_f + 5.313 \quad 0 \leq W_f \leq 50 \quad [5.16]$$

$$\text{Log}(\psi) = -0.0135W_f + 2.068 \quad 50 < W_f \leq 90 \quad [5.17]$$

5.9.2 Soil suction estimation

During sample preparation for the zero swelling test, suction test, and consolidated undrained test, each sample produced compacted specimens with different moisture content and dry unit weight. A sharpened drive cylinder pushed into the compacted soil specimen, producing a cylindrical sub-specimen 75 mm in diameter and 70 mm long for the soil suction test. Soil suction tests using the filter paper method were conducted according to ASTM D5298-16. Materials used for the test included filter paper, sensitive balance (0.0001 g nearest accuracy), a temperature-controlled chamber, a drying oven, moisture tins, a glass jar, tweezers, gloves, a PVC O-ring, electrical tape, and an aluminium block. Table 5.4 in Appendix 2 shows the filter paper datasheet. Figure 5.12 shows the soil suction test using a filter paper configuration. Cutting the soil specimen in half allowed the best contact with the filter paper for matric suction measurement. The filter paper achieved the best contact with flat and smooth specimen surfaces. Tweezers provided the best way to handle the filter paper during placement. With the top half in place, electrical tape provided an outer seal around the filter paper-specimen interface (yellow tape in Figure 5.12).

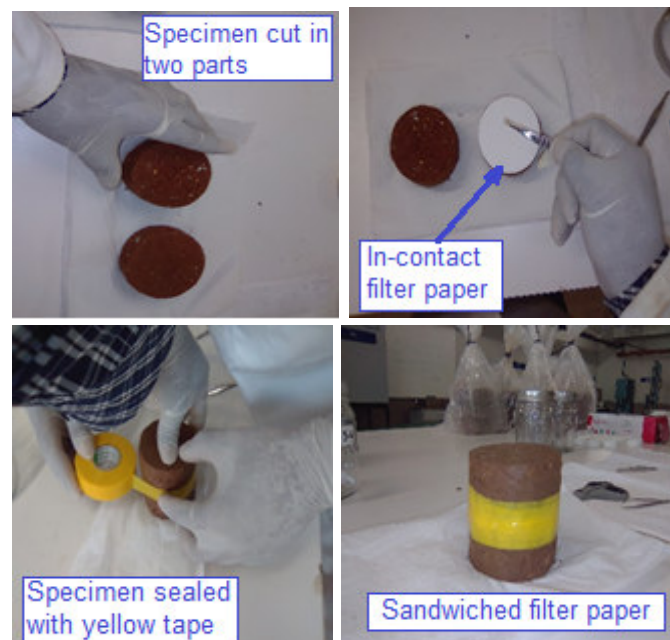


Figure 5.12: In-contact filter papers for matric suction measurement

Figure 5.13 describes the next steps for soil suction measurement. A clean PVC O-ring was inserted, with the sharp edge facing up, on top of the sealed soil specimen. The non-contact filter paper was placed over the PVC O-ring for total suction

measurement. The glass jars were sealed using an airtight lid to prevent moisture exchange. The glass jars were labelled and placed in a temperature-controlled ice chest for an equilibrium time of 3 weeks. The temperature fluctuations are maintained low, preferably 25 ± 1 °C. Table 4.1 in Chapter 4 shows the equilibrium time for filter paper (Leong *et al.* 2002).



Figure 5.13: Non-contact and in-contact filter paper for total and matric suction measurement

The filter papers were retrieved and weighed after the equilibrium time was completed (Figure 5.14). The water contents (W_f) in non-contact filter paper and contact filter paper were calculated using Equation 5.18 for total suction and matric suction measurements, respectively. The suction values were determined using the calibration curves for water content versus suction in Figure 5.11.

$$W_f = \frac{M_w}{M_f} = \frac{m_1 - m_2 - T_c + T_h}{m_2 - T_h} \times 100 \quad [5.18]$$

Where:

W_f = water content of FP [%]

m_1 = mass of wet FP + cold tin mass [g]

m_2 = mass of dry FP + hot tin mass [g]

T_c = cold tin mass [g]

T_h = hot tin mass [g]

M_w = mass of water in FP [g]

M_f = mass of dry FP [g]



Figure 5.14: Weighing the filter paper in the moisture tin

5.10 Design of the soil water characteristic curve

The soil water characteristic curve (SWCC) represents the relationship between total suction and the amount of moisture present in a soil specimen. Total suction is the equivalent suction derived from the measuring the partial pressure of water vapour in equilibrium with free pure water (Fredlund *et al.* 2012). Several authors have developed models that fit their experimental data and describe the SWCC. This research compares the experimentally measured values of volumetric moisture content with predictions made by models from Van Genuchten (1980), Durner (1994), Fredlund and Xing (1994), Kosugi (1996) and Seki (2007). In this work, the SWRC Fit software performs non-linear fitting of the SWCC using the five models mentioned above. It determines fitting parameters, the Akaike information criterion (AIC), and the coefficient of determination. The SWRC Fit program is written in GNU Octave (Seki, 2007). It employs a logarithmic scale to accommodate higher suction values, with the air-entry value chosen to provide the best fit. The filter paper technique discussed earlier provides accurate water content values at prescribed suction levels from compacted soil specimens. Other water content/soil suction values are estimated by interpolating the measured values. Chapter 2, Section 2.5.2, describes the mathematical equations of the DB, FX, VG, LN, and BL models. Table 5.5 summarises the types of SWCC models.

Table 5.5: Model types for soil-water characteristic curves

References	Unimodal	Bimodal
Van Genuchten (1980)	VG	-
Wolfgang Durner (1994)	-	DB
Fredlund and Xing (1994)	FX	-
Kosugi (1996)	LN	-
Seki (2007)	-	BL

5.11 Zero swelling test (ZST)

The swelling stress acts on the interface between the soil specimen and the device, preventing the soil from moving further upward due to wetting. This method is also called the zero swell test (Basma *et al.* 1995; Attom and Barakat, 2000). The method selected for this research is the zero swell test. The ZST was conducted in line with ASTM D4546-14. Cantillo *et al.* (2017), Tu and Vanapalli (2016), Forouzan (2016), and Israr *et al.* (2014) proposed predictive equations to measure the swelling stress of expansive soils as an alternative to oedometer testing. The oedometer device confines the specimen in the lateral direction via a steel ring, allowing only vertical deformation. Therefore, swelling will exhibit upward vertical movement, while compression or shrinkage will exhibit downward movement. Figure 5.15 illustrates the general behaviour of a soil specimen undergoing the stages of a swell test. In the figure, the specimen has an initial void ratio of 0.80. Stage 1 loads the specimen with a vertical stress of 100 kPa, causing it to compress slightly to a void ratio of 0.785. Stage 2 unloads the specimen, which rebounds slightly along the green line. Stage 3 (blue line) inundates the specimen, causing it to swell to a void ratio of 0.908. Stage 4 (red line) applies a staged vertical load until the void ratio returns to its value after stage 1. The stress needed for this condition is the swell pressure (at a given moisture content). Variations of swell tests apply more or less vertical stress or change the moisture content to provoke a different degree of swelling in stage 3.

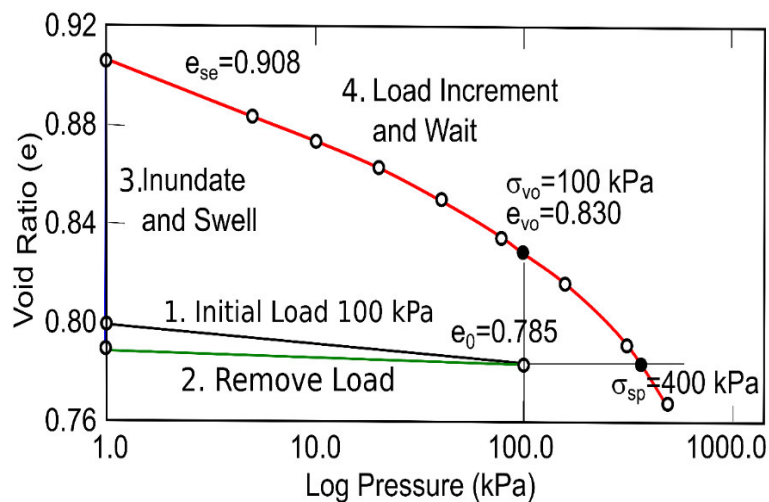


Figure 5.15: Deformation versus vertical stress, single-point test (ASTM D4546-14)

Another set of compacted soil specimens, identical to the one used for the suction test, provided the material for the swelling stress tests. A jack inserted the compacted soil specimen inside the consolidation ring without disturbing the soil specimen. The consolidation ring with the compacted soil specimen and vertical loading system prevented any vertical movement during inundation (Stage 3, Figure 5.15). The consolidation ring and compacted soil specimen were placed in the circular consolidation cylinder. Figure 5.16 shows the conventional oedometer setup for swelling stress estimation. Before submerging the soil specimen in water, the load application bar was adjusted and reset to zero to measure the vertical displacement of the compacted soil specimen due to the addition of water. The soil specimen was soaked using tap water. Surcharges were added in small increments to prevent the soil specimen from swelling. The process continued until the soil specimen ceases to heave. The test completion occurred once no further deformations (< 0.05 mm) were observed for several hours. Therefore, the total stress value applied to prevent the swelling of the soil specimen was considered the swelling stress and calculated using Equation 5.19.



Figure 5.16: Swelling stress test setup

$$P_s = \frac{\left[\sum_{i=1}^n M_i \right] \times g \times L_r}{\frac{\pi \times \varphi^2}{4} \times 1000} \quad [5.19]$$

Where:

- P_s = swelling stress [kPa]
- $\sum_{i=1}^n M_i$ = total surcharges [kg]
- g = gravity (9.81) [$\text{m} \cdot \text{s}^{-2}$]
- L_r = beam ratio [m]
- n = number of surcharge [unit]
- φ = internal diameter of the consolidation ring [m]

5.12 Triaxial testing

The triaxial specimens were prepared, compacted, and tested according to TR 2002/21 CSIR Protocol for Triaxial Testing, which agrees with ASTM D4767-99 but uses different specimen dimensions. The specimen measured 300 mm in height and 150 mm in diameter with a diameter/height ratio of 1.2. The soil specimens were prepared within the range of 11 – 12 kg. After oven drying, a sufficient mass of soil (plus 10 %) and water were mixed to produce a specimen. The water volumes were based on Proctor test results. Vibro-compaction required five equal layers, each with a duration of five minutes. The vibro-compaction device appears in Figure 5.17, where the lower table vibrates the entire specimen assembly. An extruder removed the specimen from the mould, which was then trimmed to the required dimensions and weighed (Figure 5.18). The specimens awaiting triaxial testing appears in Figure 5.19.



Figure 5.17: Vibro-compaction setup



Figure 5.18: Specimen extruder setup



Figure 5.19: Specimens awaiting triaxial testing

The equipment utilised for triaxial testing comprises the following main components (Figure 5.20): a computer-controlled hydraulic frame, the pressure chamber that houses the specimen during testing, electronic instrumentation including a load cell, and displacement transducers. Instron Triaxial Software recorded the test data and generated stress-strain curves and failure envelopes for all tests. The consolidated undrained (CU) triaxial test measured the shear strength characteristics of the prepared soil specimens. Pore pressures varied significantly during the consolidation phase, where only an average value could be estimated. However, pore pressures were measured during the shearing phase, enabling the determination of effective stress conditions. A set of three static triaxial tests provided data for each density/water content condition. The static triaxial tests were conducted for all soil specimens awaiting triaxial testing. The confining pressures for these three tests are 50 kPa, 100 kPa, and 150 kPa.



Figure 5.20: Triaxial testing apparatus

The axis-translation technique is commonly employed for testing unsaturated soils due to its adaptability with existing equipment for unsaturated soil testing by incorporating a high air entry (HAE) porous disk and air pressure source. This technique is effectively applied to volume changes and shear strength testing of unsaturated soils, including triaxial cells (Matyas and Radhakrishna, 1968; Wheeler and Sivakumar, 1995) and direct shear box (Gan *et al.* 1988). Fredlund and Rahardjo (1993) highlighted that the axis-translation technique for unsaturated soil measurement requires an extended period frame. As the test progresses, pore air diffuses through the water in the HAE disk, forming air bubbles underneath the disk, potentially introducing errors in estimating pore water pressure and water volume. Romero (2001) noted that the axis-translation technique may not yield rapid results when used for matric suction estimation. Nevertheless, the axis-translation technique requires maintaining continuity between the pore water in the soil material and water in the measurement device to accurately represent the existing suction within the soil specimen. Another

study by Blight (2013) revealed several challenges related to the shear strength measurement for unsaturated soils. These challenges include the need to conduct a large number of tests to establish the variation of shear strength with matric suction. Achieving equilibrium in soil specimens before testing requires a significant amount of time. The testing process demands specialised equipment for unsaturated soil testing, making it both expensive and complex. The axis-translation technique directly measures matric suction values within a suction range of 0 to 1500 kPa. Rao and Revanasiddappa (2000) stated that in compacted fine-grained soil specimens, the matric suction values fall within the range of 50 kPa to 8000 kPa at saturation values of 35% to 90%. Fondjo *et al.* (2021) reported that for compacted expansive soils, the matric suction values range from 40 kPa to 7700 kPa, and the total suction values range from 55 kPa to 10 000 kPa. Therefore, the axis-translation technique's suction range was found to be smaller than the matric suction range in compacted expansive soils.

The filter paper was employed to measure the pore air and pore water pressure, with the difference between these pressures equivalent to the matric suction within the compacted soil specimens used in triaxial testing. Fredlund *et al.* (2012) reported that the conventional triaxial apparatus could be utilised to conduct the undrained test on unsaturated soil specimens, and the results can be described in the context of unsaturated shear strength. However, the friction angle (ϕ^b) and related matric suction cannot be directly estimated from the conventional triaxial apparatus. Adapting the conventional triaxial cell was necessary to accommodate the measurement of pore water pressure and pore air pressure for unsaturated shear strength parameters. In this study, the unsaturated shear strength was determined using Equation 3.30. The friction angle (ϕ^b) for the unsaturated shear strength related to the matric suction was measured as the slope between the shear strength and matric suction curves.

Figure 5.21 depicts the procedures for setting up the compacted soil specimens for testing. Awaiting compacted soil specimens were positioned on the base pedestal of the triaxial cell (Figure 5.21a). The soil specimen was sealed airtight using a triaxial protective rubber membrane, held in place by a cylinder mould (Figure 5.21b). The cylinder mould was retrieved from the soil specimen, and the soil specimen was

connected with pore-pressure pipes (Figure 5.21c). The base pedestal was cleaned from debris using pressurised air, and the pressure chamber that houses the soil specimen was positioned using tie rods and connected with cell pressure pipes accordingly. The triaxial cell was then placed on a triaxial loading frame for testing (Figure 5.21d).

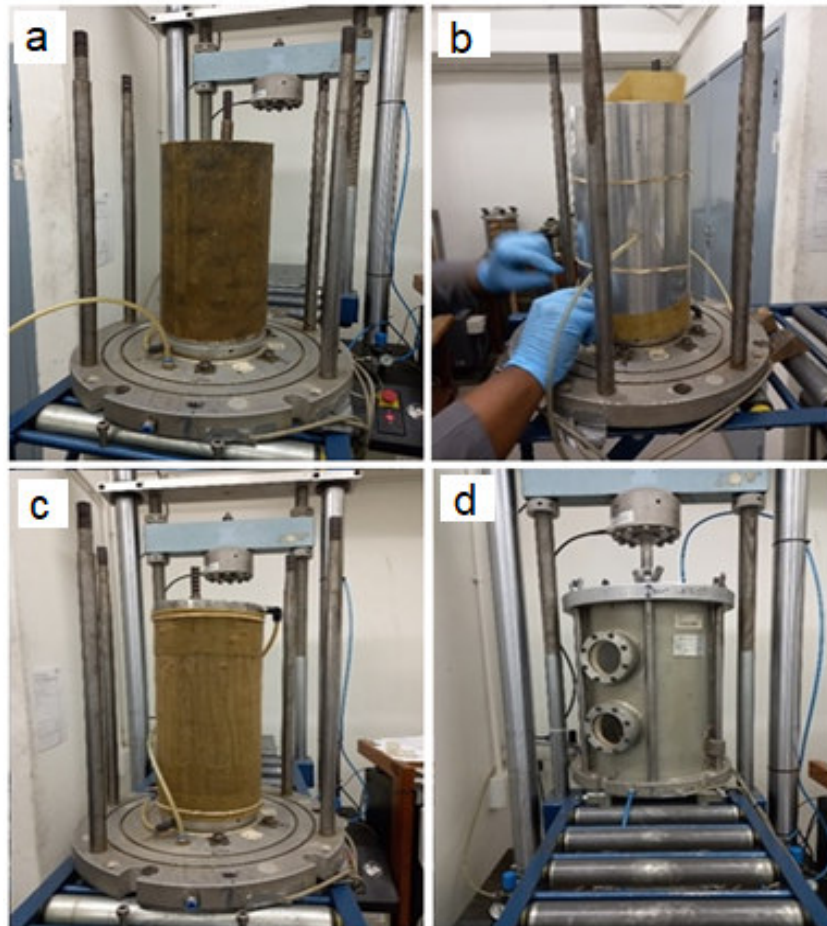


Figure 5.21: Main procedures of setting up specimen

The triaxial tests were carried out on the prepared soil specimens. Figure 5.22 illustrates a typical specimen before and after shear failure. During the triaxial test, the specimen was subjected to vertical loading and deformed at a constant rate of 2 mm per minute of vertical displacement until shear failure occurred on the soil specimen. Simultaneously, the vertical strain was recorded using a linear displacement transducer. The shear strength characteristics of the soil samples were obtained at different water content. The testing is generally isotropic and referred to as a consolidated isotropic undrained triaxial test (CIU), which may or may not include pore pressure measurements. Extending the Mohr-Coulomb shear strength criterion to

unsaturated soils requires evaluating the saturated shear strength characteristics (c' , ϕ'). The friction angle (ϕ^b) for the unsaturated shear strength related to the matric suction was determined as the slope between the shear strength and matric suction curves.

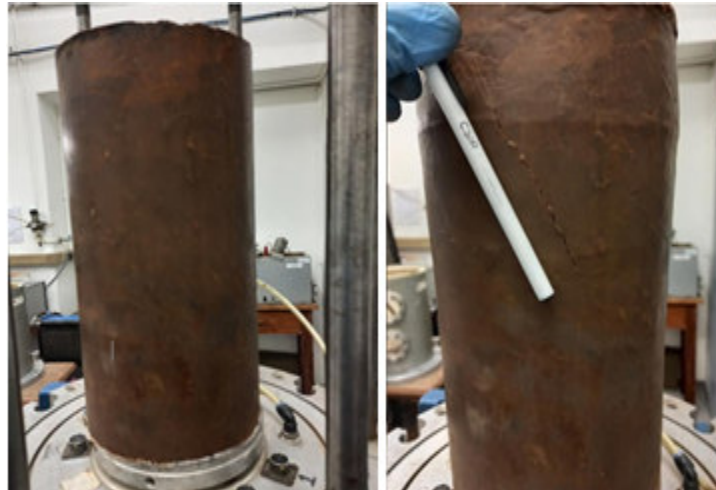


Figure 5.22: Typical specimen before and after failure

5.13 Geotechnical predictive model design

In geotechnical engineering practice, particularly for large-scale construction projects, estimating geotechnical characteristics in the laboratory, such as mineral composition, compaction characteristics, suction measurement, and unsaturated shear strength characteristics, is expensive and requires significant effort and time. These resources can be saved by utilising geotechnical predictive models during the initial phases. Designing models to predict the geotechnical characteristics of saturated and unsaturated soils becomes critical. The term “modelling” in this research work refers to multivariate regression analysis for designing geotechnical predictive models. Modelling is a quantitative examination technique used when the investigation involves analysing and modelling a few variables, where the relationship includes an outcome variable and one or more covariates. In other words, multivariate regression analysis is a method to test the relationship between an outcome variable and one or more covariates. The modelling process conducted in this study follows these steps: relevant data collection, selection of covariates (relevant soil parameters) for each geotechnical predictive model, model specification, and model validation. The geotechnical predictive model was designed using NCSS 12 Software.

The data collection was performed as follows: Laboratory investigations were conducted according to the testing program of this study. The data collection includes the mineralogy composition, physical properties, swelling properties, hydro-mechanical characteristics, and unsaturated soil characteristics. The outcome variables and the covariates were classified as quantitative. The value of R-squared (R^2) and the probability value (p-value) were essential to select the relevant covariates related to each geotechnical predictive model. However, the primary criterion for selecting the relevant covariate was the p-value. A hypothesis test was performed to determine the p-value coefficient of each covariate. When the p-value coefficient of the covariate was found to be higher than 0.05, the coefficient was considered zero, viewed as insignificant, and removed from the model. The data was divided into the following groups:

- Unsaturated soil characteristics (soil suction, SWCC, air entry value, unsaturated shear strength).
- Saturated soil characteristics (saturated shear strength, compaction features, specific gravity).
- Swelling properties (swelling stress, free swell index, and free swell ratio).
- Geotechnical soil indexes (consistency limits, initial water content, and clay activity).
- Grain size distribution features (gradation, measure of gradation) and mineralogical composition.

The model selection was conducted by testing different models, including logarithmic, quadratic, linear, cubic, power, exponential, and growth. It was found that the linear model displays a higher strength correlation and describes the best-fitting equation. Johnson (2017) reported that the predictive multilinear model takes the form of Equation 5.20.

$$Y = +\xi_0 + \left[\sum_{i=1}^n \xi_i \cdot Z_i \right] + \varepsilon \quad [5.20]$$

Where:

Y = outcome variable

ξ_0 = intercept

ξ_i = regression coefficients, $i = 1, 2, 3, \dots, n$

Z_i = covariates, $i = 1, 2, 3, \dots, n$

n = number of covariates

ε = random error term

The validation process of the designed geotechnical predictive model was conducted as follows: Compare the predicted values with the values obtained from laboratory testing, compare the predicted values with the values obtained from other existing models found in recent literature. Then assess the R-squared (R^2), probability value (p-value), and standardised residual pattern (SRP). The degrees of correlations between the dependent variables were interpreted according to Table 5.6 proposed by Kalayci (2010). Finally, the validation of the geotechnical predictive model was achieved using other experimental soil datasets available in recent literature.

Table 5.6: Degrees of correlation between dependent variables (Kalayci, 2010)

Coefficient of correlation (R^2)	Degrees of correlation
0.00 - 0.25	Very weak correlation
0.26 - 0.49	Weak correlation
0.50 - 0.69	Moderate correlation
0.70 - 0.89	Strong correlation
0.90 - 1.00	Very strong correlation

CHAPTER 6: ANALYSIS OF GEOTECHNICAL TEST RESULTS AND MODELLING

6.1 Soil classification

6.1.1 Particle size analysis

In soil mechanics, it is essential to quantify the grain sizes in soil. Soil material consists of grains of different sizes. The grain sizes are classified using the grain size distribution. Figures 6.1 through 6.4 show particle size distribution (PSD) curves of tested soils. Table 6.1 presents the summary of particle size analysis of twelve samples. The results revealed that the fine-grained content is not uniform among the tested soils. WES exhibited the highest quantity of fine-grained material among the other soil samples, followed by WIS, BFS, and BES. A fraction greater than 50 % of these soils passed the No. 200 (0.075 mm) sieve. Nonetheless, BES 1 and 2 show respective fine-grained soil contents of 45.14 % and 48.31 %, higher than the sand content. The PSD curves are well-graded with no gaps and display a significant fraction of fine-grained material.

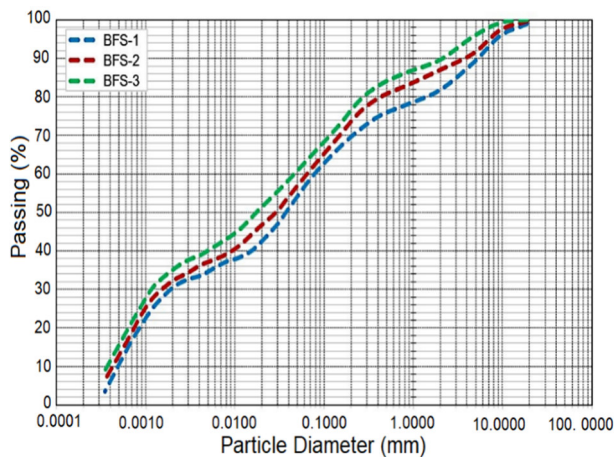


Figure 6.1: BFS particle size distribution curves

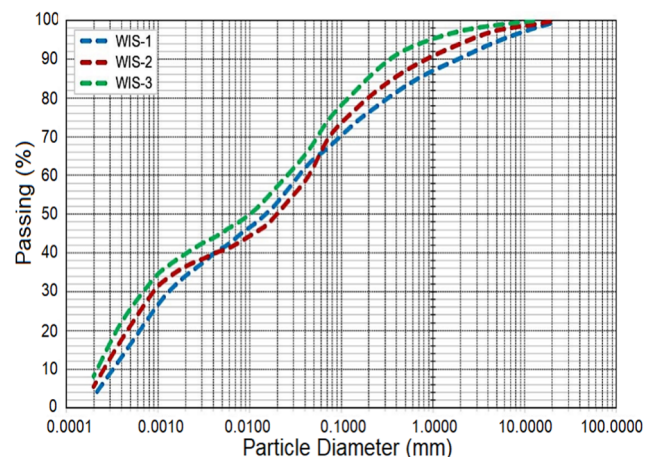


Figure 6.2: WIS particle size distribution curves

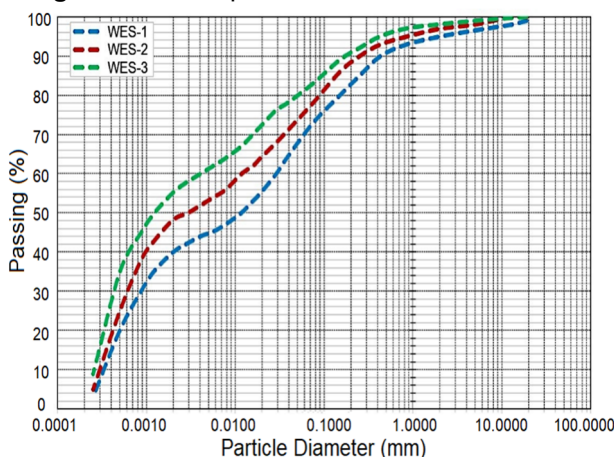


Figure 6.3: WES particle size distribution curves

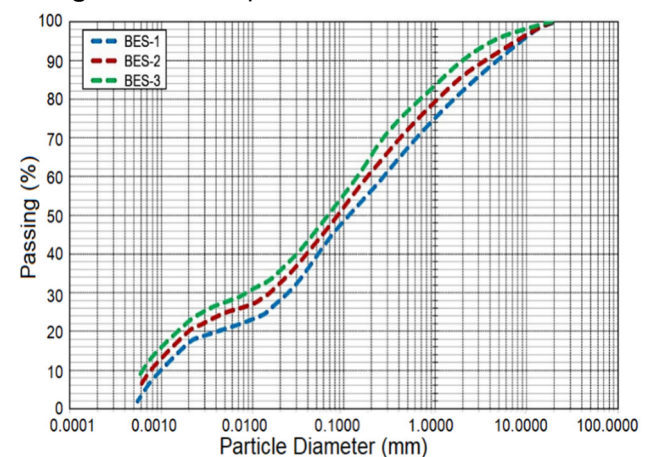


Figure 6.4: BES particle size distribution curves

The measure of gradation influences the geotechnical properties such as shear strength, compressibility, and hydraulic conductivity. The coefficients of uniformity (C_u) and coefficient of curvature (C_c) of soil material are given in Equations 6.1 and 6.2, respectively.

$$C_u = \frac{D_{60}}{D_{10}} \quad [6.1]$$

$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} \quad [6.2]$$

The C_u describes the soil grade and the range of particles in the soil material. The recorded C_u and C_c values of studied soil samples range from 18.5 to 269 and 0.067 to 3.36, respectively. The coefficients of uniformity ($C_u > 4$) indicate that BFS, WIS, WES, and BES soil samples are well-graded with no gaps and exhibit more than one type of particle, consisting of fine-grained soils, followed by sand and gravel. The findings are consistent with studies published by Lucian (2008), Chowdhury (2013), Deliktaş (2016), Fondjo *et al.* (2021a), Fondjo *et al.* (2021d), Fondjo *et al.* (2023a), and Fondjo *et al.* (2024). However, some recorded discrepancies can be attributed to the variability of soil material.

Table 6.1: Particle size analysis of twelve samples

Soils	Gradation (%)				Diameters			Coefficients	
	Fines		Sand	Gravel	D ₁₀ μ m	D ₃₀ μ m	D ₆₀ mm	C _c	C _u
	Clay	Silt							
BFS-1	30.40	29.11	29.39	10.09	0.50	2.00	0.080	0.100	160
BFS-2	32.20	29.63	29.49	8.38	0.40	1.50	0.060	0.094	150
BFS-3	35.07	30.11	30.48	4.32	0.36	1.25	0.045	0.097	125
WIS-1	34.03	33.49	26.80	4.85	0.34	1.33	0.036	0.145	106
WIS-2	36.50	33.60	27.20	2.45	0.27	0.90	0.045	0.067	167
WIS-3	39.73	35.05	23.98	1.21	0.23	0.70	0.026	0.082	113
WES-1	40.00	33.00	23.50	2.56	0.33	0.90	0.030	0.082	90.9
WES-2	48.31	29.80	18.71	1.98	0.30	0.60	0.013	0.092	43.3
WES-3	55.25	27.73	15.92	1.10	0.27	0.43	0.005	0.137	18.5
BES-1	17.11	28.03	43.76	11.10	1.00	25.0	0.186	3.36	186
BES-2	20.11	28.21	43.61	8.08	0.8	17.50	0.190	2.01	238
BES-3	22.67	28.88	43.95	4.50	0.65	9.0	0.175	0.712	269

6.1.2 Consistency limits

Figure 6.5 illustrates the consistency limits of the twelve samples. All samples show reasonable variation within their respective groups (BFS, WIS, WES, BES). Such variation may result from the percentage of fines as well as the distribution of remaining particles. Samples BES 1, 2, and 3 exhibit consistency that would classify as “Low plasticity Clay” (CL), while the other three fall within the zone designated as “High Plasticity Clay” (CH). These results are consistent with studies published by Lucian (2008), Chen *et al.* (2012), Chowdhury (2013), Deliktaş (2016), Fondjo *et al.* (2021a), Fondjo *et al.* (2021d), Fondjo *et al.* (2023a), and Fondjo *et al.* (2024) who found similar limits for their samples with nearly identical particle size distributions.

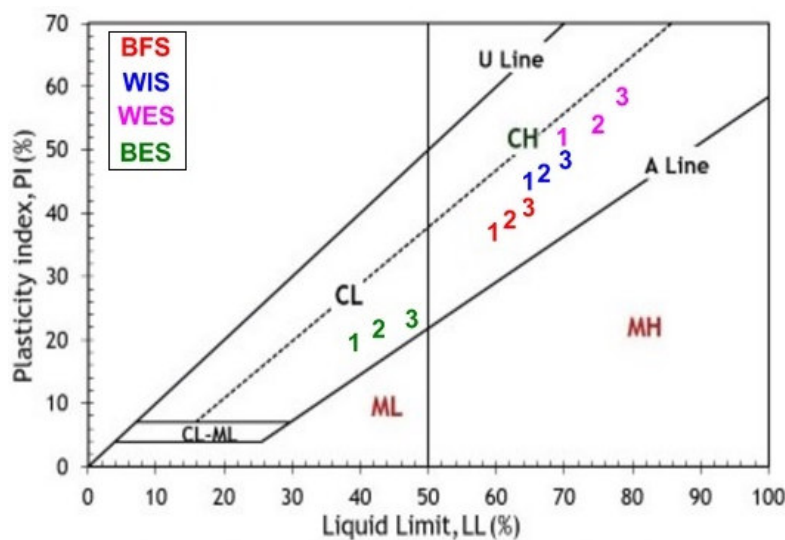


Figure 6.5: Casagrande plasticity chart of twelve samples

The plasticity charts describe the plasticity characteristics of soil material, and the soil samples are classified based on the Universal Soil Classification System (USCS). BFS, WIS, and WES have high plasticity, and BES has low plasticity. Linear shrinkage is the reduction in length expressed as a percentage of the initial soil material length when the moisture content at the liquid limit decreases under an oven-dried state. BFS 1, 2, and 3 have a linear shrinkage values of 9.28 %, 8.93 %, and 8.47 %. The recorded linear shrinkage values of WIS 1, 2, and 3 are 7.69 %, 7.41 %, and 6.97 %. WES 1, 2, 3 exhibit linear shrinkage values of 6.12 %, 5.72 %, and 5.38 %. The linear shrinkage values of BES 1, 2, and 3 are 16.67 %, 15.41 %, and 13.89 %. BES soils display the highest value of linear shrinkage, followed by BFS, WIS, and WES. These results can be explained by the smallest fines content in BES soils, followed by BFS, WIS, and

WES. The specific gravity is the ratio of the density of soil particles to the density of water. The final result is the average value from three tests for each soil specimen. Table 6.2 shows the consistency limits, linear shrinkage, and specific gravity of the studied soils.

Table 6.2: Consistency limits values of twelve samples

Soils	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Linear shrinkage (%)	Specific Gravity (Gs)	Natural water content (%)	USCS
BFS-1	58.98	22.16	36.82	9.28	2.64	18.31	CH
BFS-2	61.27	23.02	38.25	8.93	2.68	19.20	CH
BFS-3	64.60	24.27	40.33	8.48	2.71	22.31	CH
WIS-1	63.78	21.30	42.48	7.69	2.73	15.31	CH
WIS-2	66.22	22.12	44.10	7.41	2.76	20.21	CH
WIS-3	70.64	23.60	47.04	6.97	2.78	22.25	CH
WES-1	69.45	19.58	49.87	6.12	2.73	21.31	CH
WES-2	74.31	20.95	53.36	5.72	2.78	23.22	CH
WES-3	78.94	22.26	56.68	5.38	2.83	20.05	CH
BES-1	40.29	21.06	19.23	16.67	2.55	22.55	CL
BES-2	43.59	22.79	20.80	15.41	2.59	20.05	CL
BES-3	48.37	25.28	23.09	13.89	2.63	18.62	CL

6.1.3 Swelling potential analysis

Expansive soils are highly plastic materials that contain swelling clay minerals. The volume of these soils changes with moisture variation and can induce damage to constructions. The determination of the swelling potential of these soils is essential in geotechnical engineering practice. In this study, the evaluation of the swelling potential of soil materials follows both direct and indirect measurements. The direct measurement includes the expansive soil classification based on the free swell ratio (FSR) by Sridharan and Prakash (2000) and the expansive soil classification based on the free swell index (FSI) following IS 2720 Part 40 protocol. The indirect measurements include the classification of expansive soil based on liquid limit suggested by Holtz and Gibbs (1956) and the classification based on the Activity index with clay content developed by Carter and Bentley (1991).

Table 6.3 shows the test results of the swelling potential for the direct and indirect measurements. The results indicate that all the samples exhibit swelling features to

some extent. Nonetheless, some similarities and discrepancies appear among the four classification methods. Sridharan and Prakash (2000) and IS 2720 Part 40 classification results show that WES displays a high expansivity. BES exhibits a moderate expansivity based on IS 2720 Part 40 classification, and a low expansivity based on Sridharan and Prakash (2000) classification. Sridharan and Prakash (2000) and IS 2720 Part 40 expansivity classifications are very similar for BFS, WIS, and WES, identified as high plastic clay (CH). However, discrepancies were observed in the classification of BES, identified as low plastic clay (CL). Holtz and Gibbs (1956), and Carter and Bentley (1991) classification results revealed that BFS, WES, and WIS portrayed high expansivity. Nonetheless, Holtz and Gibbs (1956) classified BES as moderate expansivity. Carter and Bentley (1991) classified BES as low expansivity. Holtz and Gibbs (1956) and Carter and Bentley (1991) overestimate the swelling potential compared to Sridharan and Prakash (2000) and IS 2720 Part 40 results for high plastic soils BFS, WIS, and WES. Holtz and Gibbs (1956) and Carter and Bentley (1991) classification results reveal that BFS, WIS, WES portray high expansivity. Nonetheless, Holtz and Gibbs (1956) classified BES as moderate expansivity. Carter and Bentley (1991) classified BES as low expansivity. Holtz and Gibbs (1956) and Carter and Bentley (1991) overestimate the swelling potential compared to Sridharan and Prakash (2000) and IS 2720 Part 40 results for high plastic soils BFS, WIS, WES. Nonetheless, the four classification techniques portray similar results for low plastic soils BES.

These discrepancies result from differences in classification methods. The classification based on liquid limit (LL) did not efficiently evaluate the swelling potential because the LL represents the boundary between the plastic state and liquid state, and the evaluation based on activity index and clay percentage is related to the consistency limits. The similarities observed in Sridharan and Prakash (2000) and IS 2720 Part 40 methods originate from the similarity in their evaluation process. Further, the similarities observed in Holtz and Gibbs (1956) and Carter and Bentley (1991) classifications originate from the correlation between the consistency limits and the activity index. By and large, the direct measurement methods based on FSR and FSI display consistent results compared to the indirect measurement methods' classification based on LL and activity index/clay percentage.

Table 6.3: Swelling potential test results of twelve samples

Soils	Direct measurement				Indirect measurement			
	(IS 2720 Part 40)		(Sridharan & Prakash, 2000)		(Holtz and Gibbs, 1956)		(Carter and Bentley, 1991)	
	FSI (%)	Expansivity	FSR	Expansivity	LL (%)	Expansivity	Activity Index (Ac)	Expansivity
BFS-1	64.31	Moderate	1.64	Moderate	58.98	High	1.21	High
BFS-2	66.66	Moderate	1.70	Moderate	61.27	High	1.19	High
BFS-3	70.19	Moderate	1.79	Moderate	64.60	High	1.15	High
WIS-1	81.37	Moderate	1.73	Moderate	63.78	High	1.25	High
WIS-2	84.66	Moderate	1.80	Moderate	66.22	High	1.21	High
WIS-3	90.30	Moderate	1.92	Moderate	70.64	Very high	1.18	High
WES-1	116.60	High	2.20	High	69.45	High	1.25	High
WES-2	124.60	High	2.35	High	74.31	Very high	1.13	High
WES-3	132.60	High	2.50	High	78.94	Very high	1.03	Very high
BES-1	35.81	Moderate	1.17	Low	40.29	Moderate	1.12	Low
BES-2	38.56	Moderate	1.26	Low	43.59	Moderate	1.04	Low
BES-3	42.85	Moderate	1.40	Low	48.37	Moderate	1.02	Low

6.2 Soil mineralogy analysis

The behaviour of expansive soil is primarily controlled by the matric suction. Nevertheless, the mineralogical features impact the geotechnical behaviour of these soils. The XRD technique determined the dominant mineral composition of BFS, WIS, WES, and BES. Table 6.4 summarises the XRD results of the studied soil samples. The outcomes showed that the smectite is the predominating clay mineral in BFS, WIS, WES, and BES soils. A small amount of illite and trace illite was observed. Silica and feldspar minerals (plagioclase, K-feldspar) are the predominant non-clay minerals. A slight amount and trace of calcite appeared in soil samples. The significant amount of smectite can explain the swelling characteristics of these soils. These outcomes concurred with the swelling potential analysis results. Additionally, these results agreed with the study published by Fondjo and Theron (2020). Table 6.4a outlines the XRD graphs for twelve samples.

Table 6.4: XRD diffraction results of twelve samples

Soils	Smectite (%)	Silica (%)	Group of feldspar minerals		Illite (%)	Calcite (%)
			K-feldspar (%)	Plagioclase (%)		
BFS-1	56.83	12.47	23.51	3.29	1.89	2.01
BFS-2	58.00	14.00	24.88	3.12	trace	trace
BFS-3	61.15	11.93	19.01	2.63	3.30	1.98
WIS-1	58.22	25.08	10.42	2.45	2.02	trace
WIS-2	59.41	27.70	9.99	2.90	trace	trace
WIS-3	63.37	20.34	10.71	1.80	2.43	1.35
WES-1	67.05	19.98	10.66	2.31	trace	trace
WES-2	71.74	13.40	9.91	1.85	1.89	1.22
WES-3	76.21	11.69	8.57	1.25	1.14	1.18
BES-1	31.67	31.39	21.22	8.15	3.36	4.21
BES-2	34.26	30.75	20.32	7.04	2.11	5.52
BES-3	38.02	38.98	17.23	5.77	trace	trace

Table 6.4a: Outline of XRD graphs for twelve samples

Sample	Figure No.	Appendix No.
BFS-1	6.9	4
BFS-2	6.10	
BFS-3	6.11	
WIS-1	6.12	
WIS-2	6.13	
WIS-3	6.14	
WES-1	6.15	5
WES-2	6.16	
WES-3	6.17	
BES-1	6.18	
BES-2	6.19	
BES-3	6.20	

6.2.1 Analysis of smectite content response to soil features

Table 6.5 in Appendix 6 shows the response of smectite to soil features. The highlighted row displays the subsets used to correlate the smectite content. The influences of PI, FSR, FSI, and Gs on smectite content are analysed. Figures 6.21 and 6.22 depict the relationship between smectite content and soil parameters: PI, FSR, Gs, and FSI. Figure 6.21 shows the surface plot of smectite, PI, and FSR. The smectite content increases with the rise in PI and displays a strong correlation ($R^2=0.97$) described in Equation 6.3, Appendix 7. The smectite content increases with

an increase in FSR, showing a strong correlation ($R^2=0.90$) in Equation 6.4, Appendix 7. Based on the results, the presence of smectite in soil material leads to an increase in PI. Holtz and Gibbs (1956) noted the impact of PI on swell potential using LL to classify swelling potential. The results support the analysis conducted by Spagnoli *et al.* (2018) on the correlation between the PI and LL of smectite, which highlighted the influence of clay mineralogy on consistency limits. Figure 6.22 displays a surface plot of smectite, Gs, and FSI. Smectite content increases as Gs increases, showing a strong correlation ($R^2=0.85$) described in Equation 6.5, Appendix 7. Similarly, the smectite content increases with an increase in FSI, displaying a strong correlation ($R^2=0.87$) given in Equation 6.6, Appendix 7. These outcomes can be explained by the incremental nature of FSR and FSI as the smectite content increases in expansive soil material. Sridharan and Prakash (2000) emphasised the impact of smectite on FSR, using it to classify swelling potential.

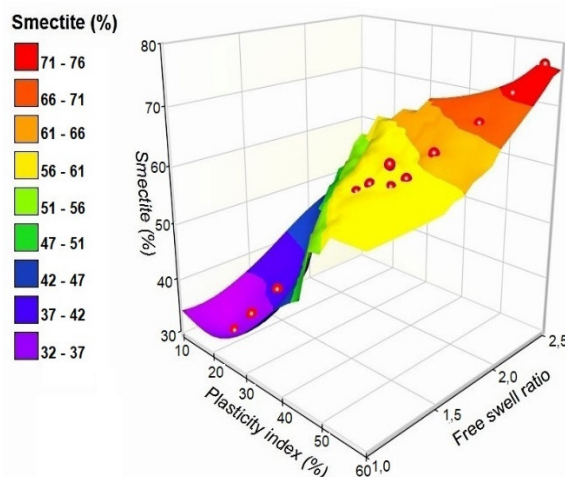


Figure 6.21: 3D response surface of smectite (%) dependence on PI (%) and FSR

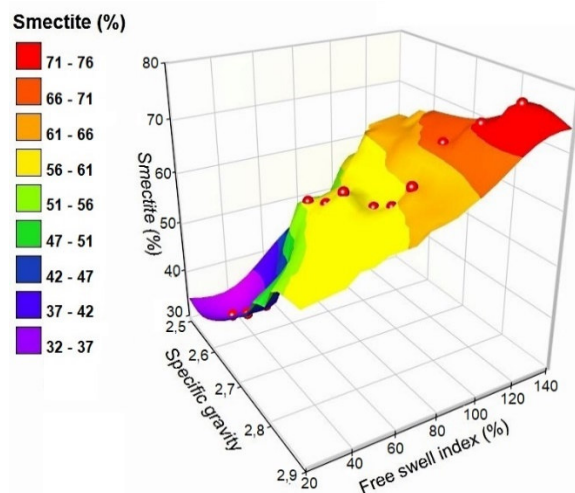


Figure 6.22: 3D response surface of smectite (%) dependence on Gs and FSI (%)

6.2.2 Investigation of silica content response to soil features

The response of silica content to soil features is presented in Table 6.6 in Appendix 6. The highlighted row showcases the subsets used to correlate the silica content. Therefore, the relationships between the silica content and PI, FSR, gravel content, and FSI are examined. Figures 6.23 and 6.24 portray the three-dimensional representation of the interactions between silica content and PI, FSR, gravel content, and FSI. Figure 6.23 displays a surface plot of silica content, plasticity index, and FSR. A moderate correlation ($R^2=0.50$) between silica content and plasticity index is

described in Equation 6.7, Appendix 7. The silica content decreases as the PI increases. An increment in FSR leads to a decrease in silica content, with a weak correlation ($R^2=0.46$) described in Equation 6.8, Appendix 7. Figure 6.24 portrays a surface plot of silica content, gravel content, and FSI. As the FSI increases, there is a reduction in silica content. The weak correlation ($R^2=0.38$) between silica content and FSI is describe by Equation 6.9, Appendix 7x. Similarly, an increase in gravel content corresponds to a decrease in silica content, showing a very weak correlation ($R^2=0.06$) as indicated in Equation 6.10, Appendix 7. Both Figures 6.23 and 6.24 show some local extremes, which stem from discrepancies in correlations.

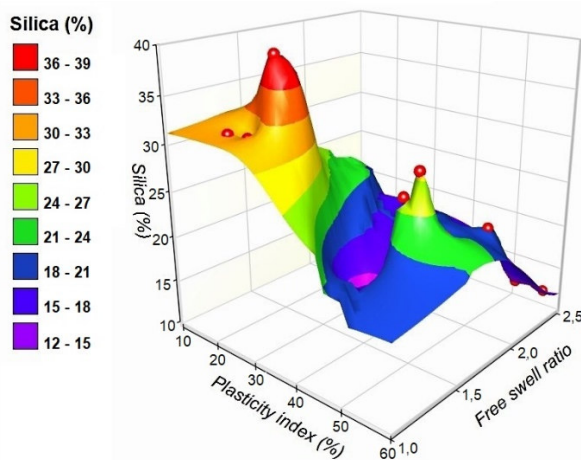


Figure 6.23: 3D response surface of silica (%) dependence on PI (%) and FSR

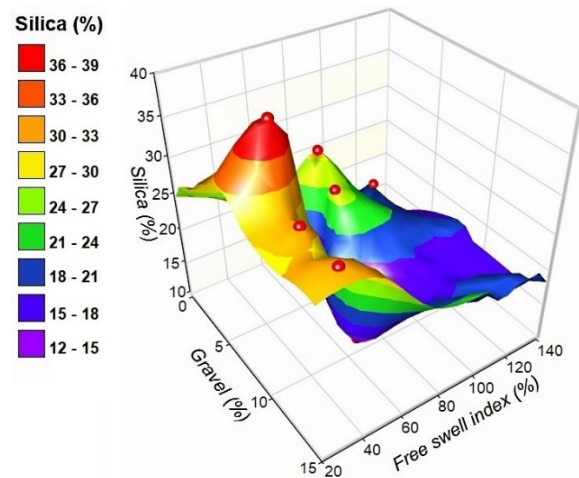


Figure 6.24: 3D response surface of silica (%) dependence on gravel (%) and FSI (%)

6.2.3 Determination of K-feldspar content response to soil features

Table 6.7 in Appendix 6 shows the response of K-feldspar content to soil features. The highlighted row indicates the appropriate subsets used to correlate the K-feldspar content. Thus, the correlations between the K-feldspar content and PI, FSI, Gs, and FSR are analysed. Figures 6.25 and Figure 6.26 present a three-dimensional representation of the correlation between the K-feldspar content and soil features, such as PI, FSI, Gs, FSR. The colour patterns represent different K-feldspar content values. Figure 6.25 displays a surface plot of K-feldspar content, PI, and FSI. A moderate correlation ($R^2=0.59$) between K-feldspar content and FSI is described in Equation 6.11, Appendix 7. The K-feldspar content decreases as the FSI increases. Additionally, the K-feldspar content reduces with increments in PI, portraying a weak correlation ($R^2=0.49$) as shown in Equation 6.12, Appendix 7. Figure 6.26 represents a surface plot of K-feldspar content, Gs, and FSR. The K-feldspar content decreases

as Gs increases. A moderate correlation ($R^2=0.64$) between K-feldspar content and Gs is described in Equation 6.13, Appendix 7. K-feldspar content reduces with increasing FSR, displaying a moderate correlation ($R^2=0.51$) as indicated in Equation 6.14, Appendix 7. Figures 6.25 and 6.26 exhibit local extremes originating from disparities in correlations. The PI, FSI, Gs, and FSR influence the K-feldspar content.

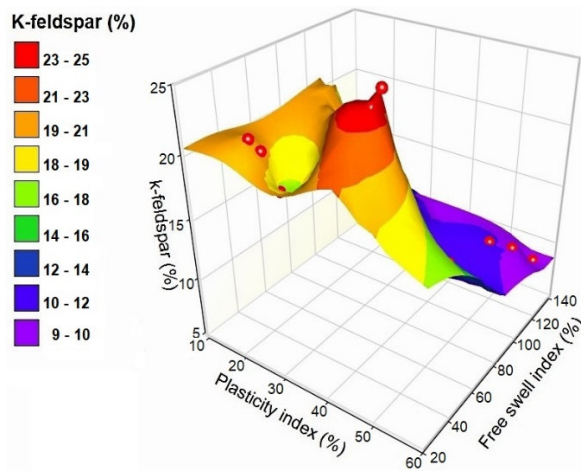


Figure 6.25: 3D response surface of K-feldspar (%) dependence on PI (%) and FSI (%)

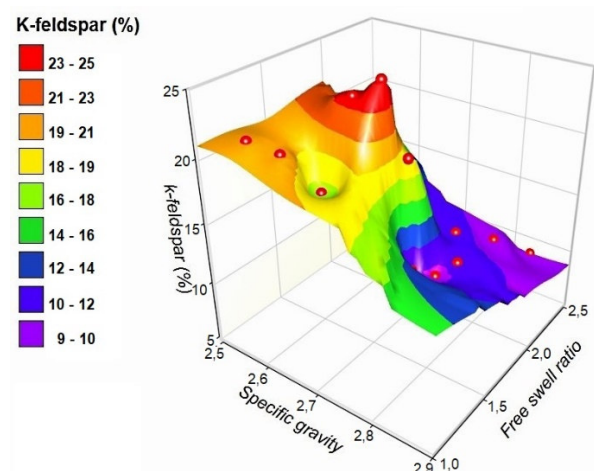


Figure 6.26: 3D response surface of K-feldspar (%) dependence on Gs and FSR

6.2.4 Assessment of plagioclase content response to soil features

Table 6.8 in Appendix 7 illustrates the response of plagioclase content to soil features. The highlighted row indicates the optimal subsets used to correlate the plagioclase content. Thus, the correlations between the plagioclase content and soil features such as LL, LS, FSI, Gs are analysed. Figures 6.27 and 6.28 depict the correlation between the plagioclase content and soil features including LL, LS, FSI, Gs. Figure 6.27 presents a surface plot of plagioclase content, LL, and LS. A strong correlation ($R^2=0.94$) with minimal disparities between plagioclase content and LL is described by Equation 6.15 in Appendix 7. The plagioclase content decreases with increasing LL. The plagioclase content decreases as LS increases, displaying a strong correlation ($R^2=0.93$) described by Equation 6.16 in Appendix 7. Figure 6.28 displays a surface plot of plagioclase content, FSI, and Gs. The plagioclase content decreases with the increment of FSI and exhibits a strong correlation ($R^2=0.72$) described in Equation 6.17, Appendix 7. Additionally, the plagioclase content reduces as Gs increases, showing a strong correlation ($R^2=0.85$) as described in Equation 6.18, Appendix 7. The local extremes observed in Figures 6.27 and 6.28 arise from some disparities in

correlation. Ultimately, LL, LS, FSI, and Gs collectively influence the plagioclase content.

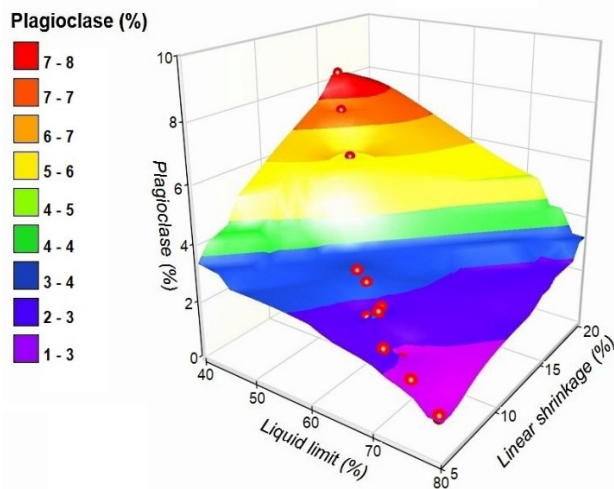


Figure 6.27: 3D response surface of plagioclase (%) dependence on LL (%) and LS (%)

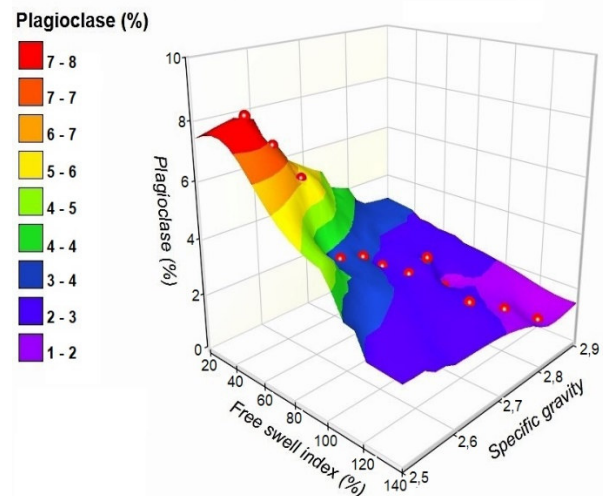


Figure 6.28: 3D response surface of plagioclase (%) dependence on FSI (%) and Gs

6.3 Compaction test results analysis

Compaction involves applying mechanical energy to soil material to decrease the void ratio and rearrange the particles. Soil compaction enhances the characteristics of an existing soil material or facilitates the placement of fill, as seen in the design of road bases, embankments, earth dams, reinforced earth walls, runways, and even in levelling surfaces during building construction. The primary goal of compaction is to increase soil shear strength and its bearing capacity, while also reducing soil permeability, minimising foundation settlement, improving the slope stability of embankments, and control undesirable volume changes. Figures 6.29 to 6.32 display the response of dry unit weight (γ_d) to water content (W) for BFS, WIS, WES, and BES. Table 6.10 in Appendix 8 presents the maximum dry unit weight (γ_{dmax}) and the corresponding optimum moisture content (OMC) for the studied soils. These results are consistent with the findings published by Chen *et al.* (2012), Chowdhury (2013), Fondjo *et al.* (2021d). Additionally, Figures 6.33 to 6.40 describe the response of compaction characteristics (OMC, γ_{dmax}) to other soil properties.

6.3.1 Dry unit weight and water content relationship for BFS and WIS

Figure 6.29 illustrates the relationship between water content (W) and dry unit weight (γ_d) for BFS. Starting at low W=12.5 %, as W increases, the particles in BFS 1, 2, and 3 develop thicker water films around them, which lubricate the particles and facilitate

their movement and reorientation into a denser structure. There comes a point where γ_d does not increase any further. At this stage, because of lower density of water compared to the density soil particles (Lambe, 1958; Daniel, 1993), γ_d decreases. The position of the γ_d vs W curves (OMC, γ_{dmax}) shifts for BFS 1, 2, and 3 due to the varying soil properties. Additionally, moisture content significantly influences the behaviour of these soils due to their higher fines content (> 50 %). These results are reasonable when one considers the impact of increasing fines content (Table 6.1).

Figure 6.30 depicts the relationship curve between γ_d and moisture content (W) for WIS. The compaction curves of WIS exhibit a similar shape. Beginning with low $W=14.5$ %, as W increases, the particles in WIS 1, 2, and 3 develop thicker water films around them, aiding in particle lubrication and facilitating their movement and reorientation into a denser structure. Similarly, there comes a point where γ_d does not increase any further. At this point, due to water lower density compared to the density of soil particles (Lambe, 1958; Daniel, 1993), γ_d reduces. The location of the γ_d vs W curves (OMC, γ_{dmax}) shifts for WIS 1, 2, and 3 due to differing soil properties. The moisture content has a significant impact on the behaviour of WIS 1, 2, 3 due to their higher fine content. These outcomes are rational when one considers the impacts of increasing fines content (Table 6.1).

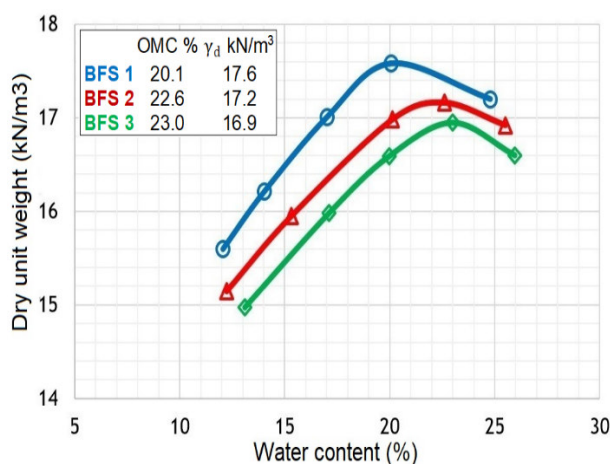


Figure 6.29: Water content-dry unit weight relationship for BFS

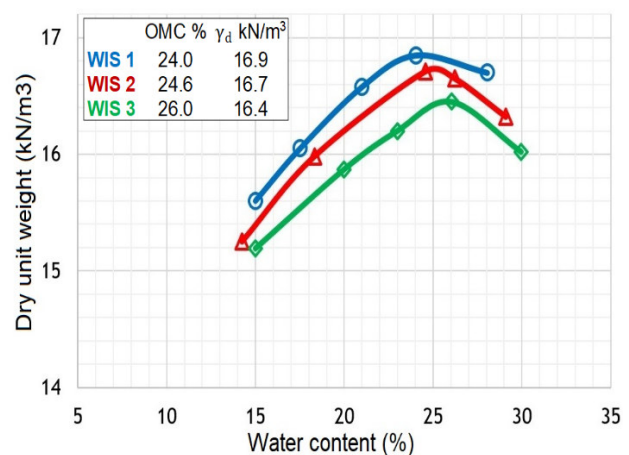


Figure 6.30: Water content-dry unit weight relationship for WIS

6.3.2 Dry unit weight and water content relationship for WES and BES

Figure 6.31 depicts the correlation between γ_d and water content (W) for WES. Starting at low $W=17$ %, as W increases, the particles in WES 1, 2, and 3 develop

thicker water films around them, which lubricate the particles and facilitate their movement and reorientation into a denser structure. Additionally, W reaches a point where γ_d no longer increases. At this stage, due to the soil particles higher density than the density of water (Lambe, 1958; Daniel, 1993), γ_d decreases. The shape of γ_d vs W curves (OMC, γ_{dmax}) shifts for WES 1, 2, and 3 due to differing soil properties. Moisture content significantly impacts the behaviour of these soils due to their higher fines content (> 50 %). Tarantino and De Cole (2008) reported the significant influence of fine-grained content materials during compaction. These results are realistic when one considering the influences of increasing fines content (Table 6.1). Figure 6.32 illustrates the γ_d and moisture content (W) relationship curve for BES. The compaction curves of BES exhibit a similar shape. Starting at low $W=9.5$ %, as W increases, the particles in BES 1, 2, and 3 develop thicker water films around them, lubricating the particles and facilitating their movement and reorientation into a denser structure. W reaches a point where γ_d no longer increases. At this point, water starts to replace soil particles in the mould, and due to its lower density compared to mineral grains, γ_d decreases. The location of γ_d vs W curves (OMC, γ_{dmax}) changes for BES 1, 2, and 3 due to varying soil properties. The moisture content influences the behaviour of BES 1, 2, 3 due to their fine content. These outcomes are reasonable when considering the impacts of augmenting fines content (Table 6.1).

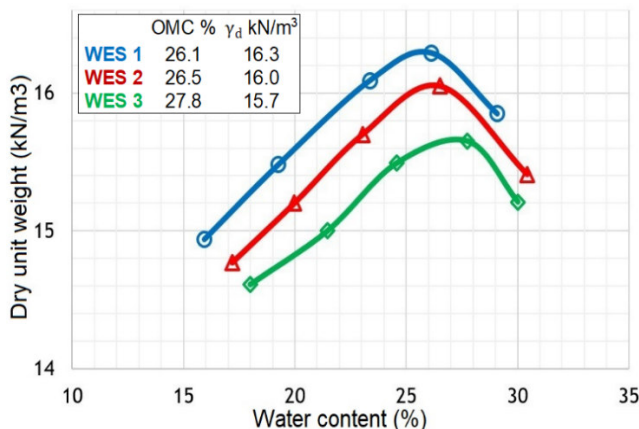


Figure 6.31: Water content-dry unit weight relationship for WES

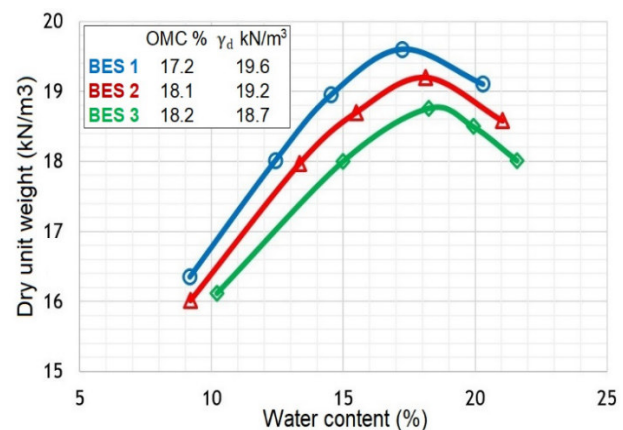


Figure 6.32: Water content-dry unit weight relationship for BES

6.3.3 Compaction features response to consistency limits

Figure 6.33 illustrates the interaction between γ_{dmax} , LL, and PI. Different colour patterns represent the distribution of γ_{dmax} values on the surface plot. The γ_{dmax} value decreases as the LL increases and exhibits a strong correlation as described in Equation 6.19, Appendix 8. Additionally, γ_{dmax} values decrease as PI increases and show a strong correlation given in Equation 6.20, Appendix 8. These results can be explained by the increase in consistency limits with higher clay content. Consequently, γ_{dmax} values decrease with an increase in clay content. These results are consistent with research published by Lah (2008), Kamarudin (2005), Naderi *et al.* (2012), Đoković *et al.* (2013), Idris *et al.* (2014), Arif *et al.* (2019), Firomsa and Quezon (2019), and Shien *et al.* (2015), which revealed a decrease in γ_{dmax} with increasing LL. The results align with studies published by Noor *et al.* (2011), Singh and Noor (2012), Idris *et al.* (2014), and Shien *et al.* (2015), which reported a reduction in γ_{dmax} with increasing PI.

Figure 6.34 presents a three-dimensional representation of the correlation between the OMC, LL, and PI. Different colours represent the distribution of OMC values on the surface plot. The OMC value increases as LL increases and demonstrates a strong correlation as described in Equation 6.21, Appendix 8. Additionally, the OMC values increase when PI increases and exhibits a strong relationship as described in Equation 6.22, Appendix 8. These results can be justified as follows: Consistency limits grow with the increase of clay content, leading to an increase in OMC. These findings align with the investigations published by Lah (2008), Kamarudin (2005), Naderi *et al.* (2012), Đoković *et al.* (2013), Idris *et al.* (2014), Arif *et al.* (2019), Firomsa and Quezon (2019), and Shien *et al.* (2015), which revealed an increase in OMC with increasing LL. The results are consistent with studies conducted by Noor *et al.* (2011), Singh and Noor (2012), Idris *et al.* (2014), and Shien *et al.* (2015), which reported an increase in OMC with the increase of PI. There is a strong correlation between OMC and consistency limits, making consistency limits good predictors of OMC.

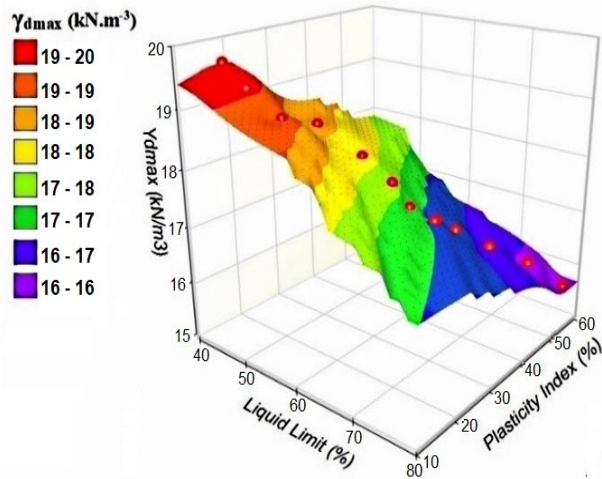


Figure 6.33: 3D response surface of γ_{dmax} dependence on LL (%) and PI (%)

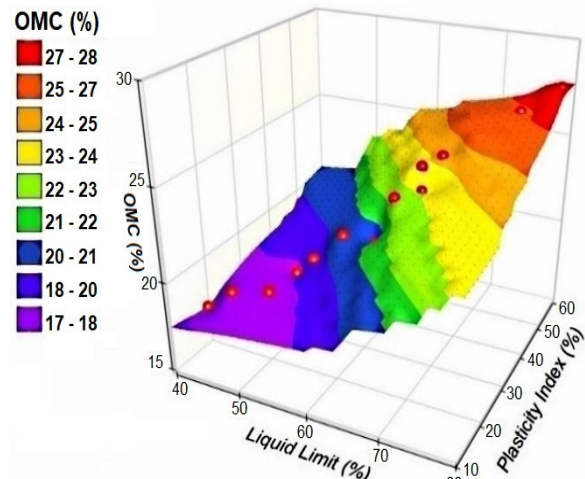


Figure 6.34: 3D response surface of OMC dependence on LL (%) and PI (%)

6.3.4 Compaction features response to fine (%) and sand (%)

Figure 6.35 illustrates the relationship between OMC and soil parameters: fine-grained content and sand content. OMC increases as the fine content increases and displays a strong correlation described in Equation 6.23, Appendix 8. OMC decreases when the sand content increases and exhibits a strong correlation as given in Equation 6.24, Appendix 8. When fine content increases, the soil absorbs more water, leading to an increase in OMC. As a result, when sand content increases, the soil's water absorption decreases, resulting in a reduction of OMC. These trends are consistent with studies reported by Naderi *et al.* (2012), Idris *et al.* (2014), and Shien *et al.* (2015) that demonstrate an increase in OMC with higher fine content. Similarly, the results align with studies by Naderi *et al.* (2012), and Shien *et al.* (2015) that highlight a decrease in OMC with increased sand content in the soil material.

Figure 6.36 presents a three-dimensional relationship between γ_{dmax} , fine-grained content, and sand content. The γ_{dmax} decreases as the fine content increases, showing a strong correlation described in Equation 6.25, Appendix 8. The unit weight per volume of the soil decreases with higher fine content, leading to a reduction in γ_{dmax} . This finding corresponds with results from investigations by Naderi *et al.* (2012), Idris *et al.* (2014), and Shien *et al.* (2015), which indicate a decrease in γ_{dmax} with increased fine content. Additionally, γ_{dmax} increases as the sand content increases, displaying a strong correlation given in Equation 6.26, Appendix 8. The unit weight per volume of the soil increases with higher sand content, resulting in an

increase in γ_{dmax} . These results are in line with studies published by Naderi *et al.* (2012), and Shien *et al.* (2015), which demonstrate an increase in γ_{dmax} with augmented sand content in the soil material. It is evident that both sand content and fine content influence γ_{dmax} .

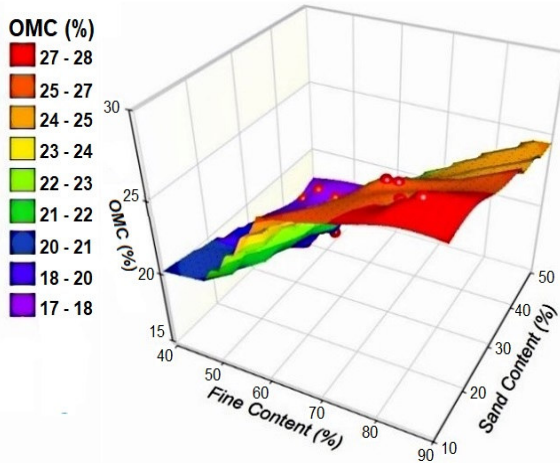


Figure 6.35: 3D response surface of OMC dependence on fine (%) and sand (%)

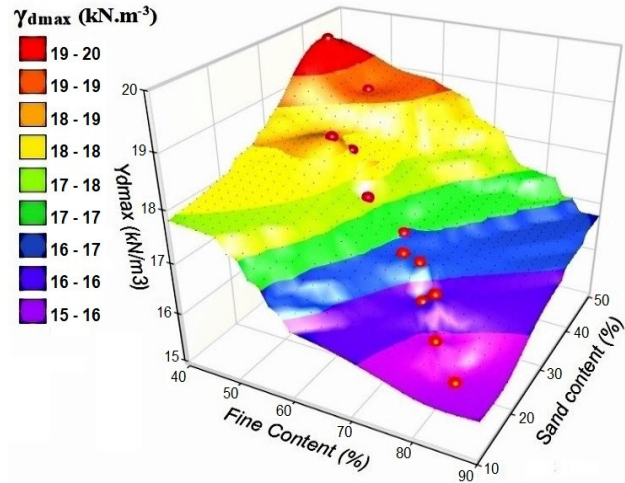


Figure 6.36: 3D response surface of γ_{dmax} dependence on fine (%) and sand (%)

6.3.5 Compaction features response to gravel (%) and specific gravity (G_s)

Figure 6.37 presents a three-dimensional representation of the relationship between γ_{dmax} , gravel content, and G_s . γ_{dmax} increases as the gravel content increases, showing a moderate correlation described by exponential Equation 6.27, in Appendix 8. The local extremes on the surface plot originate from discrepancies in the correlation between γ_{dmax} and gravel content. This trend aligns with investigations published by Lah (2008), Kamarudin (2005), Naderi *et al.* (2012), and Shien *et al.* (2015), which demonstrate that γ_{dmax} increases with higher gravel content. Additionally, γ_{dmax} decreases as G_s increases, displaying a strong correlation given by exponential Equation 6.28, in Appendix 8. This trend corresponds with studies conducted by Noor *et al.* (2011), Singh and Noor (2012), and Naderi *et al.* (2012), that indicate a decrease in γ_{dmax} upon the increment of G_s . Based on this observation, it is evident that G_s significantly influences γ_{dmax} .

Figure 6.38 displays a surface plot graph of OMC, gravel content, and G_s . OMC decreases as the gravel content increases, exhibiting a moderate correlation described by linear Equation 6.29 in Appendix 8. The result is consistent with

investigations published by Lah (2008), Kamarudin (2005), Naderi *et al.* (2012), and Shien *et al.* (2015), which demonstrate a decrease in OMC with increased gravel content. Additionally, OMC increases as G_s increases, showing a strong correlation described by linear Equation 6.30 in Appendix 8. This trend aligns with results published by Noor *et al.* (2011), Singh and Noor (2012), and Naderi *et al.* (2012) that indicate an increase in OMC upon increment of G_s . The gravel content and G_s influence the OMC of the soil material.

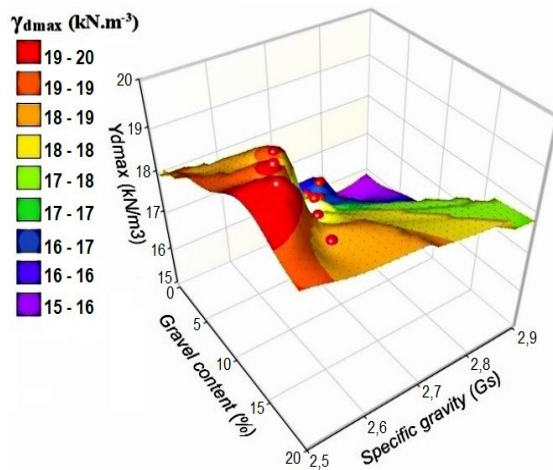


Figure 6.37: 3D response surface of γ_{dmax} dependence on gravel (%) and G_s

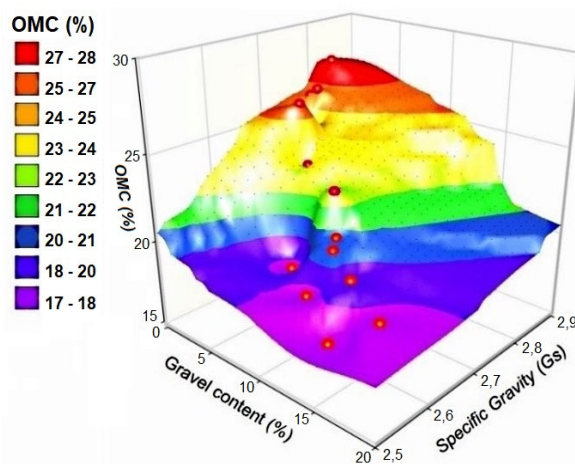


Figure 6.38: 3D response surface of OMC dependence on gravel (%) and G_s

6.3.6 Compaction features response to coefficient of curvature (C_c) and coefficient of uniformity (C_u)

Figure 6.39 presents a surface plot of γ_{dmax} , C_c , and C_u . γ_{dmax} increases as C_c increases, showing a strong correlation described by linear Equation 6.31 in Appendix 8. Additionally, γ_{dmax} increases as C_u increases, displaying a strong correlation described by exponential Equation 6.32 in Appendix 8. Figure 6.39 exhibits some local extremes. These extremes originate from discrepancies in correlation with $R^2 < 80\%$. C_c and C_u influence impact the magnitude of γ_{dmax} .

Figure 6.40 provides a three-dimensional description of the correlation between OMC, C_c , and C_u . OMC decreases as C_c increases, showing a moderate correlation described by exponential Equation 6.33 in Appendix 8. OMC decreases as C_u increases, displaying a strong correlation described by linear Equation 6.34 in Appendix 8. The local extremes in the surface plot originate from discrepancies in the correlation with $R^2 < 80\%$. It is evident that C_c and C_u influence the OMC of soil material.

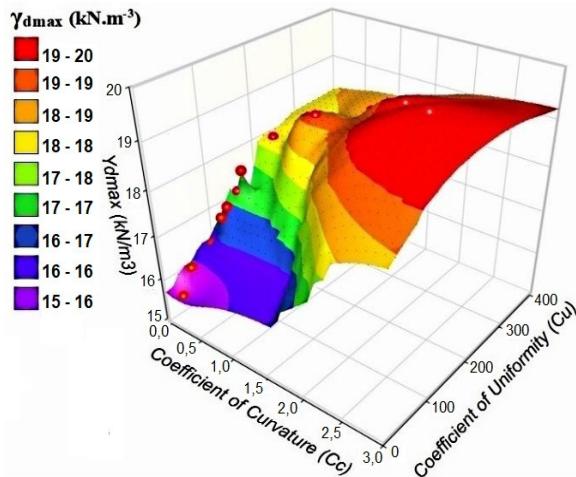


Figure 6.39: 3D response surface of γ_{dmax} dependence on C_c and C_u

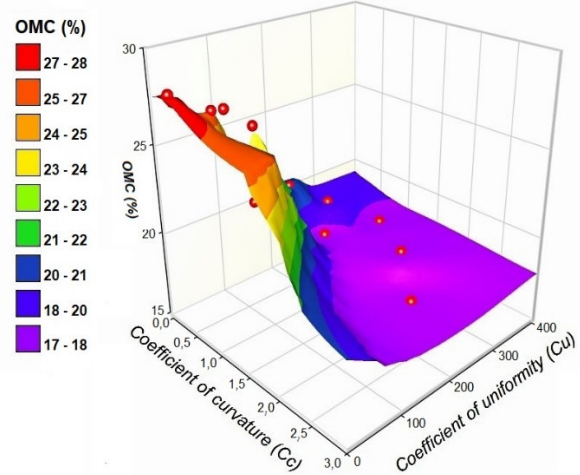


Figure 6.40: 3D response surface of OMC dependence on C_c and C_u

6.4 Soil suction test outcomes

Soil suction is an essential property for studying the behaviour of compacted unsaturated expansive soil. The assessment of soil suction was conducted using the Whatman No. 42 type filter paper technique (FPT) following ASTM D5298-16. The principle of FPT was to measure suction indirectly by relating the water absorbed by specified filter papers with suction through a calibration curve. Several calibration curves for Whatman No. 42 filter paper (FP) were available in the literature. However, it was advisable to develop a calibration curve for a study involving different filter paper lots (Deka *et al.* 1995). The calibration curves, Equations 5.16 and 5.17 in Chapter 5, were established through a calibration process using a salt solution. Total and matric suction values were evaluated on soil specimens, and osmotic suction values were measured as the variation between total and matric suction. Additionally, the response of soil suctions to water content was investigated for each tested soil specimen. The relationship between soil suction and other soil properties was assessed and presented graphically. Tables 6.12 and 6.13 in Appendices 9 and 10 each show the suction test data. The tested soils exhibit a matric suction ranging from 80 kPa to 8745 kPa. These findings were consistent with studies by Rao and Revanasiddappa (2000), Ikechukwu (2019), Fondjo *et al.* (2021a), and Fondjo *et al.* (2023a).

6.4.1 Soil suction response to water content for BFS and WIS

Figures 6.41 through 6.43 illustrate the total suction (ψ_t), matric suction (ψ_m), and osmotic suction (ψ_o) response to water content for soil samples BFS 1, 2, and 3,

respectively. The soil suction in the compacted soils decreased as the water content increased across the entire range of water content. At OMC, the contribution of ψ_m to ψ_t was about 65 %, and the contribution of ψ_o to ψ_t was about 35 % for BFS. However, for higher suctions, the contribution of ψ_m to ψ_t was approximately 81 %, and the contribution of ψ_o to ψ_t was about 19 % for BFS. The ψ_m and ψ_t curves shared a very similar in shape, particularly within the higher moisture content range. The changes in matric suction were more significant than the changes in osmotic suction. This finding was consistent with reports published by Nelson and Miller (1992), Fredlund and Rahardjo (1993), Leong *et al.* (2003), Murray and Sivakumar (2010) on soil moisture deficiency.

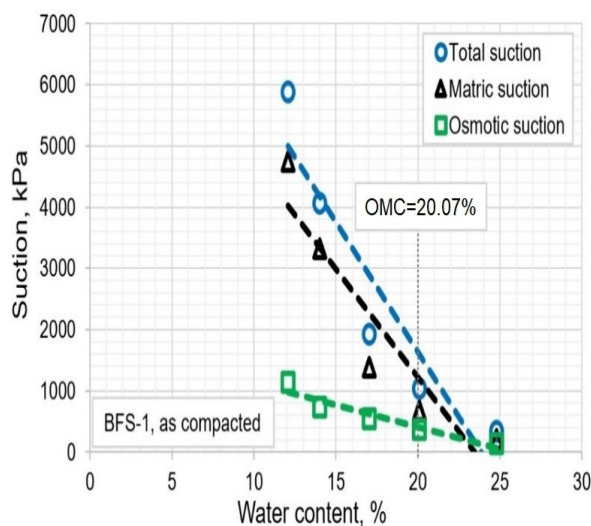


Figure 6.41: Suctions response to water content for BFS-1

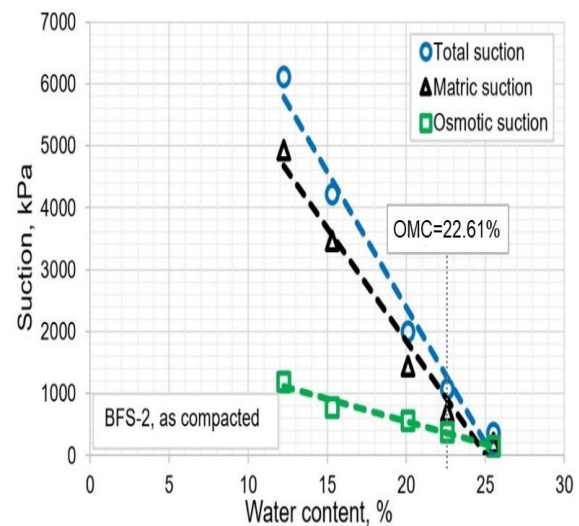


Figure 6.42: Suctions response to water content for BFS-2

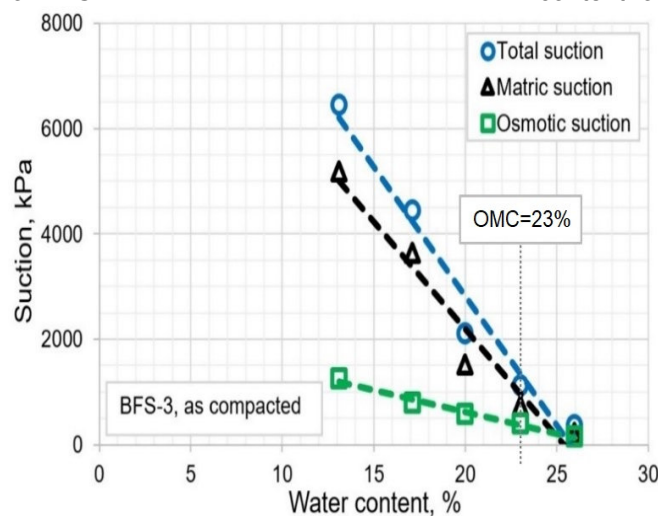


Figure 6.43: Suctions response to water content for BFS-3

Figures 6.44 and 6.45 illustrate the responses of ψ_m and ψ_t to water content for the soil samples BFS. Among the BFS samples, BFS-3 demonstrated the highest ψ_m and ψ_t values, followed by BFS-2 and BFS-1. In the case of compacted expansive soils, both ψ_m and ψ_t increased as the fine-grained content in the soil material increased and water content decreased. These results align with the study published by Fredlund and Xing (1994), which indicated that soils with a higher fraction of clay exhibited greater soil suction for the same water content compared to soils with a smaller fraction of clay.

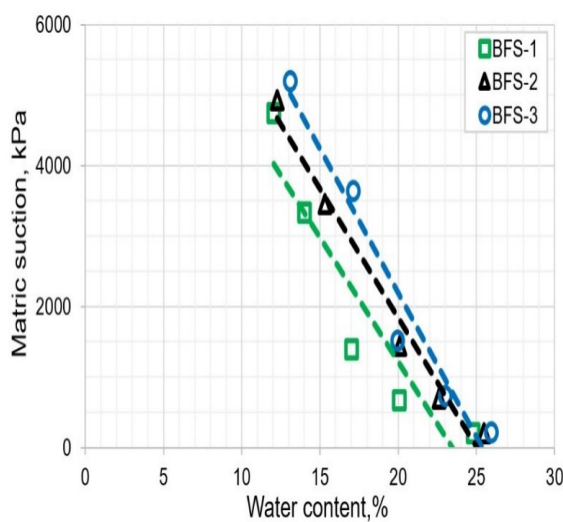


Figure 6.44: Matrix suctions response to water content for BFS

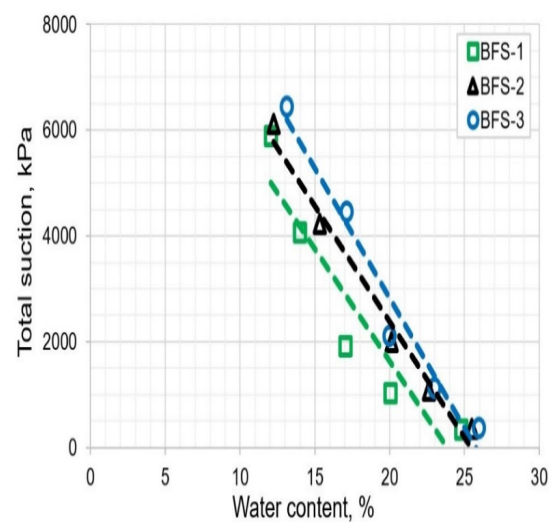


Figure 6.45: Total suction response to water content for BFS

Figures 6.46 through 6.48 illustrate the responses of total suction (ψ_t), matric suction (ψ_m), and osmotic suction (ψ_o) to water content for soil samples WIS 1, 2, and 3 respectively. At OMC, the contribution of ψ_m to ψ_t was 71.59 %, and the contribution of ψ_o to ψ_t was 28.41 %. At higher suction values, the ψ_m contributed to ψ_t by 80.45 %, while ψ_o contributed by 19.55 %. The changes in total suction was primarily influenced by variations in matric suction. This trend was consistent with another study published by Krahn and Fredlund (1972), which indicated that osmotic suction variations were minor in fine-grained soil across the range of water content. Leong *et al.* (2003) also reported that matric suction variations showed a more significant impact than osmotic suction variations. Additionally, the differences in particle size distribution, shape, and pore size contributed to the reported discrepancies in suction

values for specimens WIS 1, 2, and 3 (Fredlund and Rahardjo, 1993; Houston *et al.* 1994).

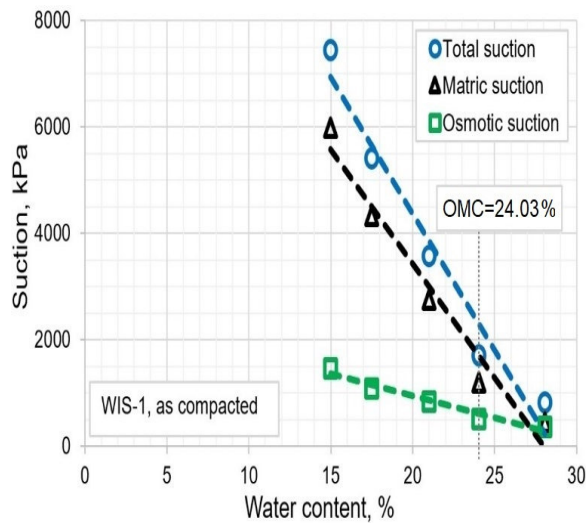


Figure 6.46: Suctions response to water content for WIS-1

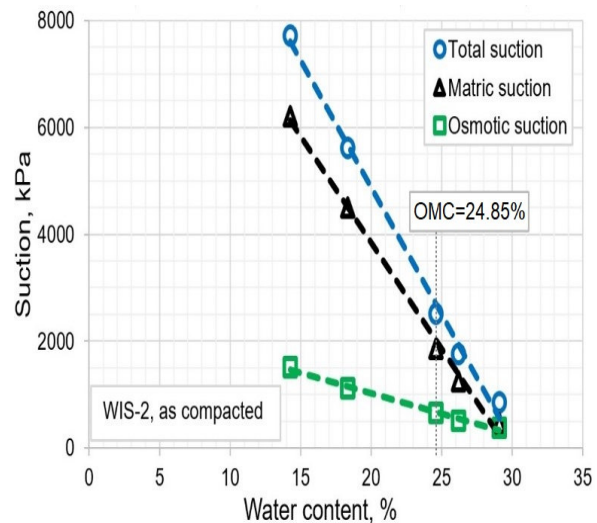


Figure 6.47: Suctions response to water content for WIS-2

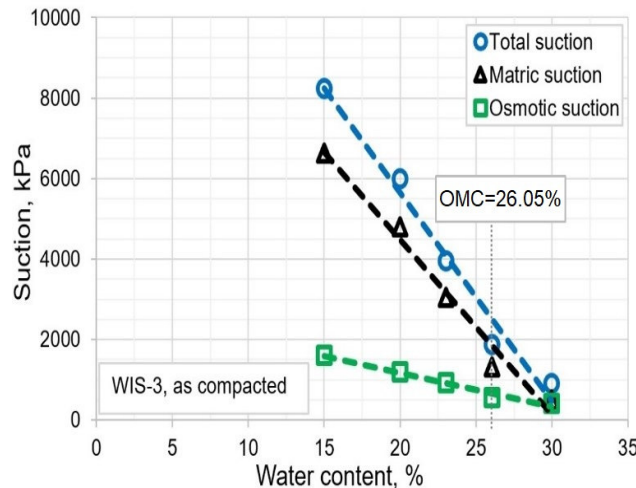


Figure 6.48: Suctions response to water content for WIS-3

Figures 6.49 and 6.50 illustrate the responses of ψ_m and ψ_t to water content for soil samples WIS 1, 2, and 3. Among the soil specimens, WIS-3 exhibited the highest values of ψ_m and ψ_t , followed by WIS-2 and WIS-1. WIS-3 portrayed a higher clay content compared to WIS-2 and WIS-1. Soil suction increased when the clay content increased at the same moisture content. These findings align with the study reported by Fredlund and Xing (1994).

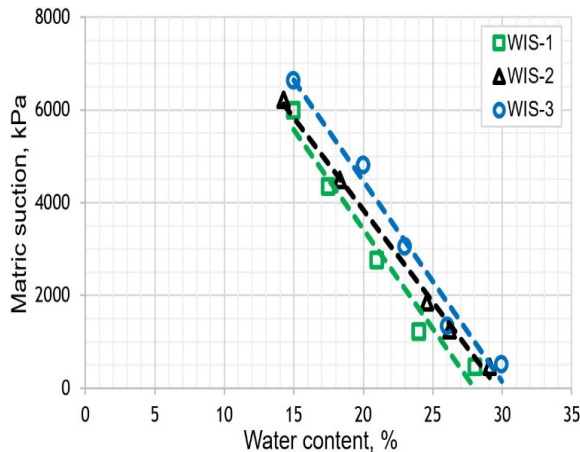


Figure 6.49: Matrix suctions response to water content for WIS

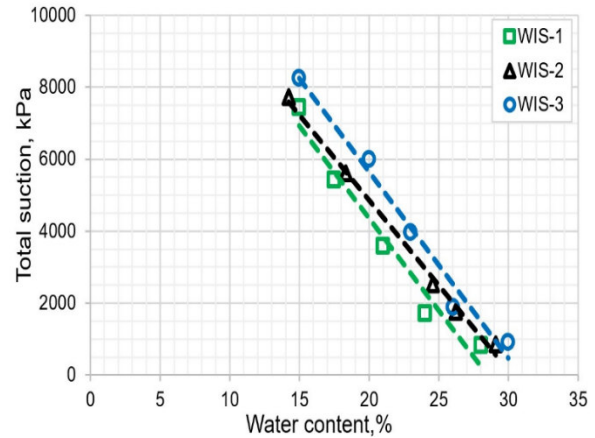


Figure 6.50: Total suctions response to water content for WIS

6.4.2 Soil suction response to water content for WES and BES

Figures 6.51 through 6.53 depict the total suction (ψ_t), matric suction (ψ_m), and osmotic suction (ψ_o) responses to water content for soil samples WES 1, 2, and 3, respectively. The recorded soil suction values of WES 1, 2, and 3 decreased as the compaction water content increased across the entire range of water content. This trend can be explained as follows: At higher water content, the pores between the soil particles were nearly filled with water, causing the air-water interface to be relatively flat. In contrast, reducing the water content led to a reduction in the radius of the meniscus, increasing the suction in the soil. This trend aligned with the findings published by Khrahn and Fredlund (1972), and Elgabru (2013), which indicated a decrease in soil suction with increasing water content. At OMC, the ψ_m contribution to ψ_t was 71.85 %, while the ψ_o contribution to ψ_t was 28.15 %. However, for higher suction values, the ψ_m contribution to ψ_t increased to around 78 %, while the ψ_o contribution to ψ_t was approximately 22 %.

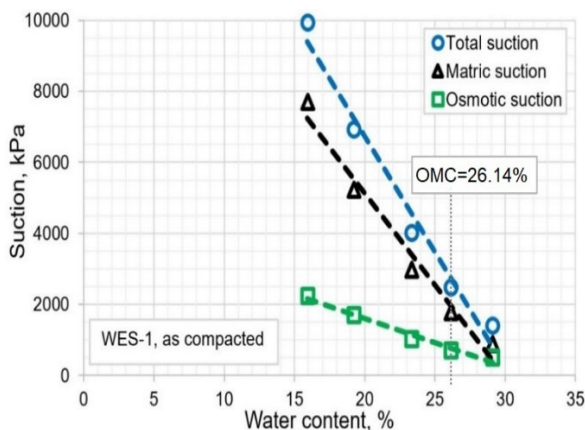


Figure 6.51: Suctions response to water content for WES-1

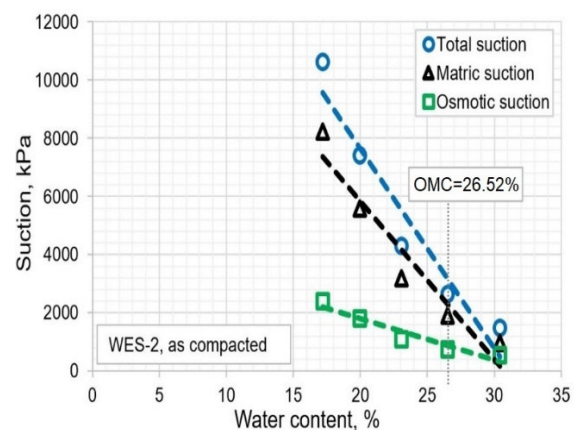


Figure 6.52: Suctions response to water content for WES-2

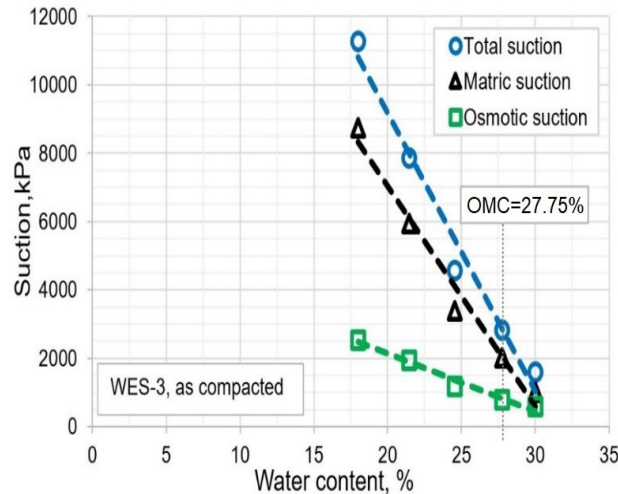


Figure 6.53: Suctions response to water content for WES-3

Figures 6.54 and 6.55 depict the ψ_m and ψ_t responses to water content for samples WES 1, 2, 3. Among these samples, WES-3 exhibited the highest ψ_m and ψ_t values, followed by WES-2 and WES-1. WES-3 exhibited a higher smectite content compared to WES-2 and WES-1. Based on these observations, it can be inferred that ψ_m and ψ_t increased when the smectite content in the soil material increased and water content reduced. Therefore, the soil's mineralogy played a significant role in influencing the relationship between suction and water content. This trend aligned with other research studies conducted by Heshmati and Motahari (2012), and Song and Hong (2020), which highlight the impact of clay minerals on the suction-water relationship in unsaturated soils.

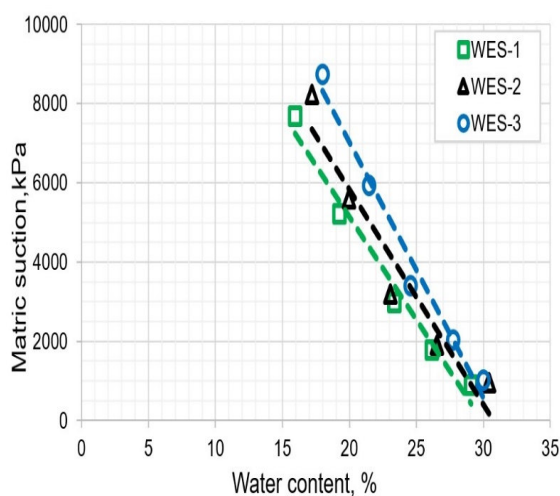


Figure 6.54: Matric suction response to water content for WES

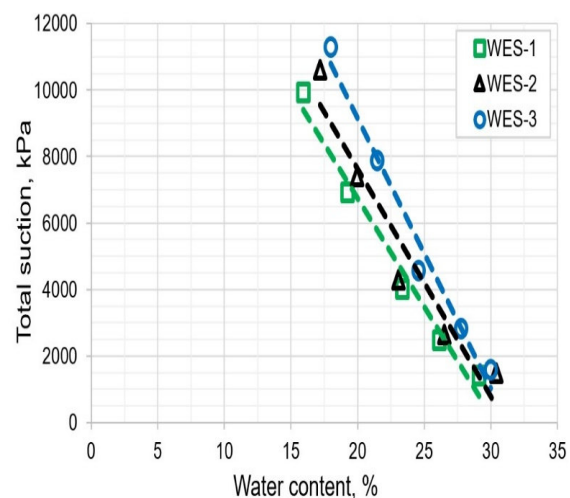


Figure 6.55: Total suction response to water content for WES

Figures 6.56 through 6.58 portray the soil suction response to water content for soil samples BES 1, 2, and 3, respectively. At OMC, the contribution of ψ_m to ψ_t was 68.65 %, and the contribution of ψ_o to ψ_t was 31.53 %. However, for higher suction values, the contribution of ψ_m to ψ_t was around 80 %, and the contribution of ψ_o to ψ_t was approximately 20 %. These results aligned with studies published by Fredlund and Rahardjo (1993), Murray & Sivakumar (2010), and Fondjo *et al.* (2021), which indicated that osmotic suction changes were generally less significant than matric suction variations.

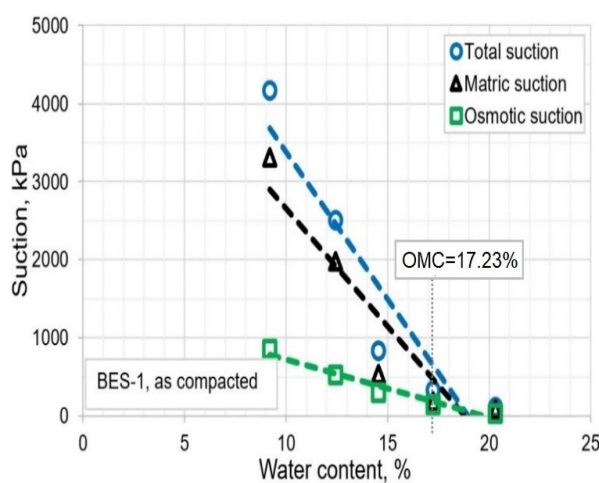


Figure 6.56: Suctions response to water content for BES-1

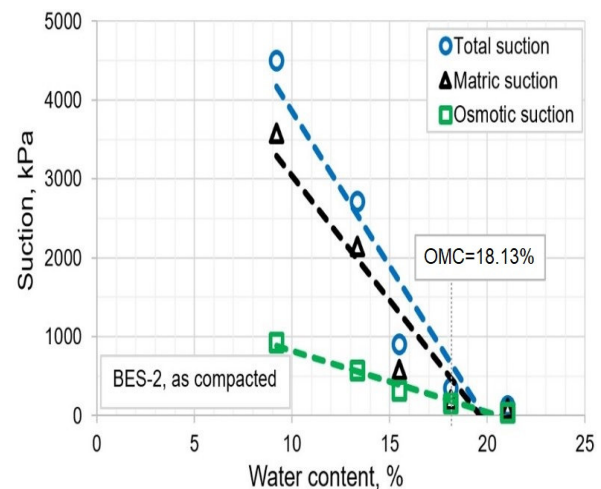


Figure 6.57: Suctions response to water content for BES-2

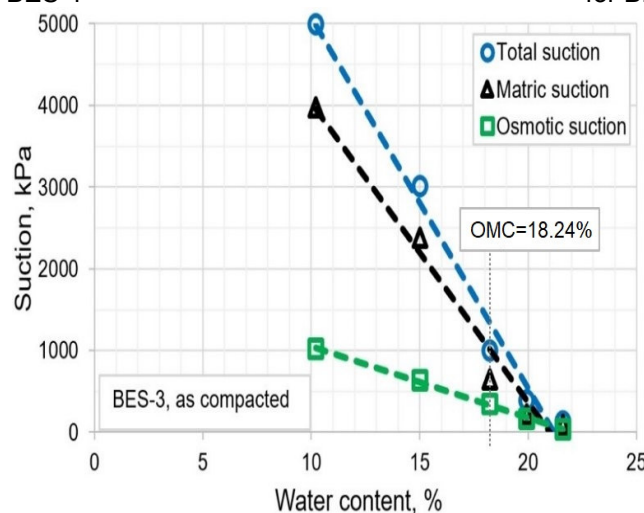


Figure 6.58: Suctions response to water content for BES-3

Figures 6.59 and 6.60 describe the ψ_m and ψ_t responses to water content in samples BES 1, 2, and 3. Soil specimen BES-3 portrayed a higher magnitude of ψ_m and ψ_t , followed by BES-2 and BES-1. BES-3 displayed a higher liquid limit values,

followed by BES-2 and BES-1. This trend indicated that soil moisture deficiency increased with the increase of liquid limit. The results aligned with the study published by Suriya *et al.* (2015) that showed the influence of the liquid limit on the soil suction-water relationship.

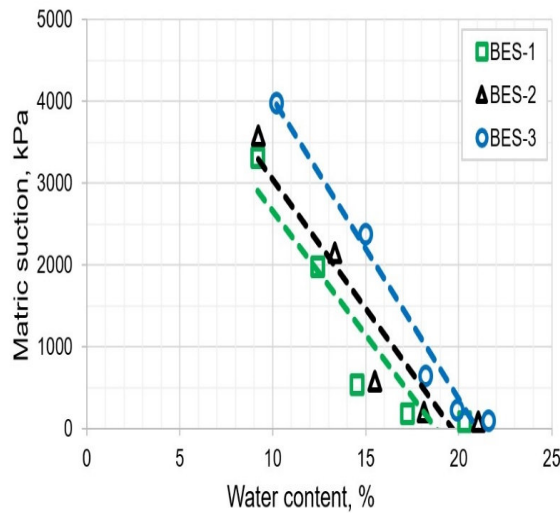


Figure 6.59: Matrix suction response to water content for BES

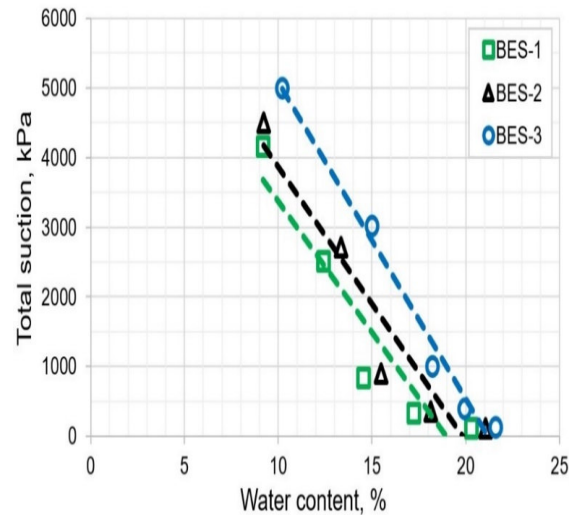


Figure 6.60: Total suction response to water content for BES

6.4.3 Analysis of correlation between matric suction, clay content, liquid limit, specific gravity, and free swell index

Figure 6.61 and Figure 6.62 describe the relationship between matric suction and soil parameters: clay content, liquid limit (LL), specific gravity (Gs), and free swell index (FSI). Figure 6.61 shows the surface plot of ψ_m , clay content, and LL. The ψ_m increased with the increase of clay content and displayed a strong correlation described in Equation 6.35, Appendix 11. The clay content influenced the matric suction values. This result was consistent with the report published by Fredlund and Rahardjo (1993), and Houston *et al.* (1994) on the influence of the particle size distribution on soil suction. ψ_m increased with the increase of LL and displayed a strong correlation given in Equation 6.36, Appendix 11. The LL impacted the magnitude of soil moisture deficiency. This result aligned with the report published by Suriya *et al.* (2015) that showed the influence of liquid limit on soil suction-water relationship.

Figure 6.62 shows a surface plot of ψ_m , Gs, and FSI. The ψ_m increased with the increase of Gs and portrayed a strong correlation given in linear Equation 6.37,

Appendix 11. ψ_m increased when the FSI increased and portrayed a strong correlation described in linear Equation 6.38, Appendix 11. Based on the current observations, Gs and FSI influenced the magnitude of matric suction of compacted expansive soils.

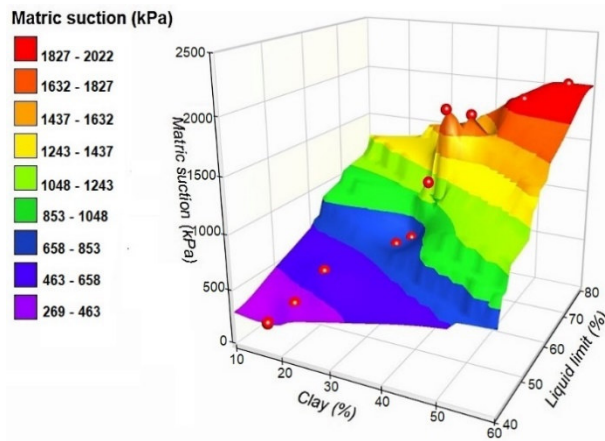


Figure 6.61: 3D response surface of matric suction dependence on clay (%) and LL (%)

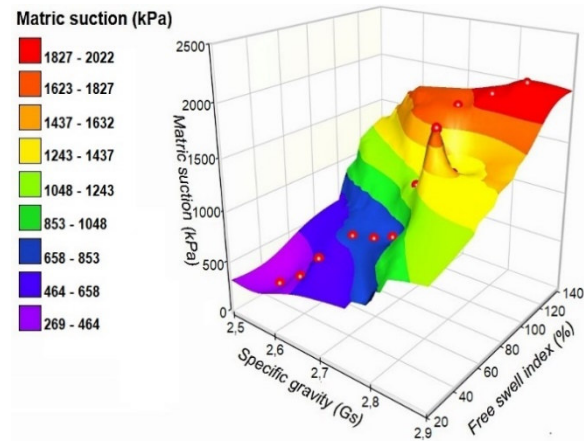


Figure 6.62: 3D response surface of matric suction dependence on Gs and FSI (%)

6.4.4 Analysis of correlation between matric suction, linear shrinkage, void ratio, free swell ratio, and dry unit weight

Figures 6.63 and 6.64 portray a three-dimensional description of the correlation between ψ_m and soil features such as LS, void ratio, free swell ratio, and γ_d . The colour patterns showed different matric suction values. Figure 6.63 shows a surface plot of ψ_m , LS, and void ratio. ψ_m decreased as LS increased and displayed a strong correlation described in linear Equation 6.39, Appendix 11. Additionally, soil moisture deficiency increased with the increment of void ratio and showed a strong correlation described in Equation 6.40, Appendix 11. Linear shrinkage and void ratio influenced the magnitude of soil moisture deficiency in compacted expansive soils.

Figure 6.64 presents a surface plot of ψ_m , FSR, and γ_d . Matric suction increased as FSR increased, showing a strong correlation between ψ_m and FSR described in Equation 6.41, Appendix 11. ψ_m decreased when γ_d increased, displaying a strong correlation in Equation 6.42, Appendix 11. This result contradicted the study reported by Wan *et al.* (1995), which showed the insignificant effect of γ_d on ψ_m . However, these results were consistent with the research published by Kim *et al.* (2019), which demonstrated a direct relationship between ψ_m and γ_d at the OMC. Figures 6.63 and

6.64 exhibited some local extremes, originating from discrepancies in correlations. Both FSR and γ_d influenced the value of matric suction in compacted expansive soils.

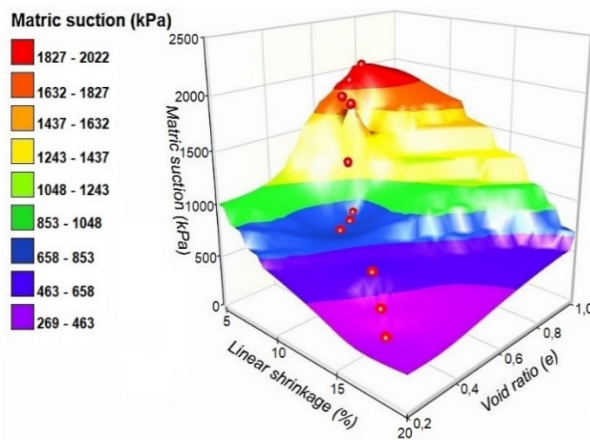


Figure 6.63: 3D response surface of matric suction dependence on LS (%) and void ratio

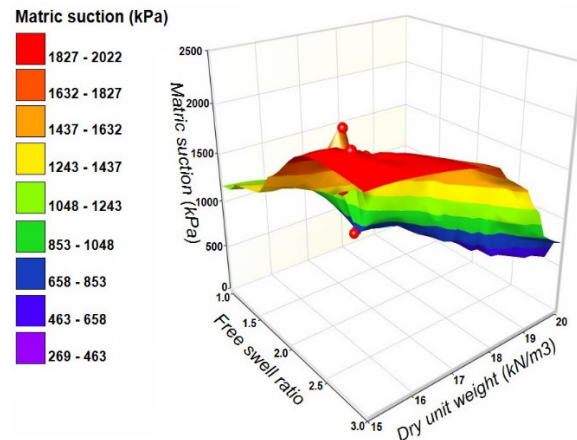


Figure 6.64: 3D response surface of matric suction dependence on FSR and γ_d

6.5 Analysis of soil-water characteristic curves (SWCC)

Section 5.10 in Chapter 5 outlined the procedures for modelling the soil-water characteristic curve (SWCC). Figures 6.65 through 6.76 present the SWCCs of soil samples BFS 1, 2, 3; WIS 1, 2, 3; WES 1, 2, 3; and BES 1, 2, 3. The Model (BL) developed by Seki (2007) and the (DB) model developed by Durner (1994) yielded better-fitting suction estimation values than the (FX) model by Fredlund and Xing (1994), the (VG) model by Van Genuchten (1980), and the (LN) model by Kosugi (1996). This observation stemmed from the bimodal pore structures of the soil samples in this study. For soil specimens with bimodal pore structures, the bimodal log-normal distribution model (BL) and (DB) model showed similar fitting performance. However, models (FX, VG, LN) exhibited the best-fit performance for unimodal pore structures. Air entry values (AEV) for the twelve samples were determined graphically on the SWCCs in Figures 6.65 through 6.76. Table 6.15 presents the air entry values (AEV) for soil samples, ranging from 6 kPa to 20 kPa. These results aligned with the findings of Fredlund and Xing (1994) and Fondjo *et al.* (2021b). WES displayed a higher value of matric suction at the air entry value (AEV), followed by WIS, BFS, and BES. This difference can be attributed to the non-uniformity of the fine-grained content in the soil samples. WES depicted a higher clay content, followed by WIS, BFS, and BES. Vanapalli *et al.* (1999) reported that fine-grained materials such as silt and clay have smaller pores and a larger relative surface area, leading to slower desaturation rates.

Additionally, an increment in compaction effort resulted in a decrease in void ratio and an increase in dry unit weight, leading to the variations in the SWCCs of the tested soil material. Compaction effort, fine-grained content, and water content influenced the properties of SWCC. Tables 6.16 through 6.19 in Appendix 12, 13, 14, and 15 present the fitting parameters of models (VG) Van Genuchten (1980), (DB) Durner (1994), (FX) Fredlund and Xing (1994), (LN) Kosugi (1996), and (BL) Seki (2007) to describe the best-fitting SWCCs of BFS, WIS, WES, and BES. The coefficient of determination (R-squared) for the fitting-parameter models (BL, DB) were higher than that of the models (FX, VG, and LN).

Table 6.15: Air Entry Value of twelve samples

Soils	Air Entry Value (AEV)	
	Matric Suction $\psi_{m(AEV)}$, kPa	Volumetric water content θ_{AEV}
BFS-1	10	0.443
BFS-2	11.00	0.458
BFS-3	12.38	0.482
WIS-1	14.82	0.443
WIS-2	15.00	0.462
WIS-3	17	0.490
WES-1	16.13	0.526
WES-2	19.20	0.557
WES-3	20.20	0.588
BES-1	6.00	0.274
BES-2	7.01	0.319
BES-3	7.23	0.338

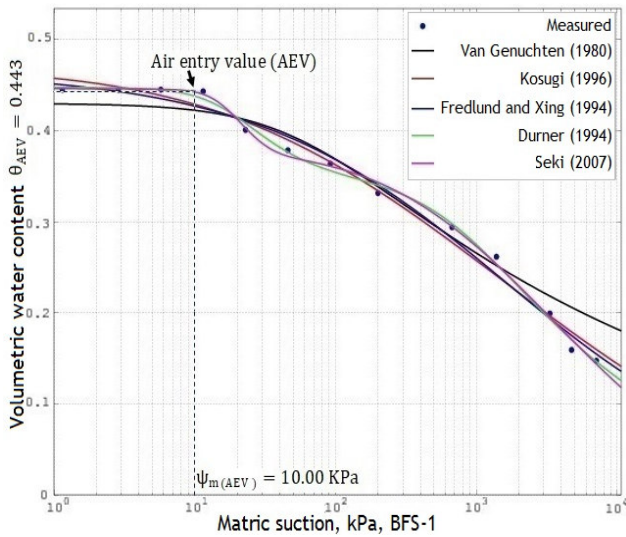


Figure 6.65: Soil-water characteristic curve for BFS-1 as compacted

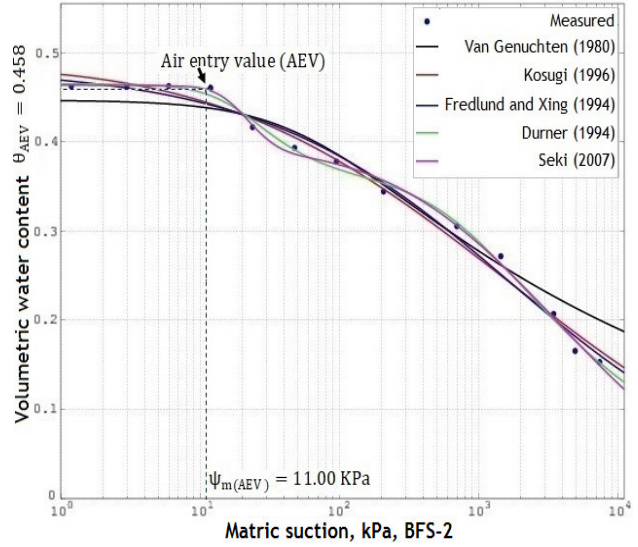


Figure 6.66: Soil-water characteristic curve for BFS-2 as compacted

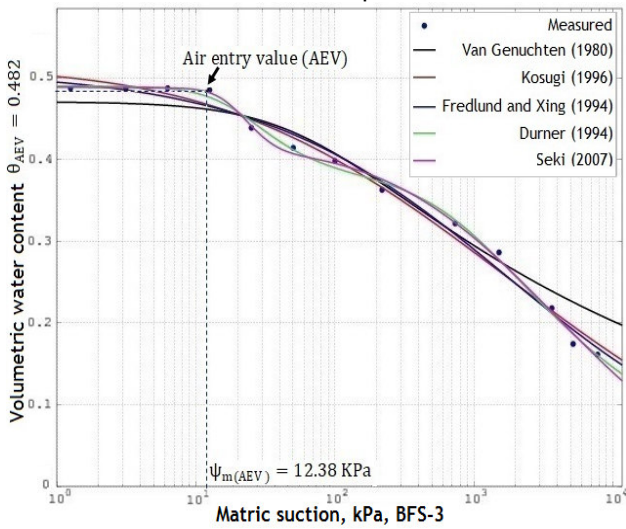


Figure 6.67: Soil-water characteristic curve for BFS-3 as compacted

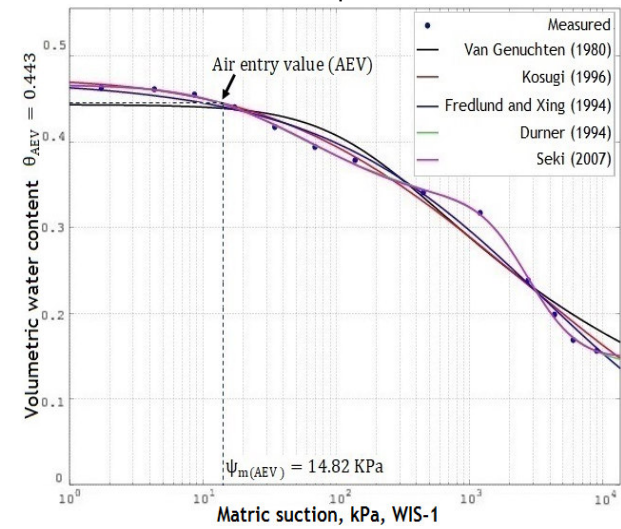


Figure 6.68: Soil-water characteristic curve for WIS-1 as compacted

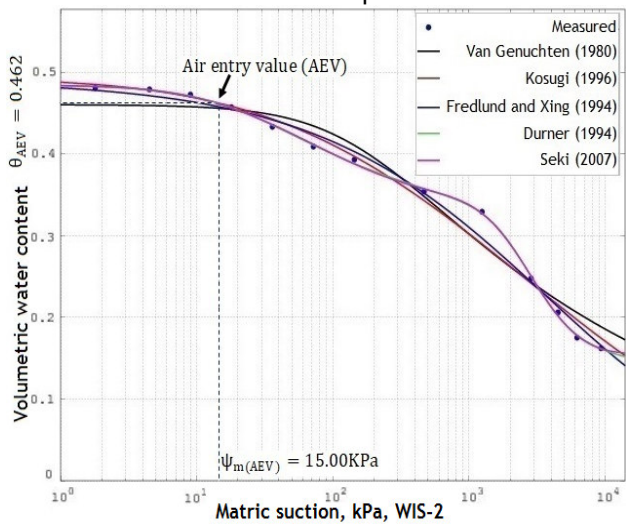


Figure 6.69: Soil-water characteristic curve for WIS-2 as compacted

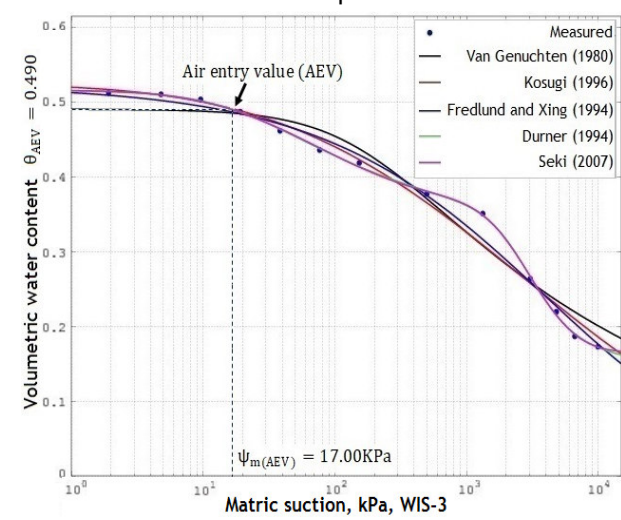


Figure 6.70: Soil-water characteristic curve for WIS-3 as compacted

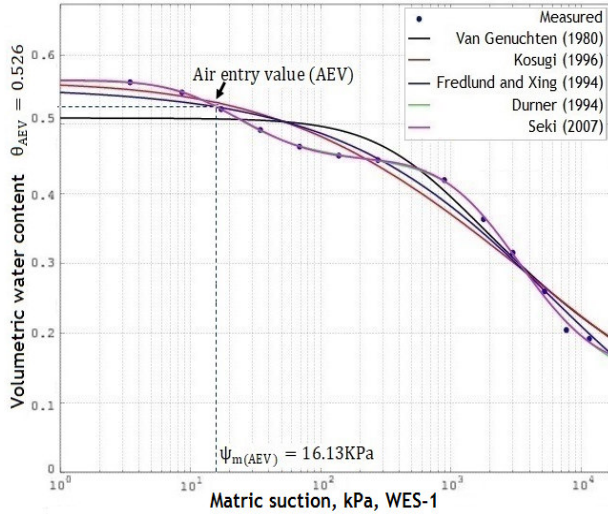


Figure 6.71: Soil-water characteristic curve for WES-1 as compacted

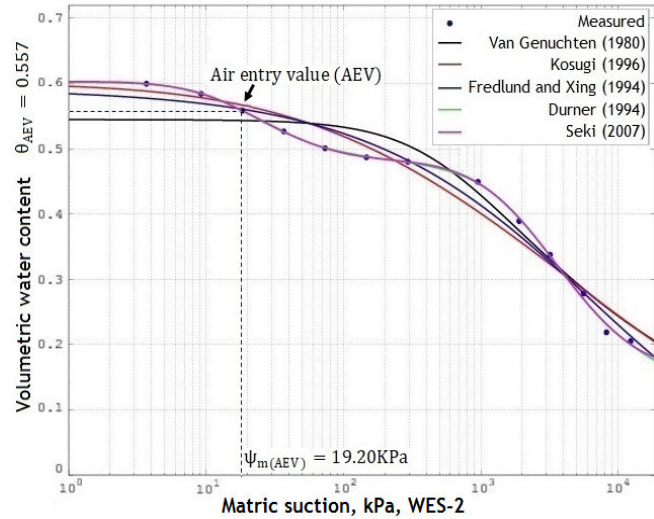


Figure 6.72: Soil-water characteristic curve for WES-2 as compacted

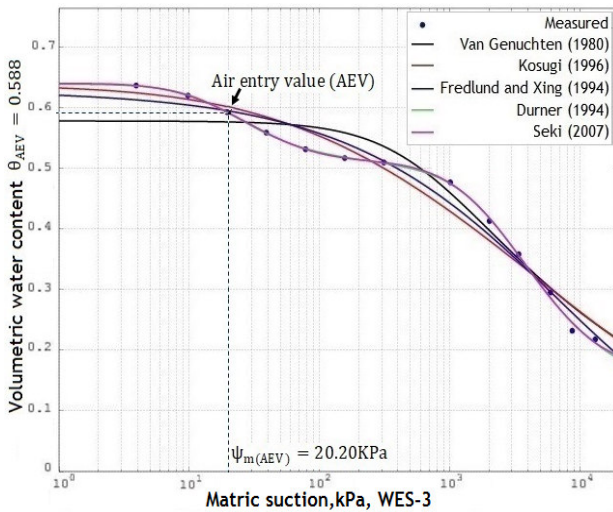


Figure 6.73: Soil-water characteristic curve for WES-3 as compacted

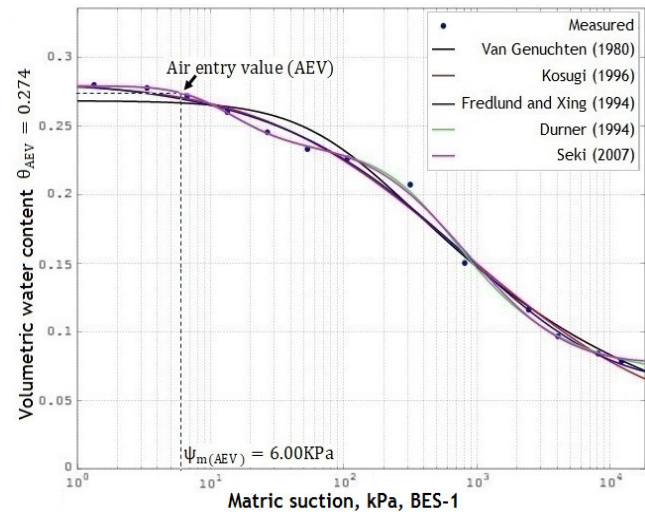


Figure 6.74: Soil-water characteristic curve for BES-1 as compacted

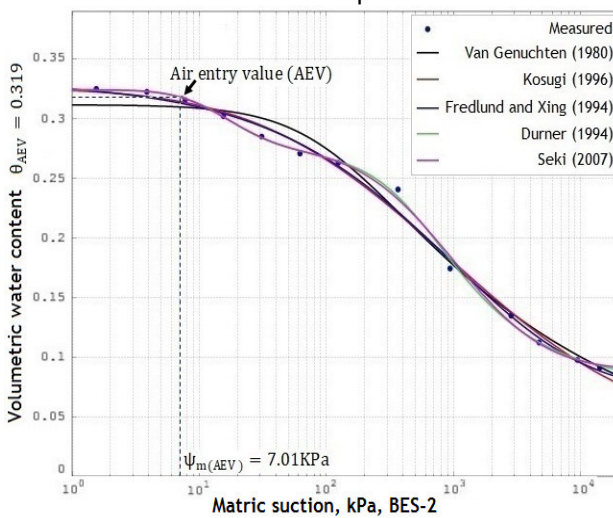


Figure 6.75: Soil-water characteristic curve for BES-2 as compacted

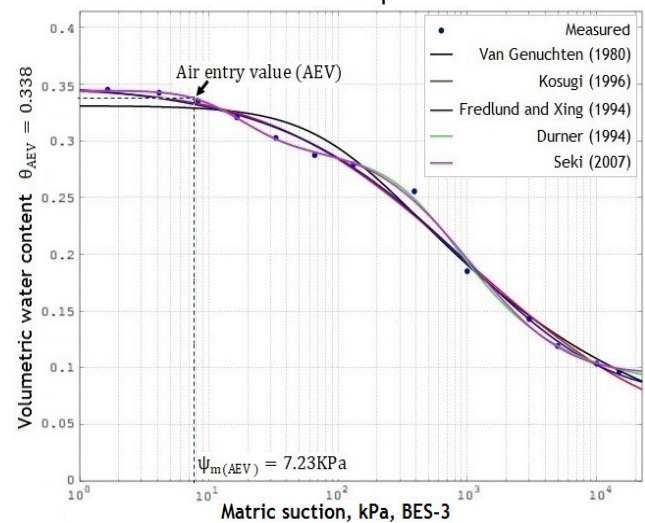


Figure 6.76: Soil-water characteristic curve for BES-3 as compacted

6.6 Swelling stress test results analysis

Tables 6.20 and 6.21 in Appendices 16 and 17 show the zero swelling test results. The reported swelling stress (P_s) values range from 107 kPa to 827 kPa. This result agrees with the study published by Ikechukwu (2019). The P_s developed in these soils originates from the significant presence of clay mineral (smectite) described in paragraph 6.2. The range of P_s is not uniform for the tested soils. WES displays the higher values of P_s , followed by WIS, BFS, and BES. The different amounts of smectite can justify these results. WES exhibits the higher smectite content, followed by WIS, BFS, and BES.

6.6.1 Swelling stress-water content relationship

Figure 6.77 describes the swelling stress (P_s) response to the water content obtained from 60 soil specimens. The P_s shows a tendency to increase as the water content increases. The P_s portrays a good exponential correlation with the water content, and the estimated determination coefficient is 85.3 %. The results align with the outcome of the investigation by Basma *et al.* (1995), Komine and Ogata (1994), and Attom and Barakat (2000), which reported that the P_s increases with an increment in water content. Rank *et al.* (2018) conducted an experimental investigation on swelling soils collected from various locations across India. The results show that the P_s increases when the water content increases.

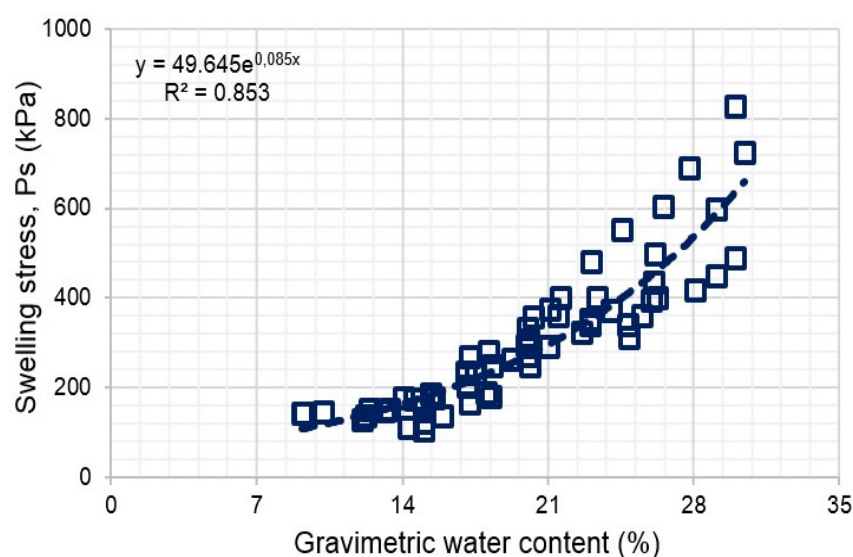


Figure 6.77: Swelling stress vs water content

6.7 Shear strength characteristics of twelve samples

6.7.1 Overview

This section describes the shear strength properties of tested soils. The stress-strain graph illustrates the compressive stress-strain response of the soil samples at failure for the confining pressures (σ_c) of 50 kPa, 100 kPa, and 150 kPa. The Mohr-Coulomb failure envelope portrays the saturated shear strength (τ_s) features of soil samples at different water content. The unsaturated shear strength (τ_u) parameters of the studied soils are determined based on the contribution of matric suction to the saturated shear strength at different water content. This section investigates the shear strength parameter's response to geotechnical index properties and soil mineralogy. A modified unsaturated shear strength (τ_{mu}) constitutive relation for compacted unsaturated expansive soils is developed based on the contribution of the matric suction and the swelling stress to the saturated shear strength of the tested soils. The modified-unsaturated shear strength is assessed based on equations established by Fredlund and Rahardjo (1993), Garven and Vanapalli (2006), and Bulolo and Leong (2019) found in Tables 3.2 and 3.3. Lastly, section 6.8 reports the design of the geotechnical predictive models (GPM).

6.7.2 Compressive stress-strain behaviour of twelve samples

In soil mechanics, the stress-strain graph describes the behaviour of soil materials. Stress refer to the rate of applied load on the cross-sectional surface of the soil specimen, while strain represents the rate of variation in dimension to the original dimensions of the soil specimen. A stress-strain graph serves as a practical tool used for understanding and predicting the response of soil materials under different loading conditions. For this research, the stress-strain behaviour of the studied soils was assessed using consolidated undrained tests. The soil specimens were subjected to vertical loading, deforming at a constant rate of 2 mm per minute of vertical displacement until shear failure occurred. A linear displacement transducer recorded the compressive strain of the soil specimens. Additionally, the stress-strain characteristics at failure of the tested soils were measured at different water content under confining pressures (σ_c) of 50 kPa, 100 kPa, and 150 kPa. Figures 6.78 to 6.86 in Appendices 18 to 26 show the stress-strain graphs of the soil samples. Table 6.21a outlines the stress-strain graphs of soil samples.

Table 6.21a: Outline of the stress-strain graphs of twelve samples

Samples	Confining pressures (kPa)	Figures No.	Appendix No.
BFS 1, 2, 3	50, 100, 150	6.78 to 6.86	18 to 20
WIS 1, 2, 3	50, 100, 150	6.87 to 6.95	20 to 22
WES 1, 2, 3	50, 100, 150	6.96 to 6.104	22 to 24
BES 1, 2, 3	50, 100, 150	6.105 to 6.113	24 to 26

The reported strain at failure for BFS is within the range of 0.043 to 0.05 at 50 kPa, 0.046 to 0.05 at 100 kPa, and 0.053 to 0.06 at 150 kPa. The recorded strain at failure for WES is within the range of 0.058 to 0.07 at 50 kPa, 0.061 to 0.07 at 100 kPa, and 0.116 to 0.14 at 150 kPa. The stress-strain graphs display the strain at failure of other soil samples (WIS, BES). The peak load reduces when the water content increases and the cohesion within the soil particles decreases. The peak load increases when the confining pressure increases at constant water content and can be explained as follows: At constant water content, the variation of confining pressure significantly influences the peak load, and the soil material takes on more stress for lower strain at the dry side. The stress-strain graph is much steeper, the elastic modulus is higher, and brittle failure occurs in this case. Additionally, at the wet side of optimum, the soil material produces more stress even for lower strain, resulting in a flatter stress-strain graph, and ductile failure occurs at higher strain values with a lower elastic modulus. The results are consistent with the studies reported by Fang and Ying (2006) and Chowdhury (2013). All tested soils exhibit a similar trend. The stress progressively increases as the strain increases to the ultimate compressive stress. Beyond the failure strain, the soil stress decreases rapidly. The behaviour of all tested soils can be justified as follows: Soil compacted to the dry optimum has a flocculated arrangement due to the attraction between soil particles resulting from low water content. In contrast, soil compacted to the wet side of optimum displays a scattered arrangement due to repulsive forces between soil particles at high water content. The interaction between the fine fraction in soil and water induces free water energy that influences stress-strain behaviour.

6.7.3 Saturated shear strength (τ_s) results

Shear strength is a fundamental soil parameter in soil mechanics. The reliability of geotechnical structures depends on the shear strength of the soil support. Examples

of geotechnical applications of shear strength of soil material include bearing capacity, slope stability, and more. Soil foundation fails under shearing conditions, including punching shear failure, local shear failure, and general shear failure. The shear strength of soil refers to the maximum shear resistance that the soil material can withstand. When this value is exceeded, the soil material is considered to have failed. The friction angle (ϕ') represents the resistance to interlocking of the soil particles, while cohesion (c') refers to the force that binds soil particles together. Saturated shear strength parameters ϕ' and c' are determined using triaxial or direct shear apparatus. Consolidated undrained triaxial (CU) tests were conducted to measure the shear strength of prepared soil specimens. Triaxial tests were performed on prepared soil specimens (BFS, WIS, WES, BES) at different water content. The Mohr-Coulomb failure envelope at each water content was recorded for each soil sample and plotted on a graph. Brenner *et al.* (1997) noted that soil specimens experiences three stresses: the vertical axial stress represents the major stress, while the intermediate and minor stresses are equal in magnitude and correspond to cell pressure. Fredlund *et al.* (2012) reported that undrained tests on unsaturated soils can be conducted using conventional triaxial cells. Figures 6.114 to 6.149 in Appendices 27 to 38 depict the Mohr-Coulomb failure envelope graphs of soil samples at different water content. Table 6.21b outlines the Mohr-Coulomb failure envelope graphs of the twelve soil samples.

Table 6.21b: Outline of Mohr-Coulomb failure envelope for twelve samples

Sample	Figure No.	Appendix No.
BFS 1, 2, 3	6.114 to 6.122	27 to 29
WIS 1, 2, 3	6.123 to 6.131	30 to 32
WES 1, 2, 3	6.132 to 6.140	33 to 35
BES 1, 2, 3	6.141 to 6.149	36 to 38

The cohesion (c') and friction angle (ϕ') are determined from the Mohr-Coulomb failure envelope (MCFE) for each soil sample. The reported results reveal that for BFS 1, 2, and 3, c' ranges from 46 kPa to 75 kPa, and ϕ' ranges from 25.14° to 52.09°. For WIS 1, 2, and 3, c' ranges from 36 kPa to 69 kPa, and ϕ' ranges from 24.58° to 50.75°. For WES 1, 2, and 3, c' ranges from 35 kPa to 62 kPa, and ϕ' ranges from

22.24° to 42.07°. Lastly, for BES 1, 2, and 3, c' ranges from 39 kPa to 78 kPa, and ϕ' ranges from 23.92° to 53.21°. Overall, the soil samples exhibit a ϕ' range of 22° to 53° and a c' range of 35 kPa to 78 kPa. The MCFE shown in Figures 6.114 to 6.149 indicates that water content has a decreasing effect on both cohesion and the internal angle of friction. These findings are consistent with studies reported by Casagrande and Hirschfeld (1960), Toll (1990), Stark and Duncan (1992), Kong and Tan (2000), Zewdie (2004), Chen *et al.* (2012), Chowdhury (2013), Wang *et al.* (2014), Fondjo *et al.* (2023a), and Fondjo *et al.* (2024). The decrease in cohesion is attributed to the increased distance between clay particles and water molecules due to attached water. This leads to an increase in the thickness of the hydration membrane, weakening the intermolecular and electrostatic static attraction forces. Similarly, the improvement in lubricity between clay particles results in a reduction in the internal friction angle reduction. Tables 6.22 in Appendix 39 and Table 6.23 in Appendix 40 present the saturated shear strength results.

6.7.4 Unsaturated shear strength (τ_u) results

6.7.4.1 Friction angle (ϕ^b) of shear related to matric suction

The unsaturated shear strength (τ_u) of soil material represents the maximum shear strength that unsaturated soil can withstand. Generally, the equation for unsaturated shear strength is derived from the contribution of matric suction to the equation for saturated shear strength. Lamborn (1986) reported that the equation for unsaturated soil shear strength can be approximated using either a bilinear or linear model. The non-linear equation is often more realistic and provides a more accurate estimation. In this thesis, the equation for saturated shear strength aligned with Equation 3.13 by Terzaghi (1936), while the equation for unsaturated shear strength aligned with Equation 3.30 by Fredlund and Rahardjo (1993). The shear strength at failure and the corresponding soil moisture deficiency of each tested soil specimen, as determined through consolidated undrained tests, were plotted to form the unsaturated soil failure envelope. Additionally, the angle (ϕ^b) of shear strength related to soil moisture deficiency was calculated as the slope between the shear strength and matric suction curves. Figures 6.150 to 6.161 depict the unsaturated failure envelopes for the tested soils BFS, WIS, WES, and BES. All graphs exhibit a similar trend, showing that the shear strength of the investigated soils increases as the soil moisture deficiency

increases within the compacted soil specimens, demonstrating a strong correlation. The behaviour of soil material can be explained as follows: When the soil features a capillary zone, the water-air interface leads to an increase in the compression of the solid matrix, and the matric suction in the soil material causes a reduction in volume and an increase in shear strength of unsaturated soils (Fredlund and Rahardjo, 1993). The outcomes of this research align with other studies by Escario *et al.* (1989) on expansive soil specimens collected across various areas in Spain. Their results reveal that the friction angle and cohesion increase as matric suction increase. Miao *et al.* (2007) reported that the shear strength of expansive soil increases with higher soil suction. Kayadelen *et al.* (2007) emphasised the significant contribution of matric suction to the shear strength of unsaturated soils. Ye *et al.* (2010) noted that the magnitude of unsaturated shear strength increases with suction within controlled range during the test. Çokça and Tilgen (2010) investigated the correlation between soil suction and the shear strength of compacted clay from Ankara at various water contents. The results demonstrate that shear strength magnitude increases with higher soil suction. The graphs exhibit a non-linear relationship between shear strength and matric suction. These findings confirm the studies by Escario *et al.* (1989), Toll (1990), Zhan and Ng (2006), and Kayadelen *et al.* (2007), and Fondjo *et al.* (2023a).

The unsaturated failure envelope of the twelve soil samples provides the angle ϕ^b , which represents the rate of changes in shear strength magnitude with variations in matric suction within the soil material. The results indicate that the angle ϕ^b for BFS 1, 2, and 3 are 6.76°, 6.07°, and 4.78° respectively. For WIS 1, 2, and 3, the values are 5.02°, 3.56°, and 2.49° respectively. Likewise, for WES 1, 2, and 3, the angles are 2.67°, 1.85°, and 1.41° respectively. BES 1, 2, and 3 exhibit angles of 10°, 8.13°, and 7.08° respectively. Among these, BES has the highest ϕ^b values, followed by BFS, WIS, and WES. Discrepancies in ϕ^b values across these soils may stem from variations in clay content within soil material. Soils with lower clay fractions tend to display higher ϕ^b values, while those with higher clay fractions tend to have smaller ϕ^b values. The unsaturated shear strength of the studied soils is influenced by properties such as soil type, different mechanisms of soil particle interaction, suction values, void ratio, clay content, fine fraction, and more. These results

corroborate the findings of Toll (1990), who stated that the relationship between the shear strength and matric suction fundamentally varies with the soil properties. The slope values of the curve (ϕ^b) are smaller than the saturated friction angle (ϕ'). This alignment with research published by Fredlund and Rahardjo (1993), Likos *et al.* (2010), Nam *et al.* (2011), Fondjo *et al.* (2023a), and Fondjo *et al.* (2024) suggests that ϕ^b is smaller than ϕ' , ($\phi^b < \phi'$). Additionally, the recorded values of unsaturated shear strength are 1.1 to 1.78 times higher than the values of saturated shear strength, due to the contribution of matric suction to shear strength at failure. Refer to Table 6.22 in Appendix 39 and Table 6.23 in Appendix 40, which provide the unsaturated shear strength results.

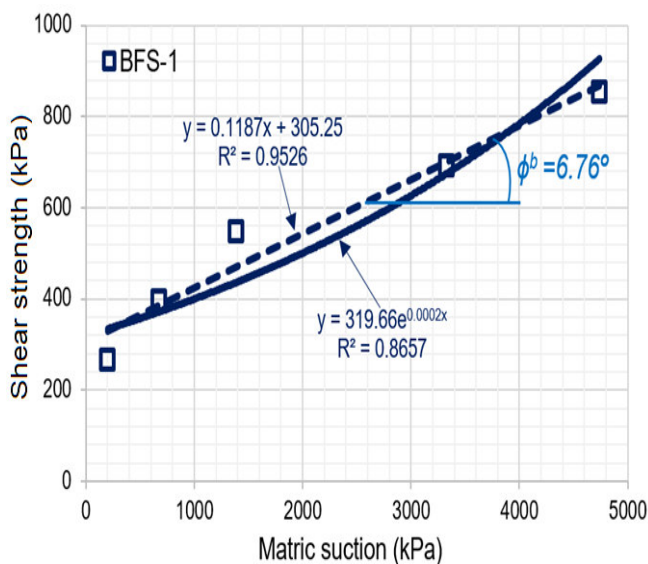


Figure 6.150: Unsaturated failure envelope for BFS-1

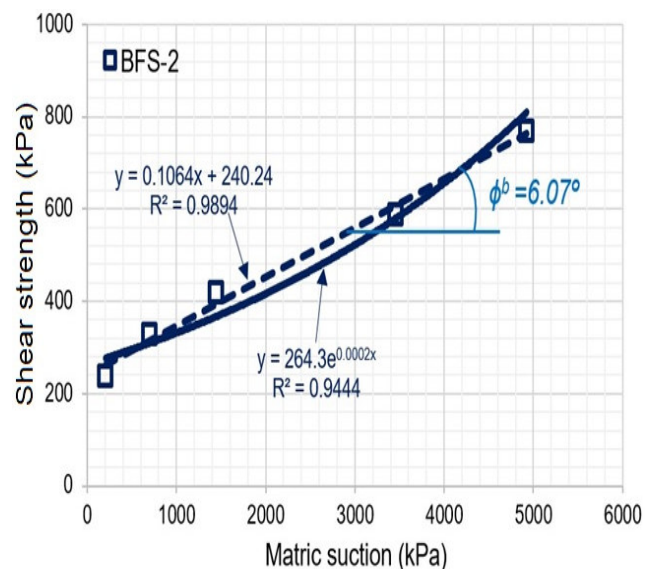


Figure 6.151: Unsaturated failure envelope for BFS-2

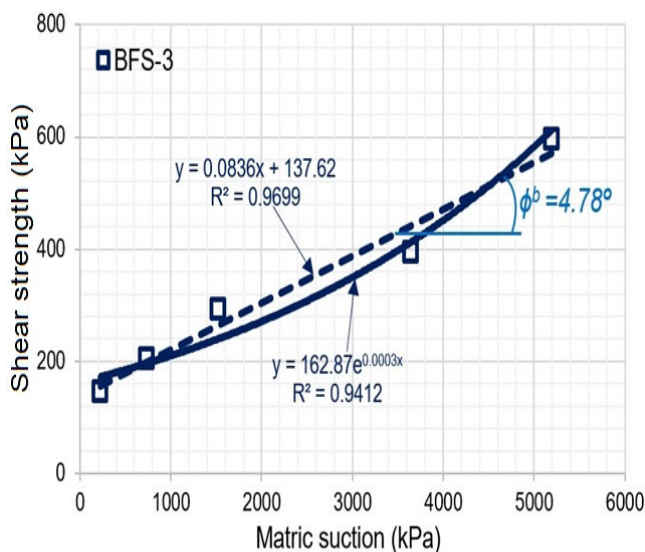


Figure 6.152: Unsaturated failure envelope for BFS-3

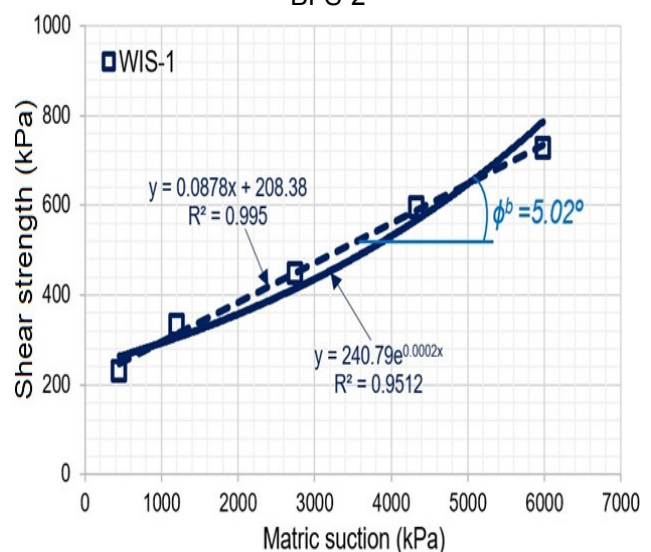


Figure 6.153: Unsaturated failure envelope for WIS-1

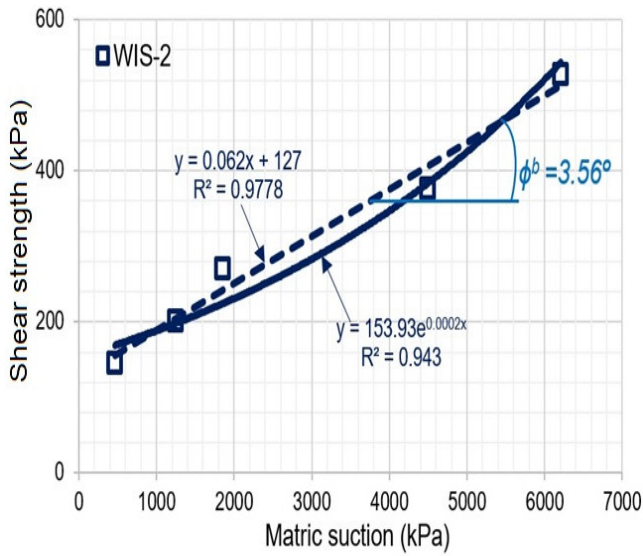


Figure 6.154: Unsaturated failure envelope for WIS-2

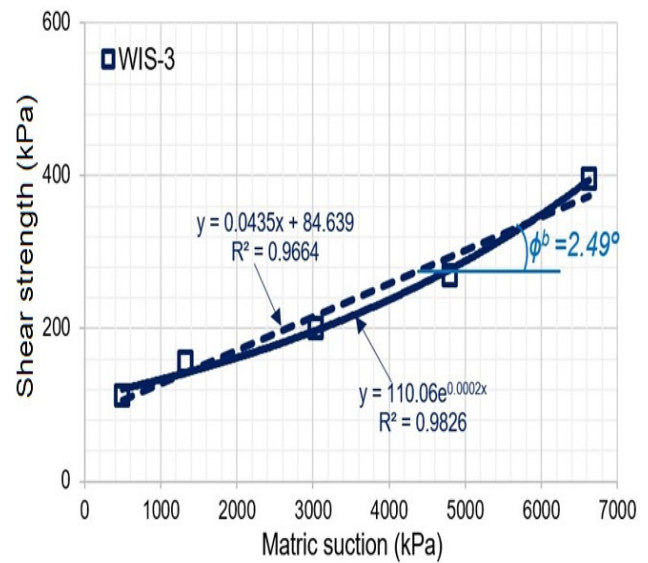


Figure 6.155: Unsaturated failure envelope for WIS-3

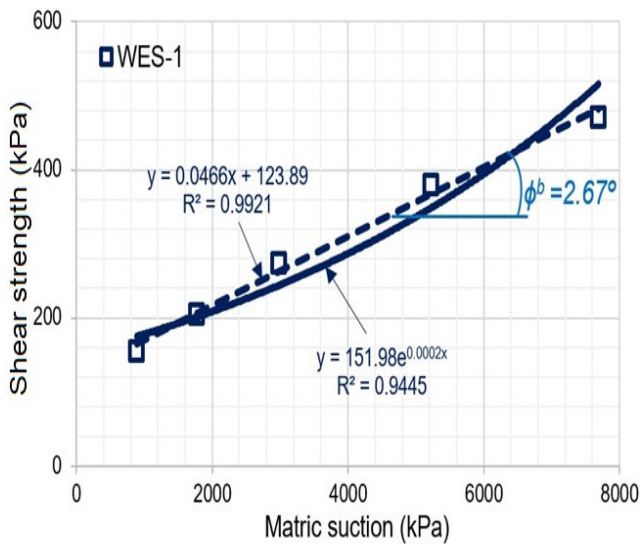


Figure 6.156: Unsaturated failure envelope for WES-1

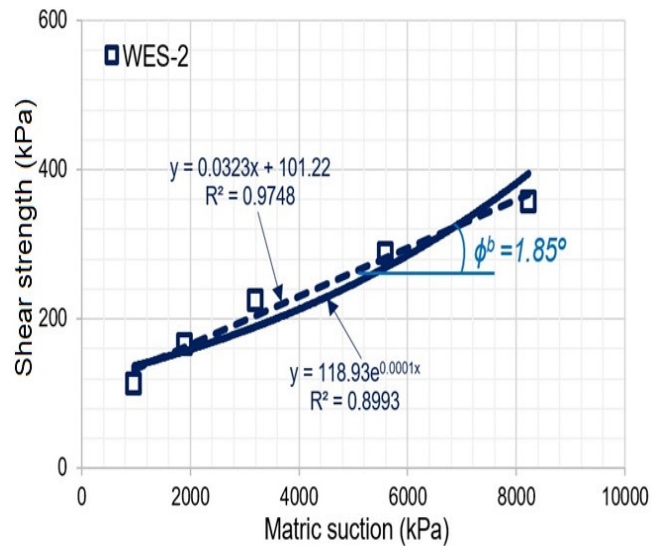


Figure 6.157: Unsaturated failure envelope for WES-2

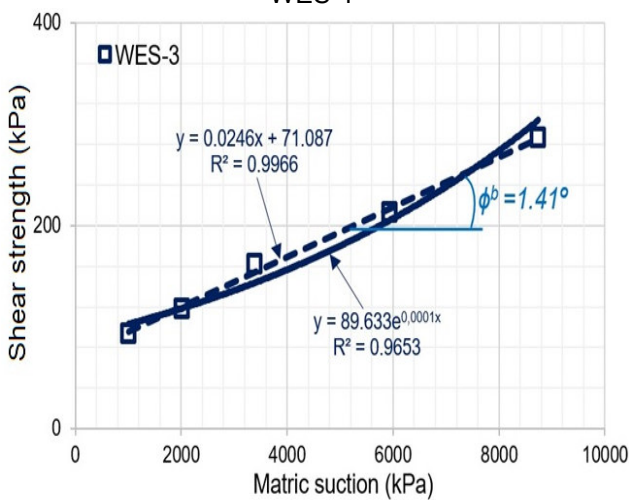


Figure 6.158: Unsaturated failure envelope for WES-3

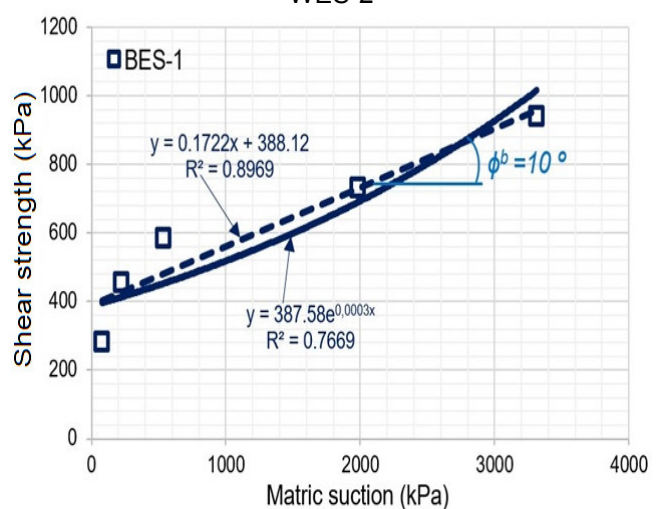


Figure 6.159: Unsaturated failure envelope for BES-1

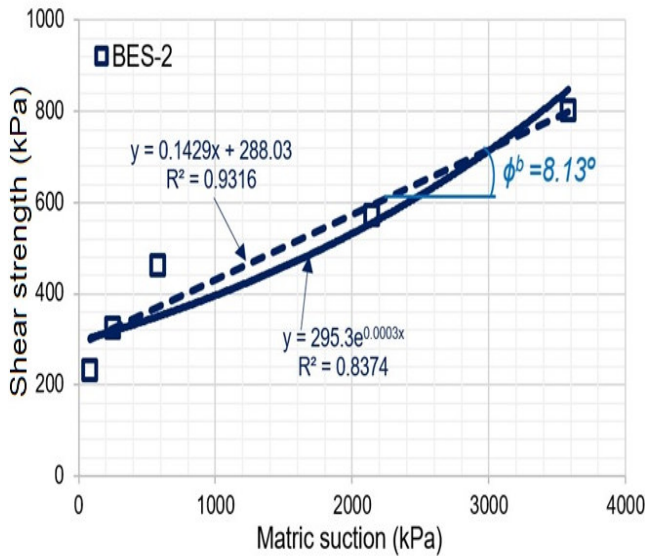


Figure 6.160: Unsaturated failure envelope for BES-2

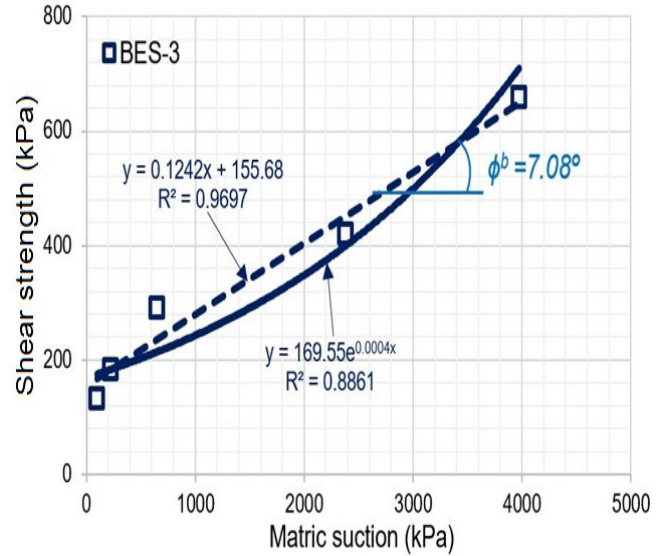


Figure 6.161: Unsaturated failure envelope for BES-3

6.7.5 Shear strength parameters' response to geotechnical index properties, and soil mineralogy

Shear strength parameters' response to geotechnical index properties and soil mineralogy was investigated in this section, based on experimental data obtained from tested samples BFS, WIS, WES, and BES. The shear strength parameters studied include the saturated friction angle (ϕ'), the angle indicating the rate of increase in shear strength with respect to a change in matric suction (ϕ^b), the angle indicating the rate of decrease in shear strength with respect to a change in swelling stress (ϕ^s) as described in section 6.7.6.1, and soil cohesion (c'). The degree of correlation between the dependent variables was interpreted according to Table 5.6 proposed by Kalayci (2010).

6.7.5.1 ϕ' response to water content, specific gravity, liquid limit, and plasticity index

Figures 6.162 and 6.163 depict the relationship between ϕ' and soil parameters: water content, Gs, LL, and PI. Figure 6.162 illustrates the surface plot of ϕ' , water content, and Gs. ϕ' decreases as Gs increases, displaying a strong correlation ($R^2=0.72$) with Gs as given in Equation 6.43 in Appendix 41. This correlation highlights the influence of Gs on ϕ' , a result consistent with the findings published by Oyediran and Durojaiye (2011). The ϕ' of tested soils also decreases as water content increases, following a strong correlation ($R^2=0.71$) provided in Equation 6.44 in Appendix 41. The rise in

moisture content within the soil material reduces the resistance to the interlocking of soil particles aligning with other studies published by Chen *et al.* (2012), Chowdhury (2013), Huzhu *et al.* (2017), Zhao *et al.* (2018), and Wang *et al.* (2020). Their studies emphasises the strong dependency of water content on ϕ' . Thus, water content and Gs significantly influence ϕ' .

Figure 6.163 exhibits the surface plot of ϕ' , liquid limit, and plasticity index. ϕ' decreases with increasing LL, displaying a moderate exponential correlation ($R^2=0.68$) described in Equation 6.45 in Appendix 41. Similarly, ϕ' decreases as the PI of the soil sample increases, showing a moderate relationship ($R^2=0.64$) described in Equation 6.46, Appendix 41. This trend can be attributed to the reduction in resistance to the interlocking of soil particles with the increase of Atterberg limits due to water content. These results correspond with the findings of Garven and Vanapalli (2006), Mousavi *et al.* (2011) and Akayuli *et al.* (2013), all indicating a moderate influence of Atterberg limits on the friction angle.

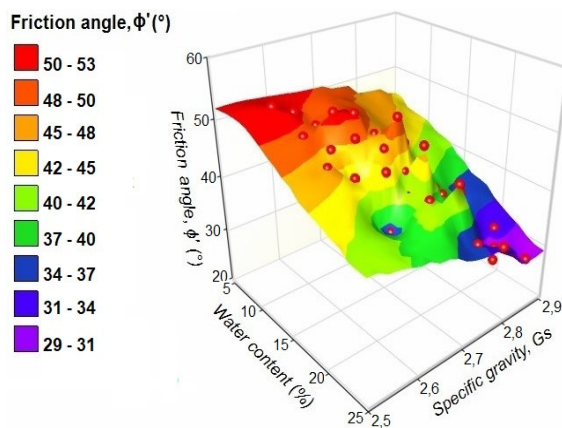


Figure 6.162: 3D response of (ϕ') dependence on W (%) and Gs

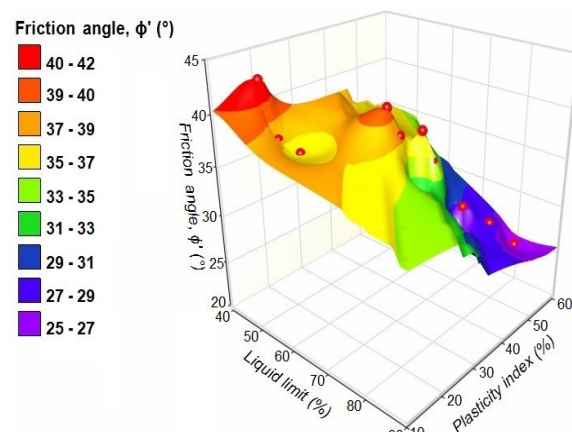


Figure 6.163: 3D response of (ϕ') dependence on LL and PI

6.7.5.2 ϕ' response to fine content, clay content, dry unit weight, and void ratio

Figures 6.164 and 6.165 present a three-dimensional representation of the relationship between ϕ' , fine content, clay content, dry unit weight (γ_d), and void ratio. Figure 6.164 displays a three-dimensional graph of ϕ' , fine content, and clay content. ϕ' decreases as the fine content increases in soil material, portraying a strong correlation ($R^2=0.72$) as provided in Equation 6.47 in Appendix 41. Additionally, the friction angle decreases with an increase in clay content within the soil, exhibiting a strong correlation ($R^2=0.74$) described in Equation 6.48 in Appendix 41. The reaction

of fine-grained particles with water reduces the resistance to the interlocking of soil particles, aligning with studies reported by Ranjan and Rao (2007) and Dafalla (2013). Figure 6.165 portrays the surface plot of ϕ' , dry unit weight, and void ratio. The ϕ' angle increases with an increase in dry unit weight, showing a moderate correlation ($R^2=0.62$) with dry unit weight as described by exponential Equation 6.49 in Appendix 41. This finding is consistent with studies published by Chen *et al.* (2012), Ojuri (2013), and Chowdhury (2013) on shear strength of expansive soils. ϕ' of soil material decreases as void ratio increases, exhibiting a moderate correlation ($R^2=0.69$) function with void ratio as described in Equation 6.50 in Appendix 41. This result aligns with research published by Warkentin and Yong (1962) and Wang *et al.* (2014) on the impact of soil fabric on shear strength.

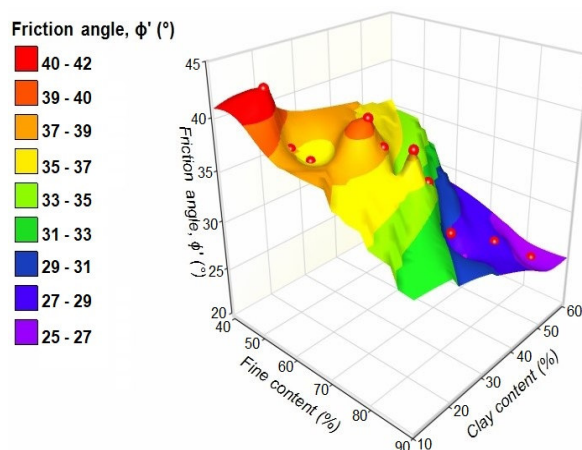


Figure 6.164: 3D response of (ϕ') dependence on fine (%) and clay (%)

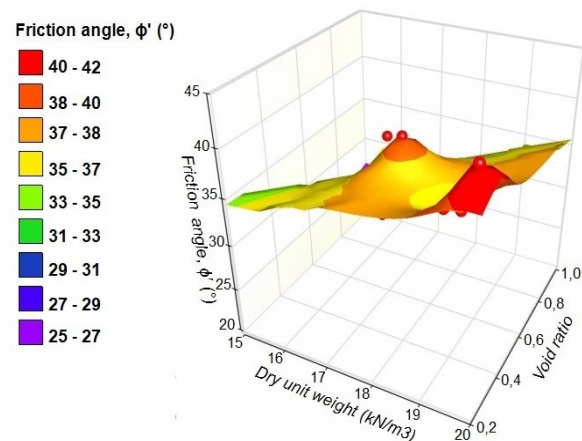


Figure 6.165: 3D response of (ϕ') dependence on γ_d and Void ratio

6.7.5.3 ϕ' response to free swell ratio, free swell index, smectite, and plagioclase

Figures 6.166 and 6.167 portray a three-dimensional description of the correlation between ϕ' and soil characteristics such as free swell ratio (FSR), free swell index (FSI), smectite content, and plagioclase content. Figure 6.166 illustrates the relationship among ϕ' , FSR, and FSI. The friction angle decreases as the free swell ratio increases. The exponential Equation 6.51 in Appendix 41 demonstrates a strong correlation ($R^2=0.74$) between ϕ' and FSR. Additionally, ϕ' decreases with an increase in the FSI of the tested soil, as indicated by the strong correlation ($R^2=0.70$) described in Equation 6.52 in Appendix 41. This indicates that ϕ' decreases with a higher swelling potential, aligning with findings from reports by Domitrović and Zelić (2013) and Deliktaş (2016) for similar expansive soils.

Figure 6.167 shows a surface plot of ϕ' , smectite content, and plagioclase content. As the smectite content increases, ϕ' decreases with a moderate correlation ($R^2=0.61$), as described by Equation 6.53 in Appendix 41. This indicates that the clay mineral (smectite) content influences ϕ' . These results are consistent with research published by Morrow *et al.* (1984), and Escario and Sáez (1986), which demonstrate a decrease in ϕ' with an increase in the quantity of clay mineral. On the other hand, ϕ' increases with an increase in plagioclase content, and portrays a moderate correlation ($R^2=0.53$) described in Equation 6.54 in Appendix 41. This trend can be attributed to the presence of plagioclase, which enhances the resistance to interlocking of soil particles.

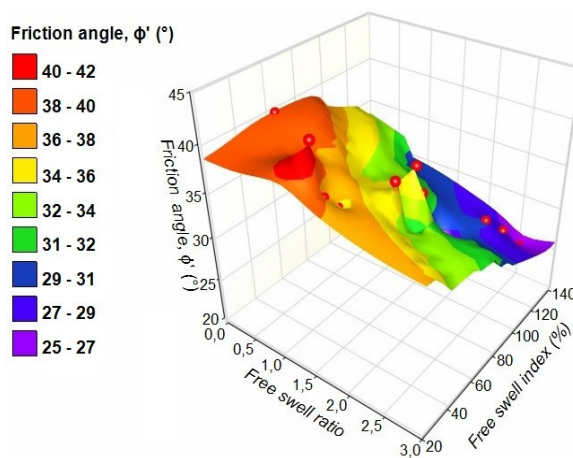


Figure 6.166: 3D response of (ϕ') dependence on FSR and FSI (%)

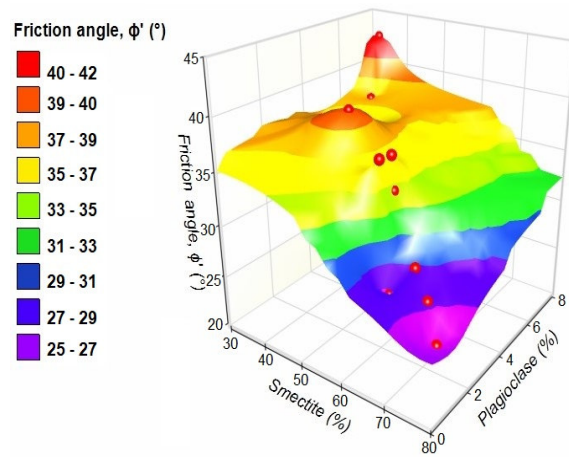


Figure 6.167: 3D response of (ϕ') dependence on smectite (%) and plagioclase (%)

6.7.5.4 ϕ^b response to fine content, clay content, liquid limit and plasticity index

Figure 6.168 and Figure 6.169 describe the correlation between the angle indicating the rate of increase in shear strength with respect to change in matric suction, denoted as (ϕ^b), and soil features such as fine content, clay content, LL, and PI.

Figure 6.168 presents a surface plot of ϕ^b , fine-grained content, and clay content. ϕ^b decreases as the fine content increases and exhibits a very strong correlation ($R^2=0.92$) given in Equation 6.55 in Appendix 41. Additionally, ϕ^b decreases with an increase in clay content and demonstrates a very strong correlation ($R^2=0.92$), as described by Equation 6.56 in Appendix 41. This trend is consistent with other studies reported by Morrow *et al.* (1984) and Escario and Sáez (1986) for similar expansive soils.

Figure 6.169 shows the correlation between ϕ^b , liquid limit, and plasticity index. ϕ^b decreases as the LL and PI increase. ϕ^b demonstrates very strong correlations with LL ($R^2=0.93$) and PI ($R^2=0.92$), as given in Equations 6.57 and 6.58 in Appendix 41, respectively. This finding aligns with results obtained by Garven and Vanapalli (2006) and Chowdhury (2013), which highlight the influence of Atterberg limits on the shear strength of unsaturated soils.

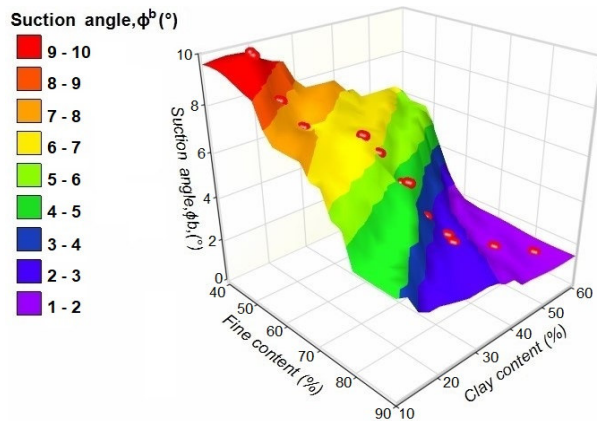


Figure 6.168: 3D response of (ϕ^b) dependence on fine (%) and clay (%)

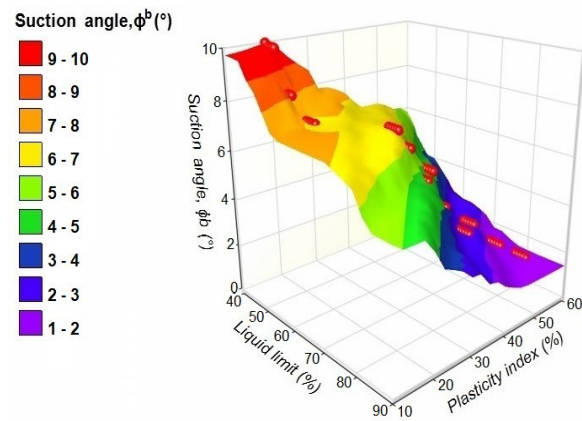


Figure 6.169: 3D response of (ϕ^b) dependence on LL (%) and PI (%)

6.7.5.5 ϕ^b response to specific gravity, dry unit weight, water content and air-entry value

Figures 6.170 and 6.171 describe the relationship between ϕ^b and soil parameters: G_s , γ_d , water content, and air entry value (AEV). Figure 6.170 illustrates the surface plot of ϕ^b , G_s , and γ_d . ϕ^b decreases as G_s increases, showing a strong correlation ($R^2=0.88$) with G_s , as described in Equation 6.59 in Appendix 41. This trend can be justified by the increase in G_s increases with the augmentation of clay content, leading to an increment of negative pore pressure and a subsequent decrease in ϕ^b . Nonetheless, ϕ^b increases with an increase in γ_d of the soil material, showing a strong correlation ($R^2=0.81$) with γ_d as given in Equation 6.60 in Appendix 41. These results are consistent with reports by Escario *et al.* (1989), Chen *et al.* (2012), and Chowdhury (2013) on the shear strength of unsaturated soils.

Figure 6.171 illustrates a surface plot of ϕ^b , water content, and AEV. ϕ^b decreases as water content increases, exhibiting a strong correlation ($R^2=0.88$) with water content as given in Equation 6.61 in Appendix 41. These results align with research published by Chen *et al.* (2012), Chowdhury (2013), Domitrović and Zelić (2013), which show a decrease in shear strength of expansive soils with increasing water

content. ϕ^b decreases with an increase in AEV, showing a very strong correlation ($R^2=0.92$) with AEV as described in Equation 6.62 in Appendix 41. This trend is consistent with other results published by Vanapalli *et al.* (1996), Tekinsoy *et al.* (2004), and Guan *et al.* (2010), which show the influence of AEV on the unsaturated shear strength.

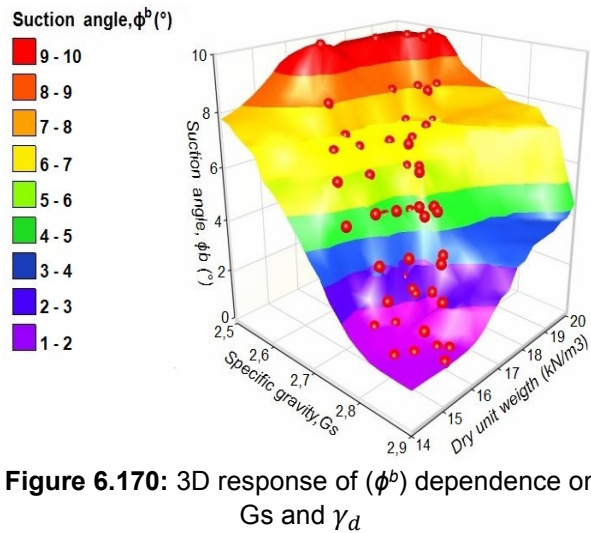


Figure 6.170: 3D response of (ϕ^b) dependence on G_s and γ_d

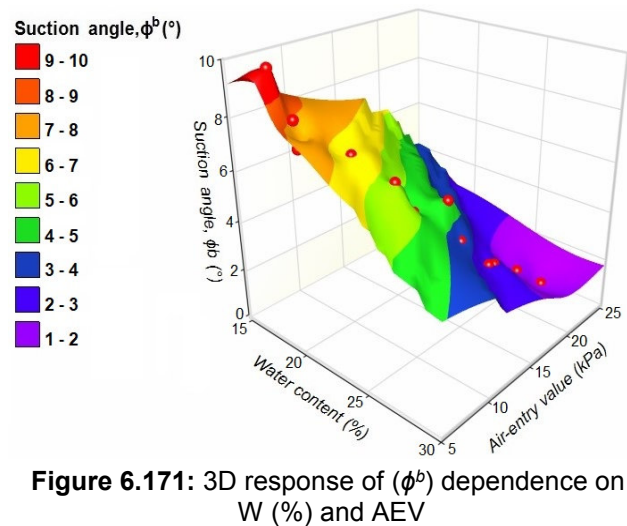


Figure 6.171: 3D response of (ϕ^b) dependence on W (%) and AEV

6.7.5.6 ϕ^b response to smectite, free swell index, plagioclase, and K-feldspar

Figures 6.172 and 6.173 illustrate the three-dimensional relationship between ϕ^b and smectite content, FSI, plagioclase content, and K-feldspar content. Figure 6.172 depicts the surface plot of ϕ^b , smectite content, and FSI. ϕ^b decreases as FSI increases, showing a very strong correlation ($R^2=0.93$) with FSI as described in Equation 6.63 in Appendix 41. This decrease in shear strength with the increment of swelling potential aligns with findings from reports by Al-Mhaidib and Al-Shamrani (2000), Domitrović and Zelić (2013), and Deliktaş (2016) on unsaturated shear strength. Additionally, ϕ^b decreases with an increase in smectite content, showing a strong correlation ($R^2=0.79$) with smectite content according to Equation 6.64 in Appendix 41. The proportion of smectite significantly impacts the shear strength of expansive soils, consistent with studies by Escario (1980), Morrow *et al.* (1984), and Escario and Sáez (1986) on similar soils.

Figure 6.173 presents a surface plot of ϕ^b , plagioclase content, and K-feldspar content. ϕ^b increases with higher plagioclase content in the soil material, showing a moderate correlation ($R^2=0.67$) with plagioclase content described in Equation 6.65 in

Appendix 41. ϕ^b increases with higher K-feldspar content, displaying a moderate correlation ($R^2=0.66$) with K-feldspar as outlined in Equation 6.66 in Appendix 41. This suggests that feldspar minerals exhibit a moderate influence on ϕ^b .

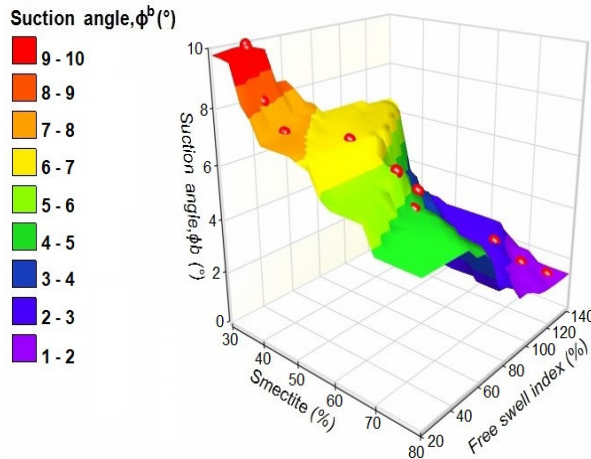


Figure 6.172: 3D response of (ϕ^b) dependence on smectite (%) and FSI (%)

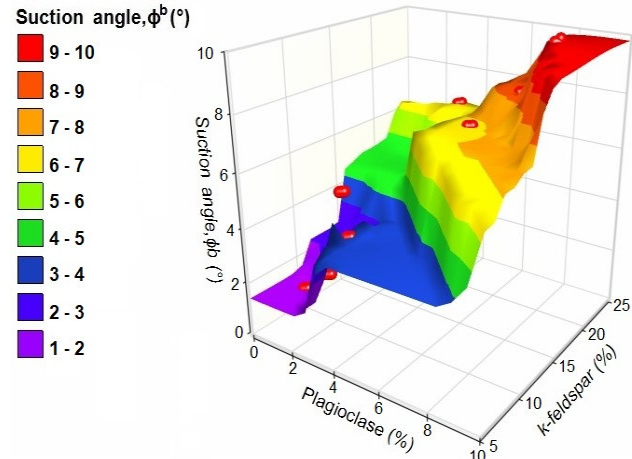


Figure 6.173: 3D response of (ϕ^b) dependence on plagioclase (%) and K-feldspar (%)

6.7.5.7 ϕ^s response to water content, specific gravity, plasticity index, and liquid limit

Figure 6.174 and 6.175 present a three-dimensional description of the correlation between the ϕ^s and soil features such as water content, Gs, PI, and LL. The angle ϕ^s indicates the rate of decrease in shear strength with respect to the variation of swelling stress, as described in section 6.7.6.1. Figure 6.174 shows a surface plot of ϕ^s , water content, and Gs. ϕ^s decreases as water content increases and displays a strong correlation ($R^2=0.70$) function described in Equation 6.67 in Appendix 42. This trend can be explained as follows: The shear strength decreases, and swelling stress grows with the increment of water content in compacted soil. ϕ^s decreases with the increase in Gs of fine-grained soils and indicates a strong correlation ($R^2=0.70$) described in Equation 6.68 in Appendix 42. This observation can be explained as follows: the gradient between shear strength and swelling stress of compacted soil reduces with the increment of water and Gs. It is evident that water content and specific gravity influence the magnitude of ϕ^s . Figure 6.175 depicts the surface plot of ϕ^s , LL, and PI. The graph illustrates the decrease of ϕ^s with the increments of LL and PI. ϕ^s demonstrates moderate correlations with LL ($R^2=0.57$) and PI ($R^2=0.57$), as given by exponential Equations 6.69 and 6.70 in Appendix 42, respectively. This trend can be

justified as follows: The PI and LL increase with the increment of water content. As a result, the angle ϕ^s decreases. The observations indicate that LL and PI impact the magnitude of ϕ^s .

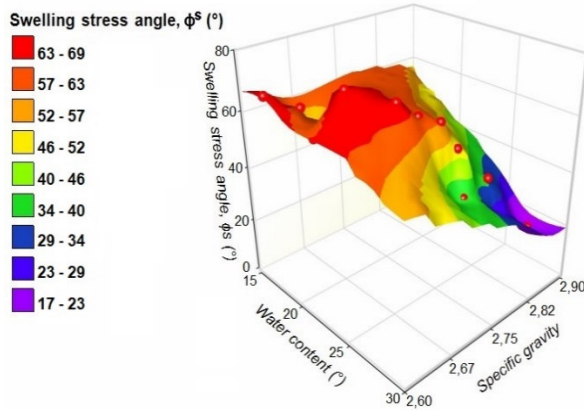


Figure 6.174: 3D response of (ϕ^s) dependence on W (%) and Gs

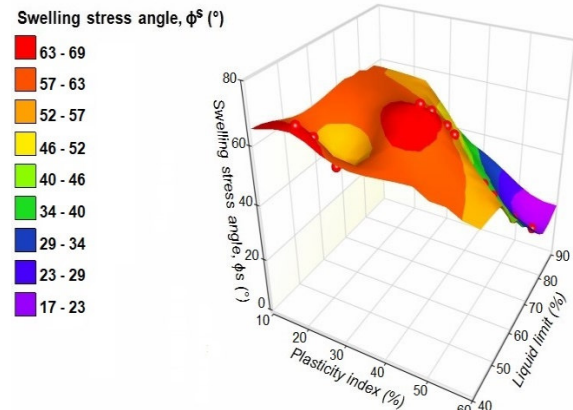


Figure 6.175: 3D response of (ϕ^s) dependence on PI (%) and LL (%)

6.7.5.8 ϕ^s response to fine content, clay content, dry unit weight, and void ratio

Figures 6.176 and 6.177 describe the correlation between ϕ^s , and soil features such as fine content, clay content, γ_d , and void ratio. Figure 6.176 shows the surface plot of ϕ^s , fine content and clay content. The swelling angle ϕ^s decreases when the fine content increases in soil material and exhibits a moderate correlation ($R^2=0.65$) described in Equation 6.71 in Appendix 42. Additionally, ϕ^s decreases when the clay content increases and shows a strong correlation ($R^2=0.75$) given in Equation 6.72 in Appendix 42. These trends can be explained as follows: The clay minerals present in heaving soils react with water, leading to increased swelling stress in the soil material. This, in turn, reduces the shear strength of the soil material and causes ϕ^s to decrease. Based on the observations, both fine content and clay content influence ϕ^s . Figure 6.177 portrays the surface plot of ϕ^s , γ_d , void ratio. There is a tendency for ϕ^s to increase when γ_d increases at the dry side and shows a moderate correlation ($R^2=0.50$) with γ_d as described in exponential Equation 6.73 in Appendix 42. This trend aligns with the observations reported by Chen *et al.* (2012) and Chowdhury (2013) on the shear strength of expansive soils. Additionally, ϕ^s tends to decrease when the void ratio increase in the soils and shows a moderate correlation ($R^2=0.60$) with the void ratio described in Equation 6.74 in Appendix 42. This observation can be justified by the fact that the void ratio increases in soil material as swelling stress grows due to

the addition of water, causing the shear strength to reduce. Wang *et al.* (2014) reported a reduction in shear strength of expansive soil with increasing void ratio. Thus, ϕ^s decreases with an increment in the void ratio. Given these trend interpretations, both γ_d and void ratio influence ϕ^s .

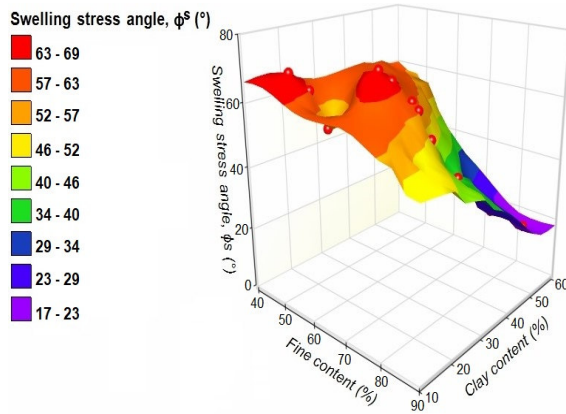


Figure 6.176: 3D response of (ϕ^s) dependence on fine (%) and clay (%)

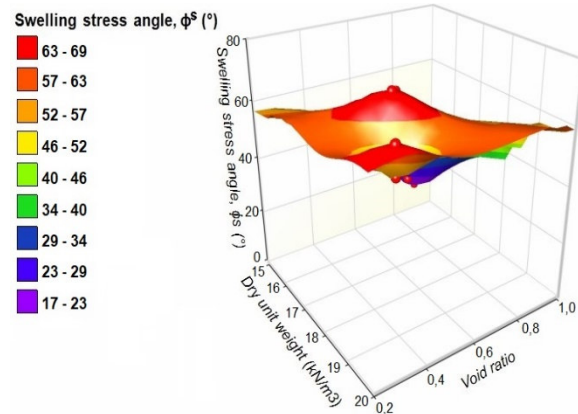


Figure 6.177: 3D response of (ϕ^s) dependence on γ_d and void ratio

6.7.5.9 ϕ^s response to free swell index, free swell ratio, smectite, and K-feldspar

Figures 6.178 and 6.179 depict the three-dimensional representation of the relationship between ϕ^s and FSI, FSR, smectite content, and K-feldspar content. Figure 6.178 shows the surface plot of ϕ^s , FSR and FSI. ϕ^s decreases when FSR and FSI increase and exhibits strong correlations ($R^2=0.76$) given in Equations 6.75 and 6.76 in Appendix 42, respectively. This trend can be justified by the fact that the magnitude of swelling potential increases with the increment of FSI and FSR in expansive soils. This observation is consistent with the reports published by Carter and Bentley (1991) and Sridharan and Prakash (2000). Thus, the swelling potential has an influence on ϕ^s .

Figure 6.179 displays a three-dimensional graph of ϕ^s , smectite content, and K-feldspar content. ϕ^s decreases with an increment of smectite content in soil material and shows a moderate correlation ($R^2=0.50$) as described in Equation 6.77 in Appendix 42. This trend can be attributed to the swelling stress developed within the soil material upon water addition due to the smectite, which reduces the shear strength. This observation concurs with the reports published by Domitrović and Zelić (2013) and Deliktaş (2016). Thus, ϕ^s decreases with an increase of smectite content.

ϕ^s increases with an increase in K-feldspar content and exhibits a moderate correlation ($R^2=0.58$) as given in Equation 6.78 in Appendix 42.

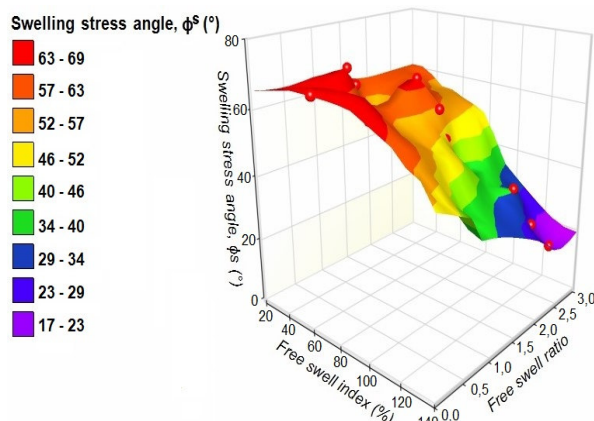


Figure 6.178: 3D response of (ϕ^s) dependence on FSI (%) and FSR

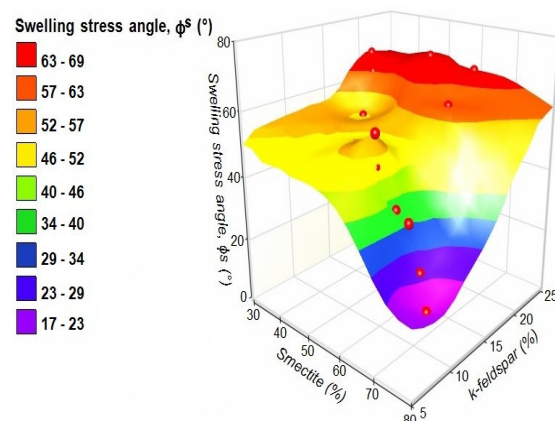


Figure 6.179: 3D response of (ϕ^s) dependence on smectite (%) and K-feldspar (%)

6.7.5.10 c' response to water content, specific gravity, liquid limit, and plasticity index

Figures 6.180 and 6.181 provide a three-dimensional description of the correlation between c' and soil features such as water content, Gs, LL, and PI.

Figure 6.180 presents a surface plot of c' , water content, and Gs. c' decreases as water content increases and demonstrates a strong correlation ($R^2=0.73$) as given in Equation 6.79 in Appendix 42. This trend can be explained by the decrease in intermolecular bonds between the water and grain particles with the increase in water content. This observation is consistent with reports published by Stark and Duncan (1992), Chen *et al.* 2012, Bláhová *et al.* (2013), Zhao *et al.* (2018), and Wang *et al.* (2014) that show a strong dependency of c' on water content. c' decreases with an increase in Gs and exhibits a strong correlation ($R^2=0.71$) as described in Equation 6.80 in Appendix 42. This finding aligns with the study by Oyediran and Durojaiye (2011) on the variability in geotechnical properties of clay soils. It can be concluded that Gs and water content influence c' .

Figure 6.181 displays the surface plot of c' , LL, and PI. The surface plot illustrates a reduction in c' with the increase in LL and PI. The correlation between c' , LL, and PI demonstrates a moderate correlations of ($R^2=0.61$) and ($R^2=0.56$) respectively, described by exponential Equations 6.81 and 6.82 in Appendix 42, respectively. This trend can be attributed to the fact that LL and PI increase with the increment of water

and reduces the intermolecular bond between the adsorbed water surrounding each grain in the soil material. The LL and PI play a role in influencing soil cohesion. This finding is consistent with the results reported by Akayuli *et al.* (2013), Mousavi *et al.* (2011), and Tafari *et al.* (2021), which show the influence of Atterberg limits on soil cohesion.

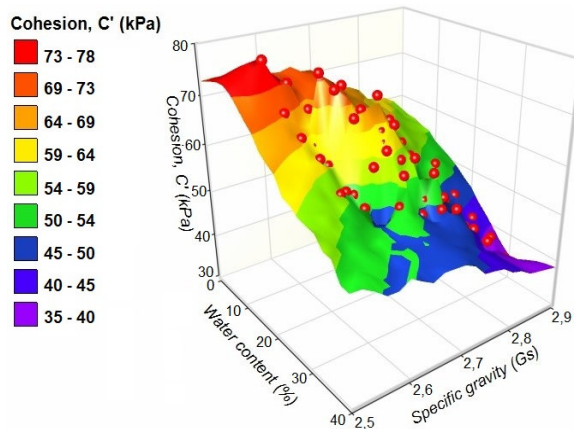


Figure 6.180: 3D response of (c') dependence on W (%) and G_s

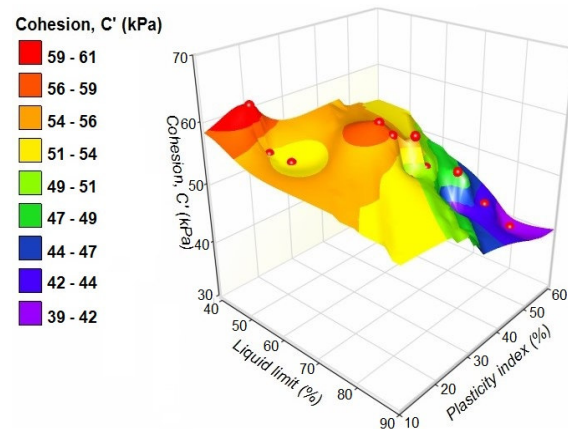


Figure 6.181: 3D response of (c') dependence on LL (%) and PI (%)

6.7.5.11 c' response to fine content, clay content, void ratio, and dry unit weight

Figures 6.182 and 6.183 describe the correlation between c' and soil features such as fine content, clay content, void ratio, and γ_d . Figure 6.182 presents the surface plot of c' , fine content and clay content. There is a tendency for c' to decrease when the fine content increases. c' shows a moderate correlation ($R^2=0.68$) with fine-grained content described in Equation 6.83 in Appendix 42. Similarly, c' tends to decrease as the clay content in the soil grows. c' portrays a strong correlation ($R^2=0.71$) with clay content, as given in Equation 6.84 in Appendix 42. Clays and silts react with water, change sizes, and reducing the intermolecular bond between the adsorbed water surrounding each grain. These results are consistent with the trends observed in the study published by Escario (1980) and Morrow *et al.* (1984) on the shear strength of swelling soils.

Figure 6.183 shows the surface graph of c' , void ratio, and γ_d . c' decreases as the void ratio increases, indicating a moderate correlation ($R^2=0.63$) described in Equation 6.85 in Appendix 42. This trend is attributed to the presence of significant amounts of clays and silts that lead to more voids and less particle-to-particle contact. Consequently, increasing the void ratio, reduces cohesion. This observation aligns

with other research studies by Warkentin and Yong (1962), Dafalla (2013), Wang *et al.* (2014), and Tafari *et al.* (2021) on the shear strength properties of soil materials. c' increases when γ_d increases and exhibits a moderate correlation ($R^2=0.54$), as described in Equation 6.86 in Appendix 42. This increment of γ_d enhances the intermolecular bond between the adsorbed water surrounding each grain in the soil material. This finding concurs with studies published by Chen *et al.* (2012), Chowdhury (2013), and Tafari *et al.* (2021) on similar soils. The void ratio and dry unit weight influence the c' of the soil.

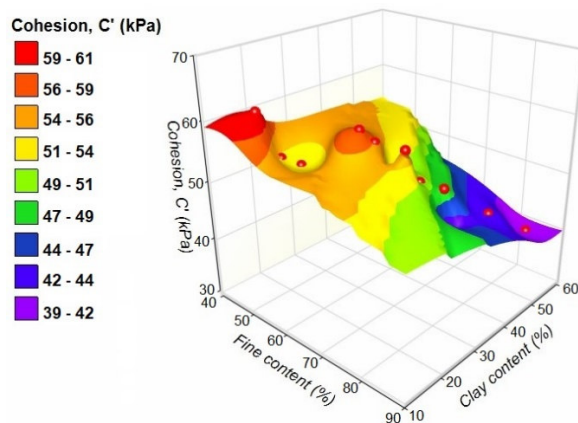


Figure 6.182: 3D response of (c') dependence on fine (%) and clay (%)

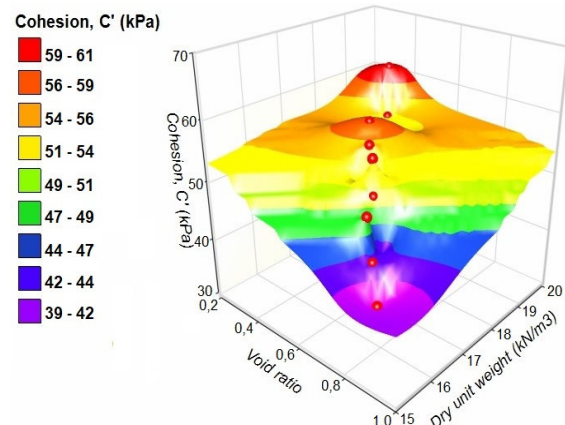


Figure 6.183: 3D response of (c') dependence on γ_d and void ratio

6.7.5.12 c' response to smectite, free swell index, K-feldspar, and plagioclase

Figures 6.184 and 6.185 describe the relationship between c' and soil parameters: smectite content, FSI, K-feldspar content, and plagioclase content. Figure 6.184 portrays the surface plot of c' , smectite content, and FSI. c' decreases with the growth of smectite content and exhibits a moderate correlation ($R^2=0.52$) with the smectite content given in Equation 6.87 in Appendix 42. These results align with the studies published by Escario (1980) and Escario and Sáez (1986), which indicate that c' , decreases with the increment of clay minerals within the soil. Another study by Morrow *et al.* (1984) also stated that c' , reduces when the quantity of clay minerals increases. Similarly, c' decreases when the FSI of the soil increases and displays a moderate correlation ($R^2=0.62$) described in Equation 6.88 in Appendix 42. This trend is consistent with other investigations reported by Domitrović and Zelić (2013) and Deliktaş (2016) for similar expansive soils. The local extremes on the surface plot originate from discrepancies in correlations.

Figure 6.185 shows the surface graph of c' , K-feldspar content and plagioclase content. c' increases when the K-feldspar content increases and shows a moderate correlation ($R^2=0.53$) with K-feldspar content, as given in Equation 6.89 in Appendix 42. c' increases when the plagioclase content grows and shows a moderate correlation ($R^2=0.50$) with plagioclase content described in Equation 6.90 in Appendix 42.

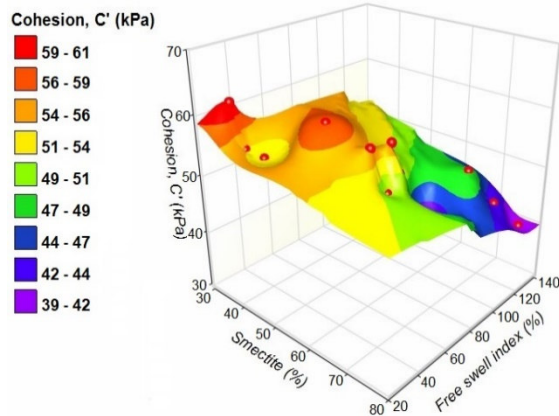


Figure 6.184: 3D response of (c') dependence on smectite (%) and FSI (%)

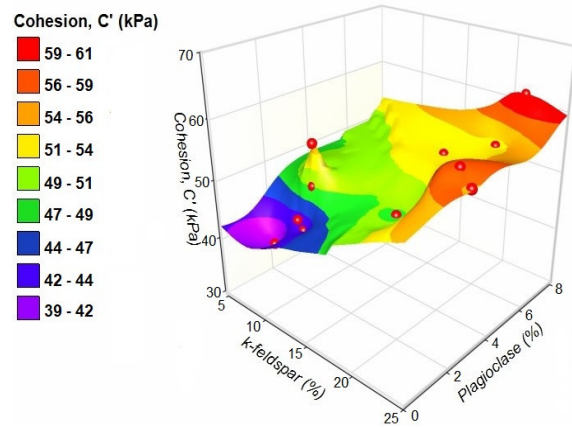


Figure 6.185: 3D response of (c') dependence on K-feldspar (%) and plagioclase (%)

6.7.5.13 Conclusion

ϕ' , ϕ^b , ϕ^s , and c' are primarily influenced by water content (%), specific gravity, and clay content (%). ϕ' exhibits a strong correlation with the free swell ratio and free swell index. ϕ^b shows a strong relationship with the liquid limit, plastic limit, and dry unit weight. ϕ^s demonstrates a strong correlation with the free swell ratio and free swell index. ϕ' , ϕ^s , and c' portray a moderate relationship with the liquid limit, plastic limit, dry unit weight, and void ratio. ϕ' displays a moderate correlation with smectite (%) and plagioclase (%). ϕ^b describes a strong relationship with smectite (%) and a moderate correlation with plagioclase and K-feldspar (%). ϕ^s shows a moderate relationship with smectite (%) and K-feldspar (%). c' depicts a moderate correlation with smectite (%), K-feldspar (%), and plagioclase (%).

ϕ' , ϕ^b , ϕ^s , and c' increase when the dry unit weight increases and decrease when the plastic limit, liquid limit, void ratio, free swell index, free swell ratio, clay fraction, and fine-grained fraction increase. ϕ' decreases when the smectite content increases, and increases when the plagioclase content increases. ϕ^b decreases when the smectite

content increases, and increases when the plagioclase and K-feldspar content grow. ϕ^s decreases when the smectite content increases, and increases when the K-feldspar content increases. c' decreases when the smectite content increases and increases when the plagioclase and K-feldspar content grow.

6.7.6 Modified unsaturated shear strength (τ_{mu})

6.7.6.1 Relationship between shear strength and swelling stress

The graph of shear strength against swelling stress provides the slope (ϕ^s) between the shear strength-swelling stress curves. The ϕ^s angle indicates the rate of reduction in shear strength with respect to a variation of swell stress, and the ϕ^b angle describes the rate of increase in shear strength of soil material with respect to a change in matric suction.

Figures 6.186 to 6.197 illustrate the graphs of shear strength against swelling stress for various soil samples. These graphs were plotted using the Microsoft Excel program. The trend line equations, including linear, logarithmic, exponential, power, and polynomial functions were analysed. Among them, exponential and linear functions were the best fit and the slope of the linear function slope was decisive in determining the angle (ϕ^s). These graphs exhibit a similar trend. The shear strength of the tested soils decreases as the swelling stress increases, and this relationship demonstrates a strong correlation. These findings align with research by Calabresi and Scarpelli (1985), Wong (1998), and Al-Mhaidib and Al-Shamrani (2000). Wiseman (1993) reported that the swelling stress and shear strength of expansive soil depend on wetting conditions, and both are not dissociable. Al-Mhaidib and Al-Shamrani (2000) noted that the shear strength value of expansive soil decreases by 10 % compared to non-swollen soil specimens. Deliktaş (2016) observed a 90 % reduction in shear strength when soil specimen reached an ultimate swelling value of 100 %. Ikechukwu (2019) found a decrease in shear strength with the increment of swelling stress in compacted expansive soils. However, the swelling behaviour of soil material is mainly influenced by the type of clay mineral present. XRD test results in Figures 6.9 to 6.20, Appendices 3 and 4, show that smectite is the predominant clay mineral. WES has higher smectite content, higher swelling stress values, and lower shear strength values. In contrast, BES has lower smectite content, smaller swelling stress values, and higher shear strength values. Overall, the results indicate that shear

strength decreases with increasing smectite content in the soil, while swelling stress increases when the smectite content increases in compacted expansive soil. The interaction of smectite with water reduces shear strength due the swelling capacity of smectite. These trends align with the findings of Escario and Sáez (1986), Morrow *et al.* (1984), and Ikechukwu (2019). Therefore, in expansive soil material, the shear strength should not be dissociated with the influence of swelling stress. Swelling stress contribution is essential for formulating a realistic constitutive relation for the shear strength equation in expansive unsaturated soils. The reported values of (ϕ^s) ranges from 59.83° to 69° for BFS, 37.16° to 57.83° for WIS, 17° to 35° for WES, and 68.33° to 51.36° for BES. The graphs of shear strength against swelling stress demonstrate that the magnitude of the angle ϕ^s , representing the rate of shear strength reduction with respect to a variations of swelling stress, is significantly influenced by the shear strength response to swelling stress.

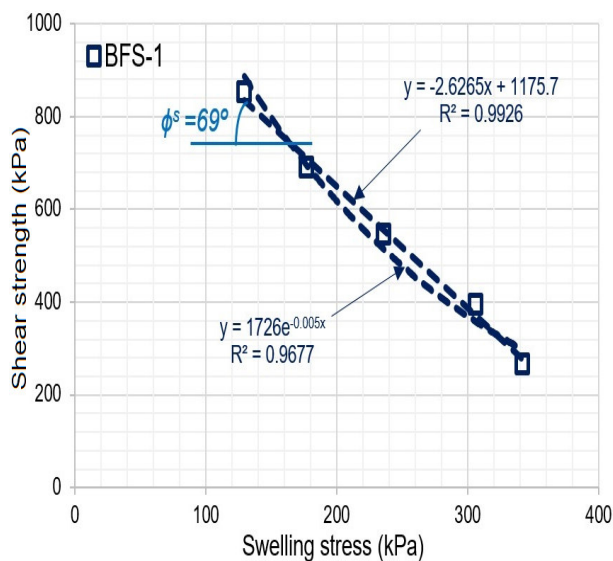


Figure 6.186: Shear strength vs swelling stress for BFS-1

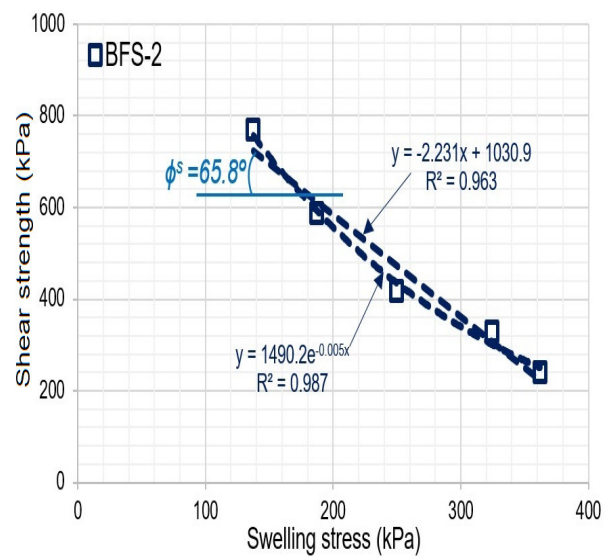


Figure 6.187: Shear strength vs swelling stress for BFS-2

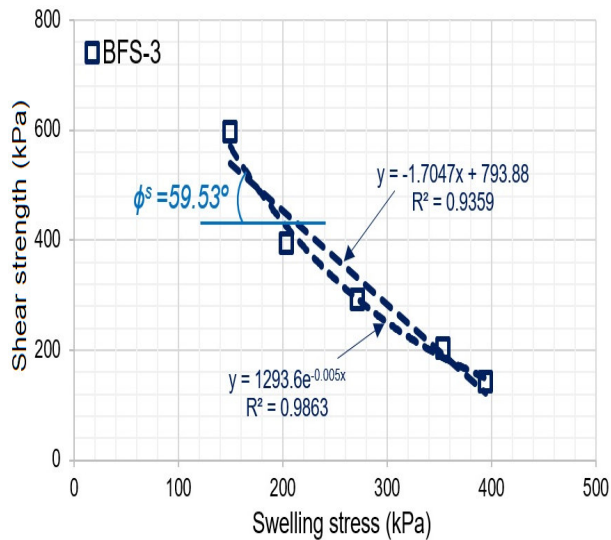


Figure 6.188: Shear strength vs swelling stress for BFS-3

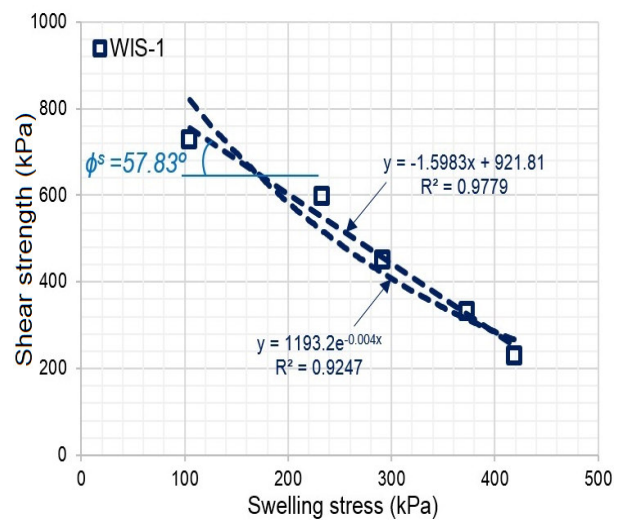


Figure 6.189: Shear strength vs swelling stress for WIS-1

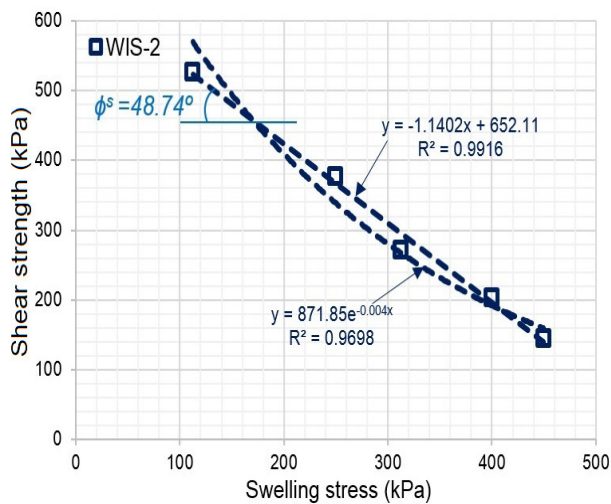


Figure 6.190: Shear strength vs swelling stress for WIS-2

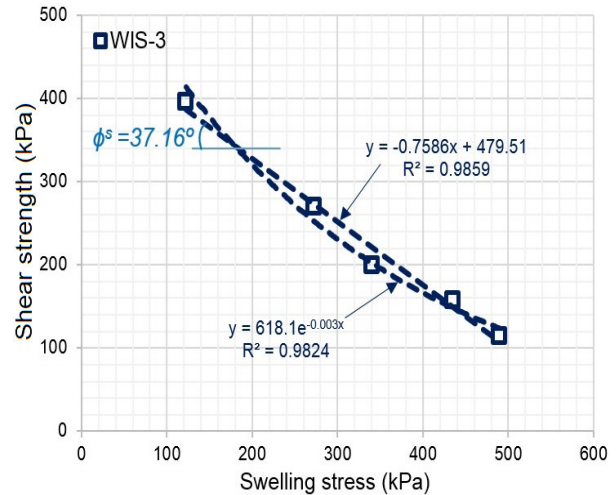


Figure 6.191: Shear strength vs swelling stress for WIS-3

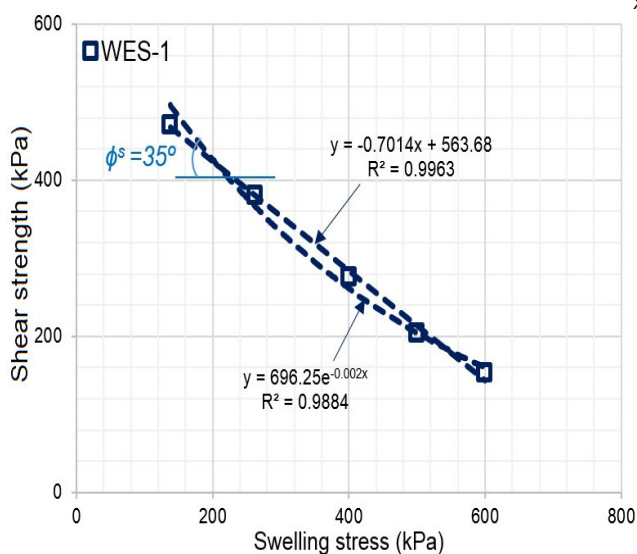


Figure 6.192: Shear strength vs swelling stress for WES-1

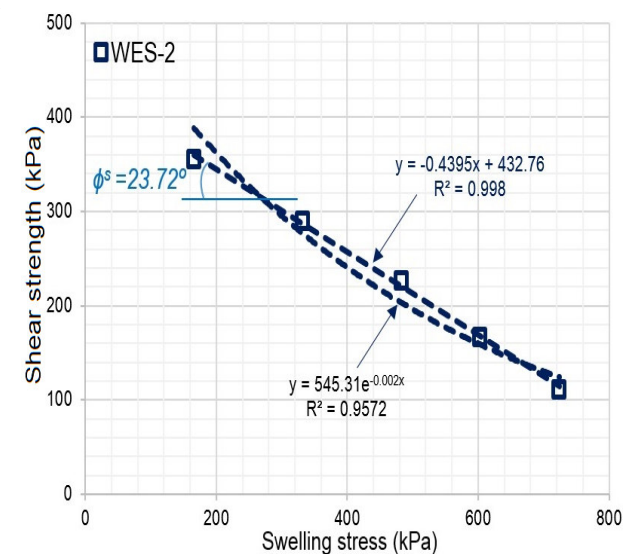


Figure 6.193: Shear strength vs swelling stress for WES-2

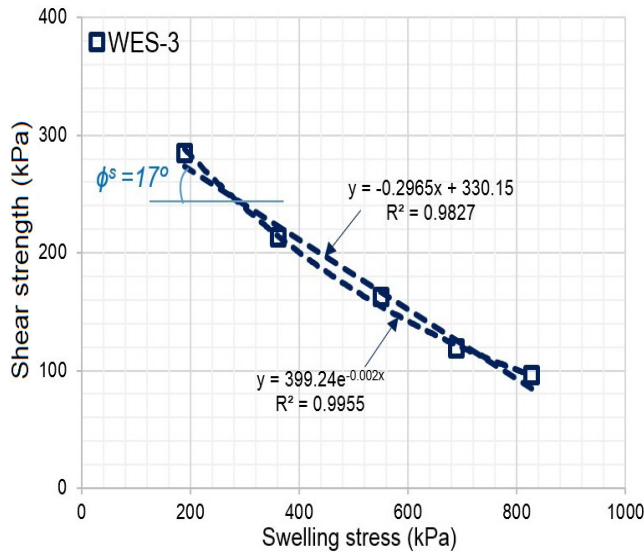


Figure 6.194: Shear strength vs swelling stress for WES-3

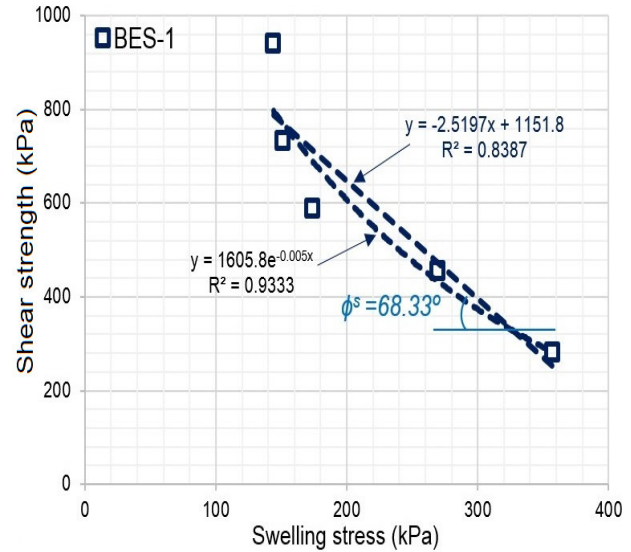


Figure 6.195: Shear strength vs swelling stress for BES-1

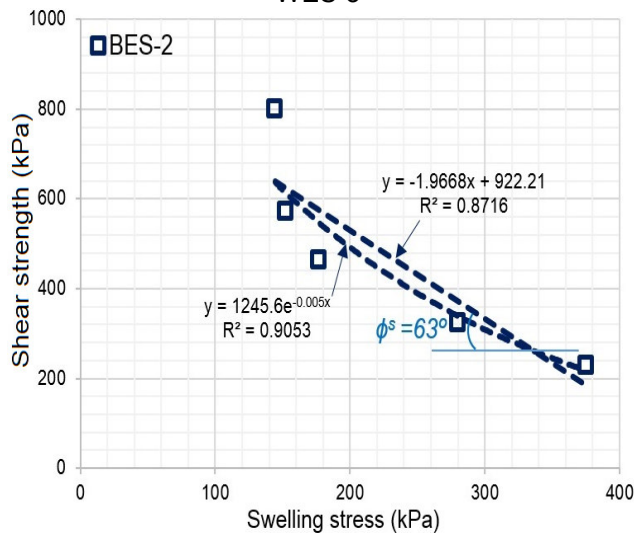


Figure 6.196: Shear strength vs swelling stress for BES-2

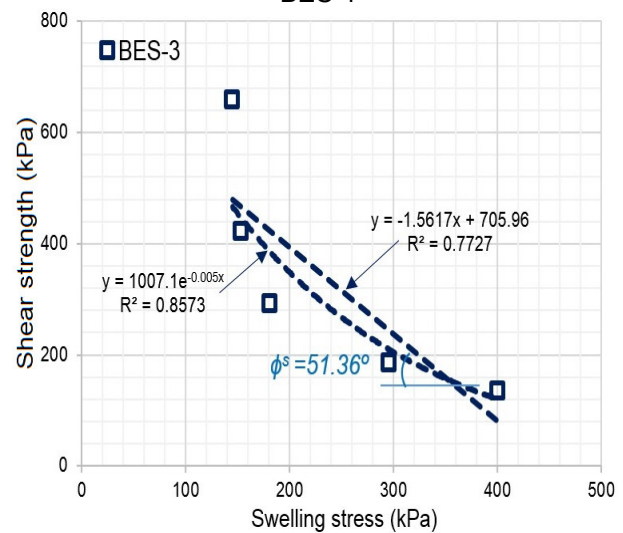


Figure 6.197: Shear strength vs swelling stress for BES-3

6.7.6.2 Modified-unsaturated shear strength constitutive relation

The stress state variable is a non-material variable used to describe the physical behaviour of a material under consideration. Stress state variables depict the equilibrium state of the system. When one or more stress state variables change, the system responds to establish a new equilibrium state. Therefore, stress state variables form the building blocks of any constitutive relation. Terzaghi (1936) utilised the effective stress variable and the Mohr-Coulomb failure criterion to formulate a mathematical equation predicting the shear strength (τ_s) of saturated soils. Fredlund and Rahardjo (1993) indicated that in unsaturated soils, matric suction ($u_a - u_w$) tends to compress the soil material, leading to increased soil cohesion, and consequently augmented shear strength. Fredlund and Rahardjo (1993) developed an unsaturated

shear strength (τ_u) constitutive relation employing two stress state variables: effective normal stress ($\sigma_n - u_a$) and matric suction ($u_a - u_w$). The contribution of matric suction is incorporated into saturated shear strength to obtain τ_u . However, when unsaturated soils exhibit swelling behaviour, a swelling stress (P_s) develops within the soil material, introducing complex issues that require thorough investigation. Calabresi and Scarpelli (1985), Wong (1998), Al-Mhaidib and Al-Shamrani (2000), Deliktaş (2016), and Ikechukwu (2019) highlighted the significant impact of swelling in reducing cohesion between soil particles in expansive soils. Swelling behaviour introduces an additional stress state variable (P_s) into the soil material. In this section, we use three stress state variables: effective normal stress ($\sigma_n - u_a$), matric suction ($u_a - u_w$), and swelling stress (P_s) to establish a constitutive relation denoted as the modified-unsaturated shear strength for expansive unsaturated soil. This approach aligns with the research conducted by Leong (2019) on the controversy and consensus of stress state variables for unsaturated soils. The study indicated that in recent years, it has been proposed that more than two stress state variables might be necessary to describe the constitutive behaviour of unsaturated soils. As a consequence of the contribution of the swelling stress, three autonomous stress tensors can be composed in three directions (x, y, z) as presented in Figure 6.198. Additionally, Equations 6.91, 6.92, and 6.93 describe the three autonomous tensors associated with effective normal stress, matric suction, and swelling stress, respectively.

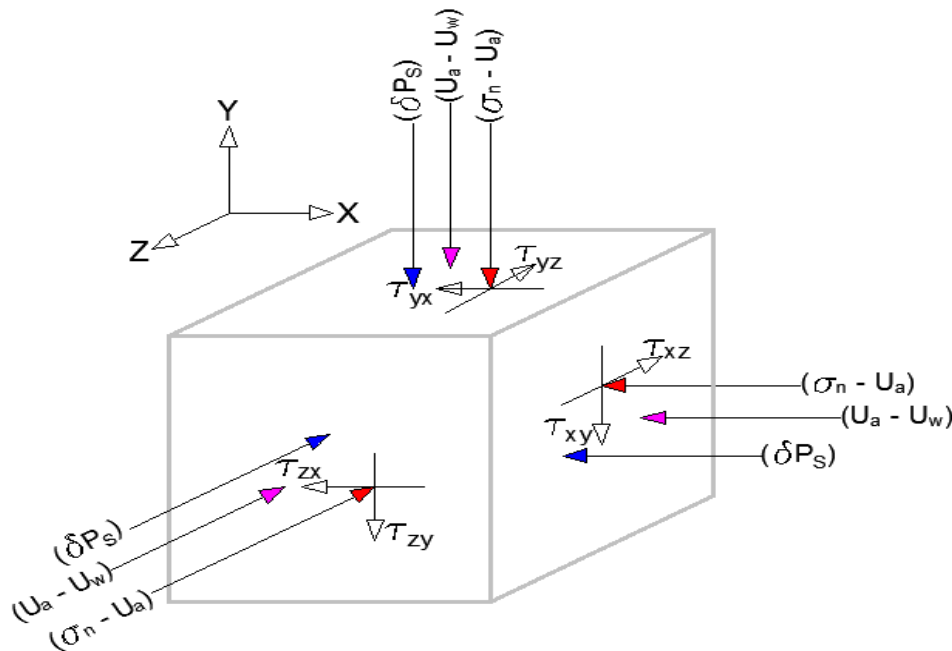


Figure 6.198: Stress state variables

$$\begin{bmatrix} \sigma_n - u_a & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & \sigma_n - u_a & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & \sigma_n - u_a \end{bmatrix} \quad [6.91]$$

$$\begin{bmatrix} (u_a - u_w) & 0 & 0 \\ 0 & (u_a - u_w) & 0 \\ 0 & 0 & (u_a - u_w) \end{bmatrix} \quad [6.92]$$

$$\begin{bmatrix} \delta P_s & 0 & 0 \\ 0 & \delta P_s & 0 \\ 0 & 0 & \delta P_s \end{bmatrix} \quad [6.93]$$

The modified-unsaturated shear strength equation (τ_{mu}) is formulated by adding the contributions of matric suction, denoted as $(u_a - u_w) \tan(\phi^b)$, and swelling stress, denoted as $P_s \tan(\phi^s)$, into the saturated shear strength constitutive relation initially established by Terzaghi (1936). Table 6.28 in Appendix 43 and Table 6.29 in Appendix 44 show the modified-unsaturated shear strength parameters of the studied soils. Equations 6.94 and 6.96 outline the modified-unsaturated shear strength equation, while Equation 6.95 presents the total cohesion. Additionally, Figure 6.199 illustrates the variation in modified-unsaturated shear strength due to changes in matric suction and swelling stress.

$$\tau_{mu} = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) + \delta P_s \tan(\phi^s) \quad [6.94]$$

$$c_t = c' + (u_a - u_w) \tan(\phi^b) + \delta P_s \tan(\phi^s) \quad [6.95]$$

$$\tau_{mu} = c_t + (\sigma_n - u_a) \tan(\phi') \quad [6.96]$$

Where:

- τ_{mu} = modified-unsaturated shear strength [kPa]
- c' = intercept of the extended Mohr-Coulomb failure envelope also called "effective cohesion" [kPa]
- c_t = total cohesion of soil material [kPa]
- $\sigma_n - u_a$ = effective normal stress state [kPa]

- ϕ' = angle of internal friction associated
with the net normal stress state variable [°]
- ϕ^b = angle indicating the rate of increase in shear
strength with respect to a change in matric suction [°]
- ϕ^s = angle indicating the rate of reduction in shear
strength with respect to a change in swelling stress [°]
- δ = δ is the swelling stress fitting parameter characterising
the swelling state of soil material,
for non-swelling soils $\delta = 0$, and for swelling soils, $\delta = -1$.
- $u_a - u_w$ = matric suction [kPa]
- p_s = swelling stress [kPa]

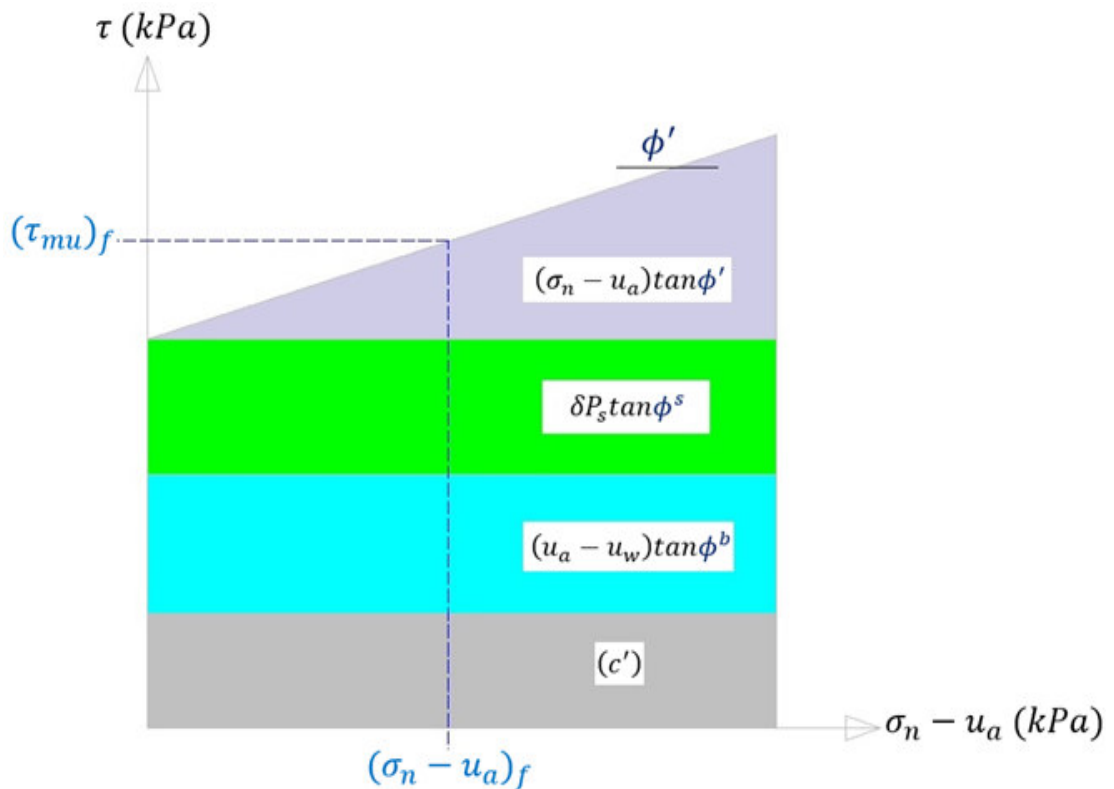


Figure 6.199: Variation of modified-unsaturated shear strength due to the matric suction and swelling stress

The reported τ_{mu} values range from 31 kPa to 1162 kPa at the dry optimum moisture content. However, τ_{mu} exhibits marginal values beyond the dry optimum range. The τ_{mu} values are smaller than the unsaturated shear strength (τ_u) values because the development of swelling stress within the soil material reduces the cohesion among

soil particles, leading to a decrease in shear strength. When soil is compacted at the dry optimum and wet optimum moisture contents, even at the same dry density, it results in different soil fabric orientations (arrangement of soil particles). Consequently, the same soil behaves differently in terms of its strength parameters, volume change. Nevertheless, the marginal τ_{mu} values reported beyond the dry optimum can be improved using suitable soil treatment techniques. Ikeagwuani and Nwonu (2019), Fondjo *et al.* (2021c), and Fondjo *et al.* (2023b) have documented relevant techniques for stabilising expansive soils.

6.7.6.3 Assessment of modified-unsaturated shear strength

The reported responses of shear strength parameters of the tested soils to the geotechnical index properties conducted in this study demonstrate that the overall shear strength parameters of the studied soils are significantly influenced by moisture content, followed by specific gravity and clay content. Moisture in soil serves both as a lubricant and a binding agent among soil particles, thus impacting the structural stability and strength of soil and geologic materials. The results of the responses of shear strength parameters to water content align with reports published by Casagrande and Hirschfeld (1960), Toll (1990), Stark and Duncan (1992), Kong and Tan (2000), Tilgen (2003), Zewdie (2004), Bláhová *et al.* (2013), Su *et al.* (2018), Fondjo *et al.* (2023a), and Fondjo *et al.* (2024). Therefore, the evaluation of the proposed modified-unsaturated shear strength in this study is conducted through a comparative approach with previous constitutive relations of the unsaturated shear strength equation established by Fredlund and Rahardjo (1993), Garven and Vanapalli (2006), and Bulolo and Leong (2019) with reference to water content, using data obtained from 60 tested soil specimens. Figures 6.200 to 6.203 depict shear strength graphs as function of water content for the tested soils BFS, WIS, WES, and BES, respectively. All the shear strength constitutive relations presented in the graphs exhibit a strong exponential correlation with water content and demonstrate a significant reduction in shear strength magnitude as water content increases within the soil. The graphs illustrating the constitutive relations for unsaturated shear strength (τ_u) developed by Fredlund and Rahardjo (1993), Garven and Vanapalli (2006), and Bulolo and Leong (2019) appear very similar, with negligible disparities. The similarities stem from the utilisation of two stress state variables, the effective normal

stress ($\sigma_n - u_a$) and matric suction ($u_a - u_w$) to formulate the unsaturated shear strength. Nonetheless, the observed marginal discrepancies in the graphs may originate from the different types of fitting parameters related to the contribution of matric suction, as presented in Table 6.30.

Table 6.30: Matric suction fitting parameters

References	Matric suction fitting parameters
Fredlund and Rahardjo (1993)	$(u_a - u_w) \tan \phi^b$
Garven and Vanapalli (2006)	$(u_a - u_w)(\theta^k) \tan \phi'$
Bulolo and Leong (2019)	$(u_a - u_w)(\chi S_r) \tan \phi'$

The values of the modified-unsaturated shear strength (τ_{mu}) engineered in this research are close to the values of the previous unsaturated shear strength relation (τ_u) at a lower water content. However, as the water content increases, the discrepancies between the modified-unsaturated shear strength equation and the previous unsaturated shear strength equations also increase. The τ_{mu} graph is located below the τ_u graphs due to the significant reduction in shear strength caused by the impact of swelling stress on the tested soils. The proposed modified-unsaturated shear strength equation describes a non-linear approximation. This trend aligns with the study reported by Fredlund *et al.* (1996) and Lamborn (1986), which indicates that shear strength for unsaturated soils is often represented using either bilinear or linear approximations, with non-linear approximation being more realistic and accurate. In general, the observed discrepancies between the modified-unsaturated shear strength and the previous equations are primarily attributed to the reduction effect of the swelling stress on the modified-unsaturated shear strength of expansive soil. Additionally, it is important to note that not all unsaturated soils exhibit expansive behaviour. However, when unsaturated soil does show a swelling behaviour, the previous unsaturated shear strength equations might overestimate the shear strength of expansive unsaturated soils because previous equations usually do not account for the influence of swelling stress. The modified-unsaturated shear strength equation serves as an efficient tool for accurately describing the shear strength of heaving soil.

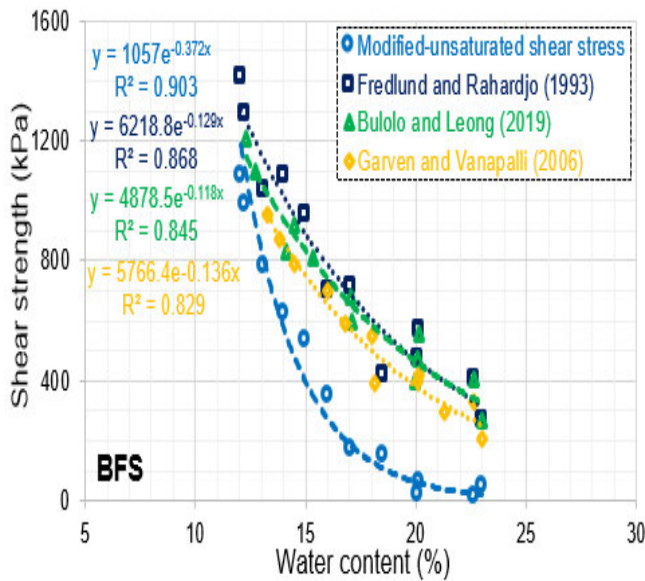


Figure 6.200: Shear strength vs water content for BFS

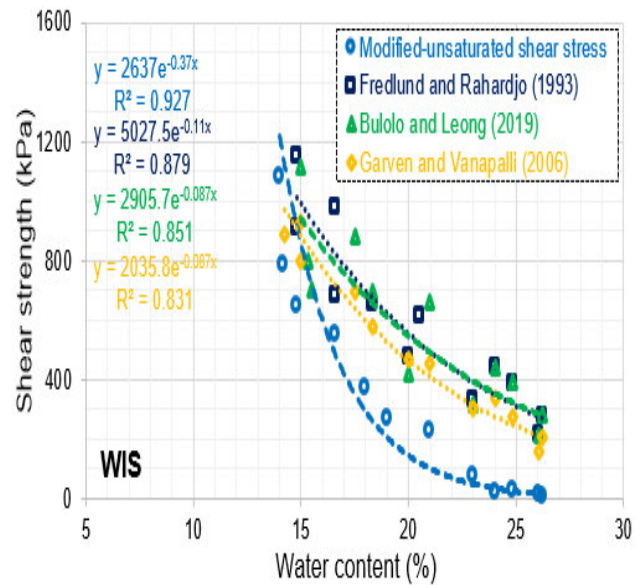


Figure 6.201: Shear strength vs water content for WIS

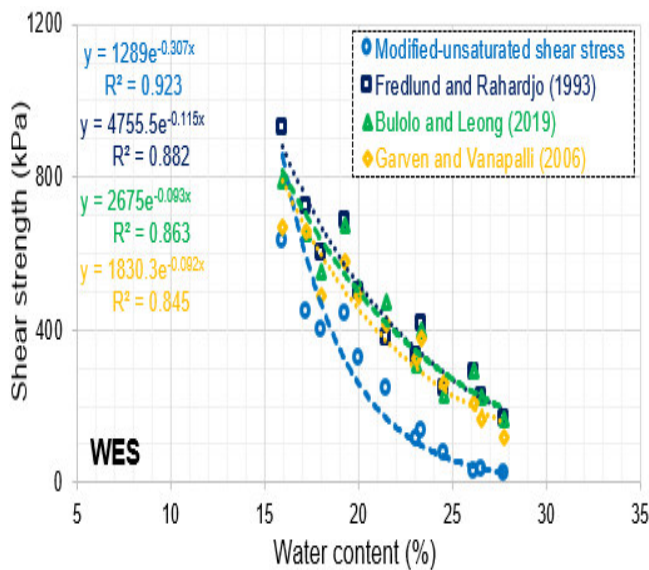


Figure 6.202: Shear strength vs water content for WES

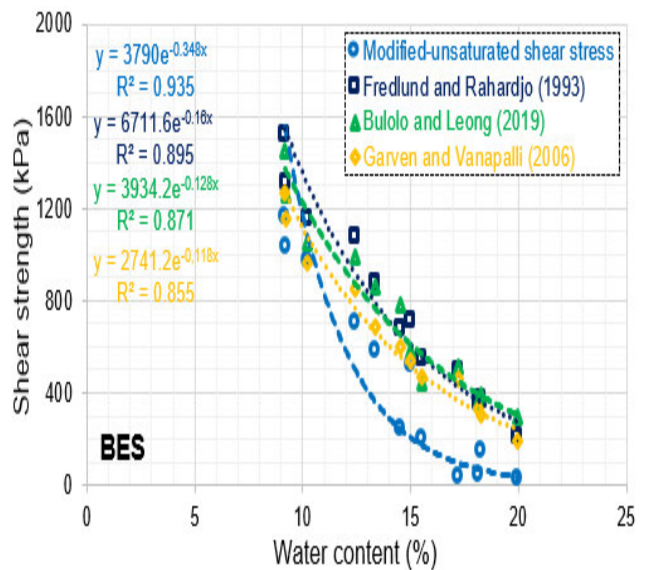


Figure 6.203: Shear strength vs water content for BES

6.7.6.4 Conclusion

In soil mechanics, when unsaturated soils exhibit a swelling behaviour, the induced swelling stress significantly influences the shear strength of the soil material. This section proposed a modified-unsaturated shear strength equation (τ_{mu}) for expansive unsaturated soils. The modified-unsaturated shear strength (τ_{mu}) was established based on the equilibrium of three stress variables; effective normal stress ($\sigma_n - u_a$),

matric suction ($u_a - u_u$), and swelling stress (P_s). The contributions of matric suction and swelling stress are incorporated into the saturated shear strength of soil material. The reported τ_{mu} values range from 30.72 kPa to 1161.83 kPa at the dry optimum. However τ_{mu} exhibits marginal values beyond the dry optimum. These τ_{mu} values are smaller than the unsaturated shear strength (τ_u) values due to the development of swelling stress within the soil material, which reduces cohesion among soil particles and consequently weakens the shear strength. The assessment of τ_{mu} was conducted by comparing it with three established precedent constitutive relations of τ_u by Fredlund and Rahardjo (1993), Garven and Vanapalli (2006), and Bulolo and Leong (2019). The response graphs of τ_u and τ_{mu} exhibit similar shapes, both showing a decrease in shear strength with the addition of water. However, the τ_{mu} graph consistently lies below the τ_u graphs variations in water content due to the reduced effect of swelling stress on shear strength. Previous unsaturated shear strength constitutive relations might overestimate the shear strength magnitude of expansive unsaturated soils, as previous equations often neglect the stress state induced by swelling stress in these soils. Considering the influence of swelling stress on the shear strength of expansive unsaturated soils in the design enhances the durability of constructions and reduces the costs of the assessment and repair of damages in the structure. The marginal values of τ_{mu} beyond the dry optimum can be improved using appropriated soil treatment techniques.

6.8 Geotechnical predictive models

In this section, geotechnical predictive models were developed. Models for soil mineralogy, models for compaction features, models for soil suction, and models for unsaturated shear strength parameters. The design of geotechnical models was meticulously presented in Chapter 5, section 5.13. The values of the applied mathematical symbols in the models denoted the correlation coefficients, intercepts, and Box-Cox transformation were derived from the regression analysis report provided by the NCSS 12 (Statistical analysis software for researchers).

6.8.1 Geotechnical predictive models for soil mineralogy

6.8.1.1 Estimated model

Table 6.5 in Appendix 5 describes the analysis of the response of smectite content to soil properties. The PI, FSR, G_s, and FSI described the best subsets. Table 6.6 in Appendix 6 illustrates the evaluation of the response of silica content to soil features. The PI, FSR, Gravel, and FSI depict the best subsets. Table 6.7 in Appendix 6 describes the analysis of the response of K-feldspar content to soil properties. The PI, FSR, G_s, and FSI demonstrate the best subsets. Table 6.8 in Appendix 7 shows the assessment of the response of plagioclase content to soil features. The LL, LS, G_s, and FSI display the best subsets. Tables 6.1, 6.2, and 6.3 are utilised as correlation matrices to perform the analysis. The models to evaluate the content of smectite, silica, K-feldspar, and plagioclase are given by Equations 6.97 to 6.100, respectively. Table 6.31 in Appendix 45 presents the model summary and coefficients.

$FSI \geq 35\%$ and $FSR \geq 1.10$

$$\text{Smectite} = +\xi_0 + \xi_1 PI + \xi_2 FSR - \xi_3 G_s - \xi_4 FSI \quad [6.97]$$

$$\text{Silica} = +\eta_0 - \eta_1 PI - \eta_2 FSR - \eta_3 \text{Gravel} + \eta_4 FSI \quad [6.98]$$

$$\text{K-feldspar} = +\mu_0 + \mu_1 PI + \mu_2 FSR - \mu_3 G_s - \mu_4 FSI \quad [6.99]$$

$$\text{Plagioclase} = -\alpha_0 - \alpha_1 LL + \alpha_2 LS + \alpha_3 FSI + \alpha_4 G_s \quad [6.100]$$

Where:

Smectite = smectite content [%]

Silica = silica content [%]

K-feldspar = K-feldspar content [%]

Plagioclase = plagioclase content [%]

PI = plasticity index [%]

LL = liquid limit [%]

FSR, FSI = free swell ratio, free swell index [%]

LS = linear shrinkage [%]

G_s = specific gravity

Gravel = gravel content [%]

LS = linear shrinkage [%]

$\xi_0, \eta_0, \mu_0, \alpha_0$ = intercepts

$\xi_i, \eta_i, \mu_i, \alpha_i$ = multivariate coefficients, $i = 1, 2, \dots, 4$

6.8.1.2 Model validation

The assessment of the models is conducted by evaluating the p-value of the term selected for each model, comparing the measured values to the predicted values, and analysing the pattern of standardised residuals (errors). The primary criterion for selecting relevant covariates is the p-value. A hypothesis test is performed to determine the coefficient of each covariate. When the p-value is higher than 0.05, the coefficient is considered to be zero, and the covariate is deemed insignificant and excluded from the model. Table 6.32 in Appendix 45 displays the p-values of each model. The p-values of the selected soil parameters are smaller than 0.05. The comparison of measured values to predicted values reveals a strong correlation, as depicted by scatter plots that are closely aligned in a 1:1 pattern, showing marginal disparities for all the models. Figure 6.204 illustrates the relationship between measured smectite content and predicted smectite content, yielding an R^2 value of 99.91 %. Likewise, Figure 6.205 displays the correlation between measured silica and predicted silica, resulting in an R^2 value of 91.43 %. Figure 6.206 demonstrates a strong relationship between measured K-feldspar and predicted K-feldspar, yielding an R^2 value of 95.13%. Additionally, Figure 6.207 depicts the correlation between measured plagioclase and predicted plagioclase, showing an R^2 value of 95.18%.

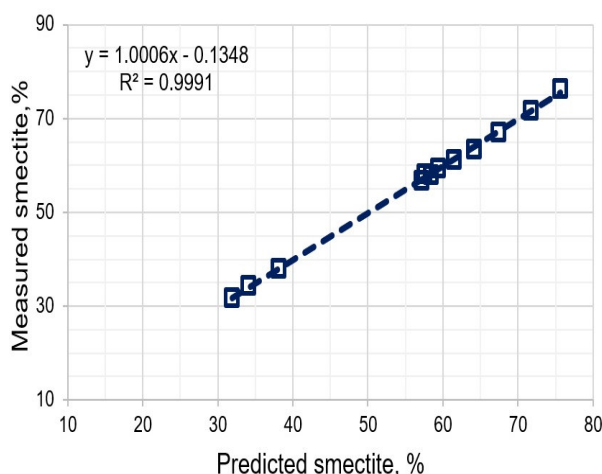


Figure 6.204: Measured vs predicted values (smectite)

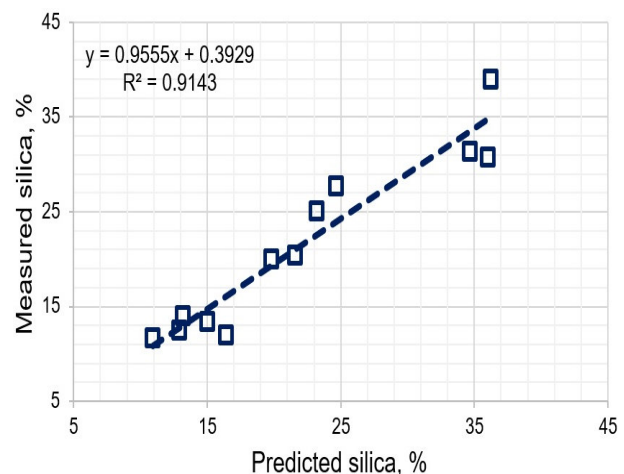


Figure 6.205: Measured vs predicted values (silica)

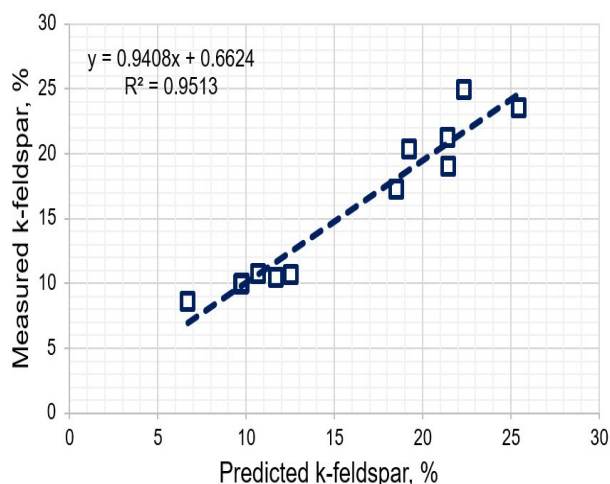


Figure 6.206: Measured vs predicted values (K-feldspar, %)

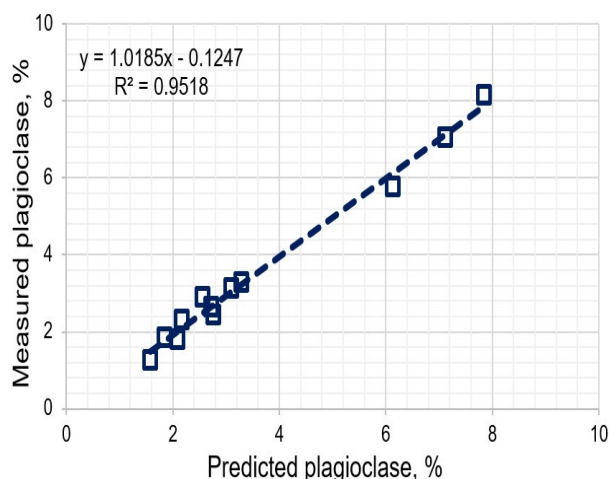


Figure 6.207: Measured vs predicted values (plagioclase)

Investigating the residuals (errors) is essential to determine the accuracy of the model. If the residuals exhibit any specific pattern, it indicates that the model has not accounted for all systematic data. For the model to perform efficiently, residuals should appear irregular. The residuals must adhere to a typical distribution with a mean of zero and a constant difference (Goyal *et al.* 2006, Montgomery *et al.* 2021). Papanastasiou *et al.* (2007) and Grivas and Chaloulakou (2006) employed the coefficient of determination between measured and predicted values to assess model efficiency. Roy *et al.* (2011) utilised these approaches to evaluate the accuracy of the model. Figures 6.208 and 6.209 depict the standardised residuals of smectite and silica content, respectively. The residuals follow a typical distribution with a mean of zero and constant variance. Likewise, Figures 6.210 and 6.211 illustrate the standardised residuals of K-feldspar and plagioclase content, respectively. The residuals exhibit a typical distribution with a mean of zero and constant variance. These results hold statistical significance at a 1 % level of importance. The fraction of smectite, silica, K-feldspar, and plagioclase can be estimated with an acceptable level of precision.

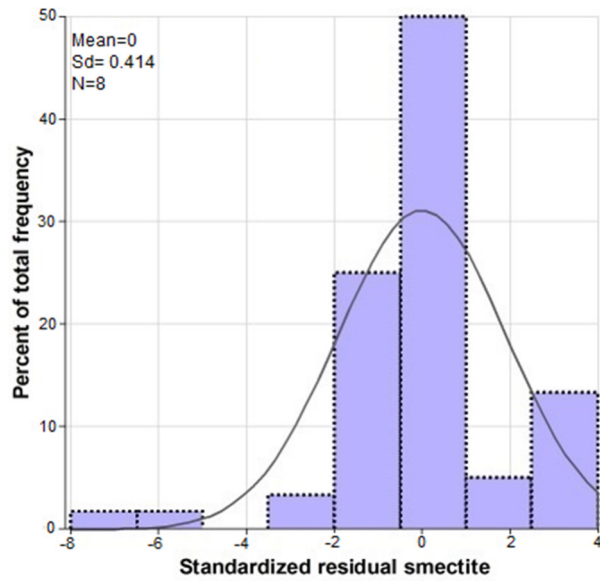


Figure 6.208: Frequency vs standardised residual (smectite)

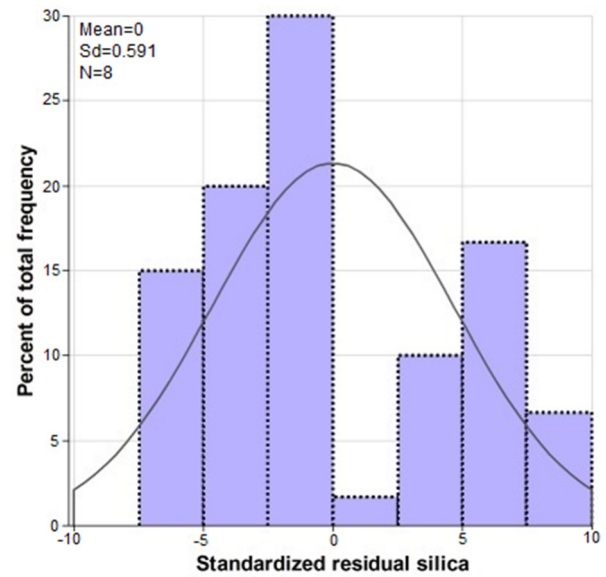


Figure 6.209: Frequency vs standardised residual (silica)

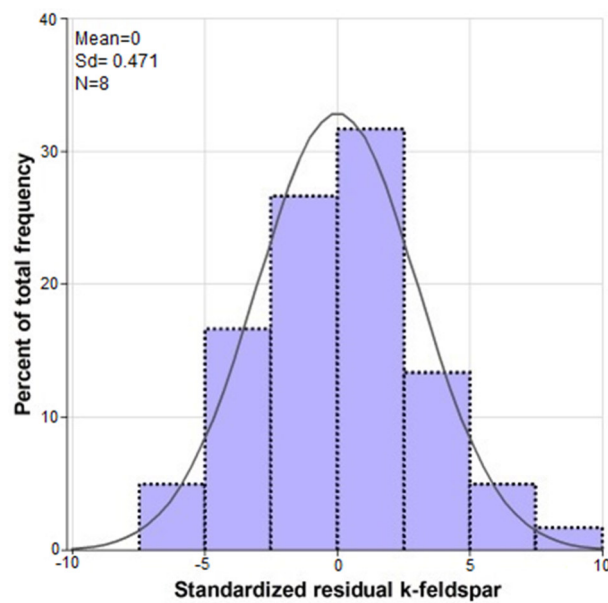


Figure 6.210: Frequency vs standardised residual (K-feldspar)

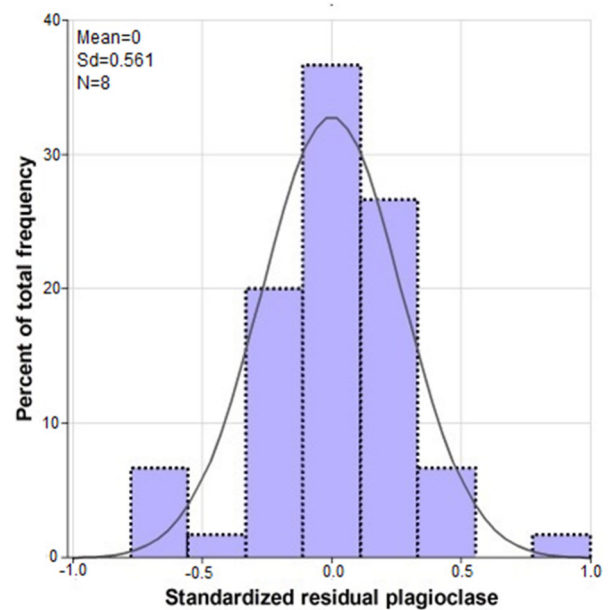


Figure 6.211: Frequency vs standardised residual (plagioclase)

6.8.2 Geotechnical predictive models for compaction features

6.8.2.1 Estimated model

Sections 6.3.3 through 6.3.6 describe response of compaction features to consistency limits, fine-grained content, sand content, gravel content, G_s , C_c , and C_u of soil samples. Consistency limits, fine content, gravel content, sand content, and G_s exhibit a strong correlation with compaction features (OMC and γ_{dmax}). The model's covariates include LL, fine-grained content, sand content, gravel content, and G_s . Tables 6.1, 6.2, and 6.10 in Appendix 8 present the correlation matrices for the analysis. Equation 6.101 describes the model for predicting OMC, and Equation 6.102 outlines the model for predicting γ_{dmax} . Table 6.33 in Appendix 45 displays the model summary and coefficients.

$$OMC = +\xi_0 - \xi_1 LL + \xi_2 Fine + \xi_3 Sand + \xi_4 Gravel - \xi_5 G_s \quad [6.101]$$

$$\gamma_{dmax} = +\mu_0 - \mu_1 LL + \mu_2 Fine + \mu_3 Sand + \mu_4 Gravel - \mu_5 G_s \quad [6.102]$$

Where:

OMC = optimum moisture content [%]

γ_{dmax} = maximum dry unit weight [$\text{kN} \cdot \text{m}^{-3}$]

LL = liquid limit [%]

Fine = fine content [%]

Sand = sand content [%]

Gravel = gravel content [%]

G_s = specific gravity

ξ_0, μ_0 = intercepts

ξ_i, μ_i = multivariate coefficients, $i = 1, 2, \dots, 5$

6.8.2.2 Model validation

The model validation involves comparing the measured with predicted values, using other soil datasets for comparison, and contrasting the current models with those of models presented by Shien *et al.* (2015), Firomsa and Quezon (2019), and Hussain and Atalar (2020), as shown in Table 6.34 in Appendix 46. Table 6.35 in Appendix 46 displays the p-values of each model. Figure 6.212 presents a plot depicting the measured OMC values obtained from compaction tests against the predicted values

of OMC. The scatter plot closely follows the 1:1 trend line, with an R^2 value of 97.53 %, describing a strong correlation between the measured and predicted values of OMC. Additionally, Figure 6.213 illustrates the comparison between measured γ_{dmax} values and predicted values. The scatter plot closely follows the 1:1 trend line, exhibiting an R^2 value of 99.30 % and demonstrating a strong correlation between the predicted and measured values of γ_{dmax} .

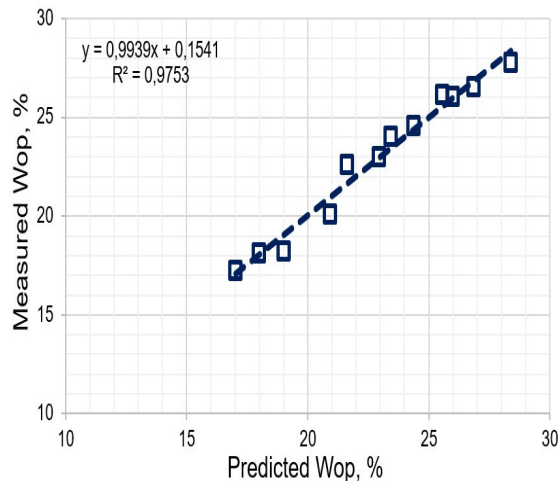


Figure 6.212: Measured vs predicted values (OMC)

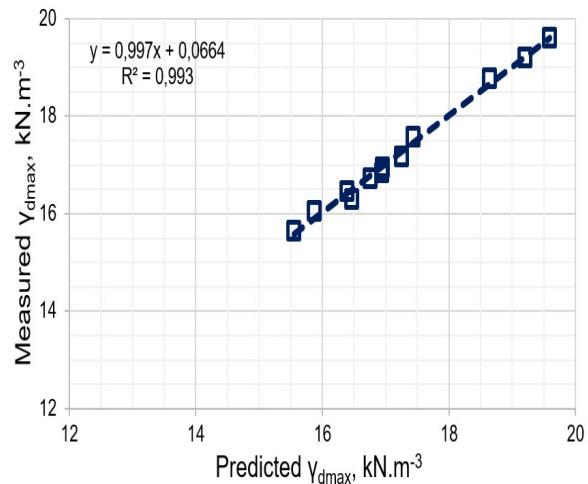


Figure 6.213: Measured vs predicted values (γ_{dmax})

Table 6.36 in Appendix 46 displays the absolute error (AE) and percentage error (PE) between the measured and predicted values of compaction features. The maximum AE value for γ_{dmax} is 0.18, and for OMC, 0.97. The maximum PE value for γ_{dmax} is 1.10 %, and for OMC, 4.33 %. Based on these observations, there exists a strong relationship between the measured and predicted values. Additionally, the validation of the proposed models is performed using other soil datasets to verify the maximum absolute error during cross-validation. The experimental datasets provided by Shien *et al.* (2015) in Table 6.37 in Appendix 47 are chosen. Table 6.38 in Appendix 47 illustrates the validation using data from Shien *et al.* (2015). Samples 1, 2, 4, 5, 7, 8, and 9 concurred with the predicted OMC values, with AE values below 1 (AE < 1). However, for soils 3 and 6, the predicted OMC values exhibit AE values greater than 1 (AE > 1). Samples 1, 2, 3, 4, 5, 6, 7, and 8 agreed with the predicted γ_{dmax} values, all with AE values below 1 (AE < 1). However, sample 9's predicted γ_{dmax} value presents an AE value greater than 1 (AE > 1). The variability in soil material can account for these observed disparities. Fondjo and Dzogbewu (2019) reported that the discrepancies between measured and predicted values could also stem from inherent

approximations in multivariate analysis theories. Figure 6.214 shows the relationship between the measured and predicted OMC values obtained from the model proposed in this study and those described by the model of Firomsa and Quezon (2019). The model presented in this study exhibits greater accuracy, with an R^2 value of 97.53 %, compared to the model by Firomsa and Quezon (2019), with an R^2 value of 94.39 %. Further, Figure 6.215 illustrates the correlation between the measured and predicted γ_{dmax} values from the model developed in this research work and the γ_{dmax} values obtained from the model by Firomsa and Quezon (2019). The model from this research demonstrates greater accuracy, with an R^2 value of 99.70 %, as opposed to the model by Firomsa and Quezon (2019) with an R^2 value of 92 %. Both Figures 6.214 and 6.215 indicate some discrepancies between the models presented in this research and those by Firomsa and Quezon (2019). This trend can be explained by the different types of covariates used in the models. Models by Firomsa and Quezon (2019) used LL and PL, whereas the models proposed in this study used LL, soil gradation, and Gs.

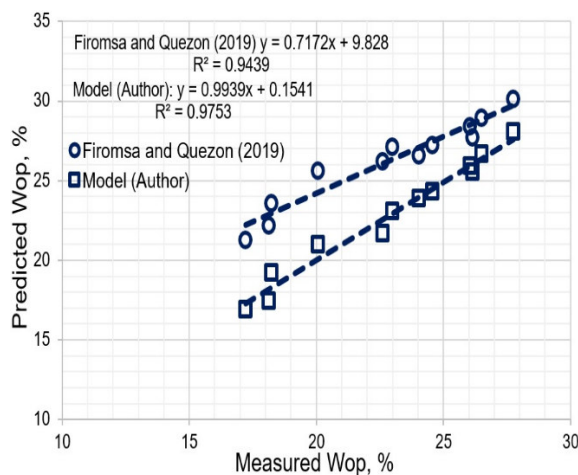


Figure 6.214: Measured vs predictive values OMC Firomsa and Quezon (2019)

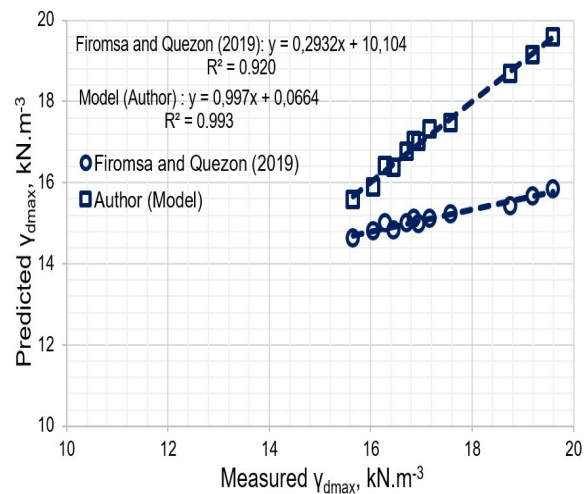


Figure 6.215: Measured vs predictive values (γ_{dmax}) Firomsa and Quezon (2019)

Figure 6.216 depicts the correlation between the measured and predicted values of OMC provided by the model from this study and the OMC values presented by Hussain and Atalar (2020). The model developed in this research work is more accurate, with an R^2 value of 97.53 %, compared to the model by Hussain and Atalar (2020), with an R^2 value of 94.22 %. Figure 6.217 illustrates the correlation between the measured and predicted values of γ_{dmax} obtained from the model developed in this research and the γ_{dmax} values obtained from the model by Hussain and Atalar (2020). The model

in this study demonstrates greater accuracy with an R^2 value of 99.30 %, compared to the model by Hussain and Atalar (2020), with an R^2 value of 94.61 %. Both Figures 6.216 and 6.217 highlight discrepancies between the predicted values from Hussain and Atalar (2020) and the predicted values from the model in this study. The differences in the number and types of covariates used account for these disparities between the models. The models by Hussain and Atalar (2020) used the LL, whereas the models in this research utilised soil gradation, LL, and G_s . The models proposed in this study for predicting the compaction features appear more precise.

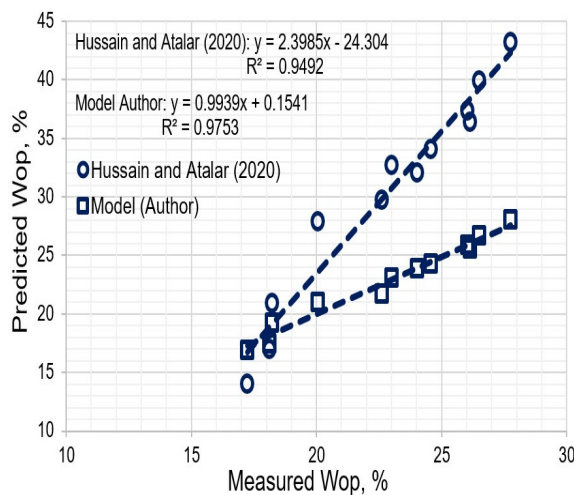


Figure 6.216: Measured vs predictive values (OMC) Hussain and Atalar (2020)

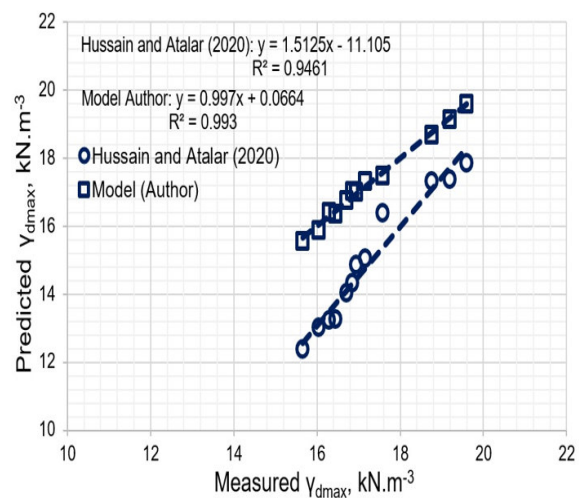


Figure 6.217: Measured vs predictive values (γ_{dmax}) Hussain and Atalar (2020)

6.8.3 Geotechnical predictive models for soil suctions

6.8.3.1 Estimated models

Sections 6.4.3 and 6.4.4 describe the relationship between matric suction and soil properties, including clay content, LL, G_s , FSI, LS, void ratio, FSR, and γ_d . Figure 6.218 illustrates the relationship between the total suction and matric suction obtained from the suction test using the filter paper technique. The total suction increases as the matric suction increases. The scatter-plotted data exhibit a strong positive correlation between total and matric suction with an R^2 value of 99.75 %. As a result of this strong correlation, the influence of soil properties on total suction is similar to their impact of on matric suction.

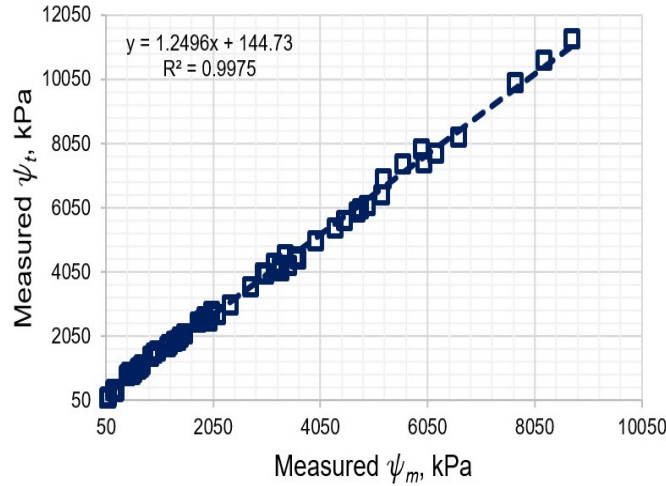


Figure 6.218: Measured total suction vs measured matric suction

The stepwise multivariate application reduces the number of covariates and enhances the accuracy of the model. A p-value higher than 0.05 induces a covariate coefficient of 0, and the covariate is considered insignificant and excluded from the model. Table 6.40 in Appendix 48 presents the p-values of the two models. The covariates include water content, fine-grained content, γ_d , void ratio, G_s , and LL. Table 6.12 in Appendix 9, Table 6.13 in Appendix 10, and Table 6.39 in Appendix 48 represent the correlation matrices used in the analysis. Equations 6.103 and 6.104 describe the models for predicting matric and total suction, respectively. Table 6.41 in Appendix 49 provides the model summary and coefficients.

$$[\psi_m]^\lambda = +\xi_0 - \xi_1 W + \xi_2 \text{Fine} - \xi_3 \gamma_d - \xi_4 e + \xi_5 G_s - \xi_6 LL \quad [6.103]$$

$$[\psi_t]^\lambda = +\eta_0 - \eta_1 W + \eta_2 \text{Fine} - \eta_3 \gamma_d - \eta_4 e - \eta_5 G_s - \eta_6 LL \quad [6.104]$$

Where:

ψ_t = total suction [kPa]

ψ_m = matric suction [kPa]

W = water content [%]

γ_d = dry unit weight [$\text{kN} \cdot \text{m}^{-3}$]

Fine = fine content [%]

G_s = specific gravity

e = void ratio

LL = liquid limit [%]

λ = Box-Cox transformation coefficient

ξ_0, η_0 = Intercepts

ξ_i, η_i = multivariate coefficients, $i = 1, 2, \dots, 6$

6.8.3.2 Models validation

Figure 6.219 depicts graph data representing the measured matric suction values and the predicted matric suction values recorded on 60 compacted specimens at different water content. The variations in the measured matric suction closely mirror the changes in the predicted matric suction. The scatter-plotted data reveal minor discrepancies. The relationship between the measured and predicted matric suction values exhibits a strong correlation, with an R^2 value of 98.57 %. Similarly, Figure 6.220 displays the plotted data of measured total suction values and predicted total suction values obtained from 60 compacted soil specimens with different moisture content. The scatter-plotted data show minor disparities, and both measured and predicted total suction values demonstrate a strong correlation, with an R^2 value of 98.87 %.

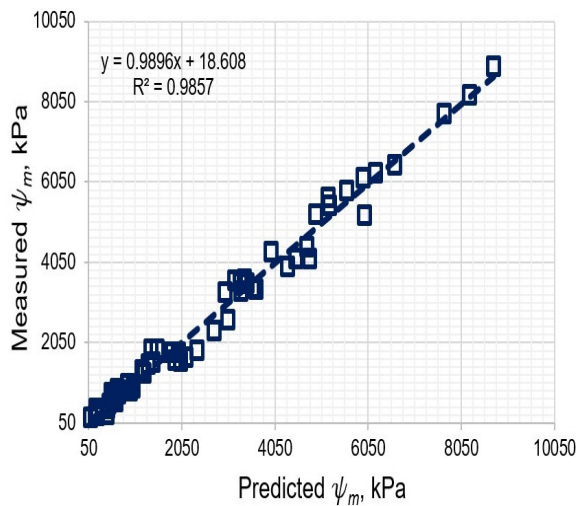


Figure 6.219: Measured vs predicted values (matric suction)

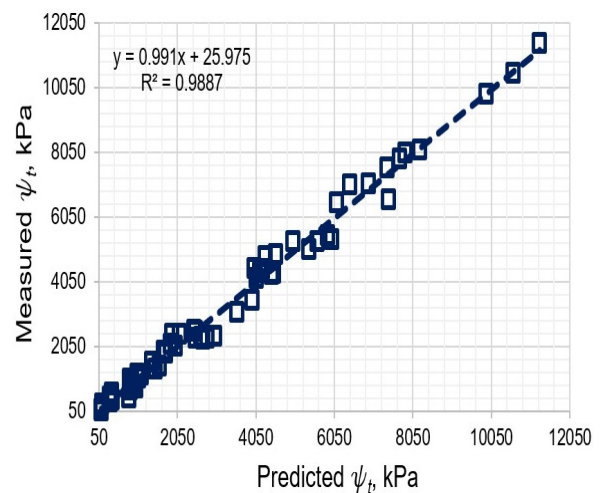


Figure 6.220: Measured vs predicted values (total suction)

Analysing the standardised residuals (errors) is critical in assessing the efficiency of a model. If the standardised residual shows any pattern, it indicates that the model is not accounting for all systematic data. For optimum model performance, residuals should be irregular and follow a standard distribution with a mean of zero and constant difference (Goyal *et al.* 2006, Montgomery *et al.* 2021). Grivas and Chaloulakou

(2006) and Papanastasiou *et al.* (2007) utilised the coefficient of determination between measured and predicted values to evaluate the efficiency of the model. Roy *et al.* (2011) implemented these methods to validate the effectiveness of predictive models.

Figures 6.221 and 6.222 illustrate the histograms of standardised residuals for matric suction and total suction, respectively. The residuals are distributed in a distinct manner with zero means. The reported determination coefficients for Equations 6.103 and 6.104 are R^2 value of 98.56 % and R^2 value of 98.84 % respectively, and are statistically relevant at a 1 % level of significance. The soil suction within the compacted expansive soil is influenced not only by moisture content but also by soil property indexes as presented in the models. This model proves to be an efficient tool for evaluating total and matric suction with acceptable accuracy.

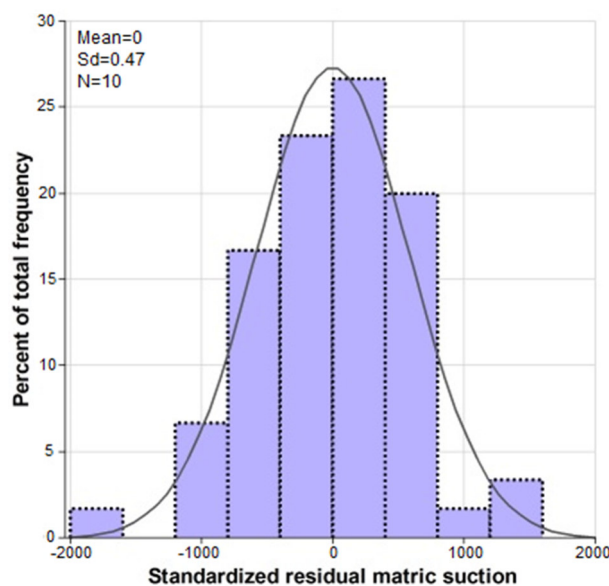


Figure 6.221: Frequency vs standardised residual (matric suction)

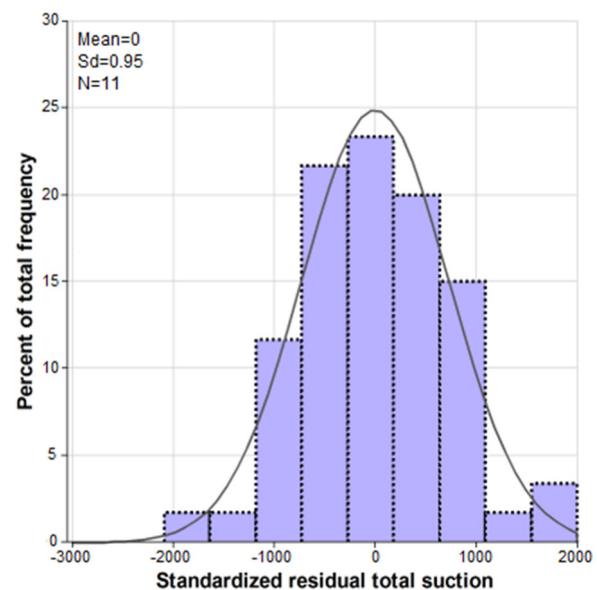


Figure 6.222: Frequency vs standardised residual (total suction)

6.8.4 Geotechnical predictive models for angles (ϕ' , ϕ^b , ϕ^s)

6.8.4.1 Estimated model (ϕ')

Section 6.7.5 thoroughly discussed the response of ϕ' to soil index properties. A stepwise regression analysis is conducted using the NCSS 12 program, based on correlation matrices in Tables 6.1, 6.2, Table 6.22 in Appendix 39, and Table 6.23 in Appendix 40. The covariate coefficient that exhibits a p-value smaller than 0.05 is selected for the model, while a covariate coefficient higher than 0.05 is considered as 0 and excluded from the model. Table 6.43 in Appendix 50 presents the p-values. Consequently, the soil parameters including water content, specific gravity (G_s), PI , LL , clay content, and fine-grained content are deemed satisfactory for the model. Table 6.42 in Appendix 49 provides the summary and coefficients in the model presented in Equation 6.105.

$$(\phi')^\lambda = -\xi_0 - \xi_1 W + \xi_2 G_s + \xi_3 PI - \xi_4 LL - \xi_5 Clay - \xi_6 Fine \quad [6.105]$$

Where:

- ϕ' = angle of saturated shear strength [°]
- W = gravimetric water content [%]
- G_s = specific gravity
- PI = plasticity index [%]
- LL = liquid limit [%]
- $Fine$ = fine content [%]
- $clay$ = clay content [%]
- λ = Box-cox transformation = 0.5
- ξ_0 = intercepts
- ξ_i = multivariate coefficients, $i = 1, 2, \dots, 6$

6.8.4.2 Model validation (ϕ')

Figure 6.223 presents graph data depicting the measured friction angle and predicted friction angle values obtained from 60 compacted specimens with different water content. The variations in the measured friction angle closely resemble the predicted friction angle values. The scatter-plotted data reveals minimal disparities, and the

relation between the measured and predicted friction angle values describes a strong correlation, with a determination coefficient R^2 of 92.48 %.

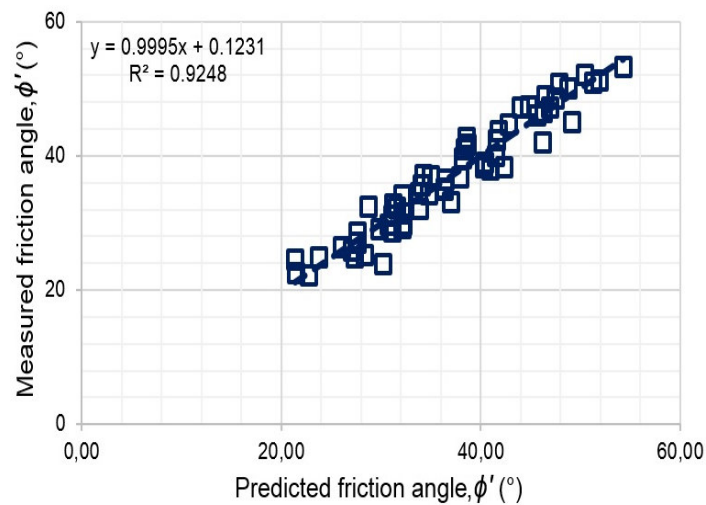


Figure 6.223: Measured vs predicted values of (ϕ')

The present model is compared with two previous models proposed by Chen *et al.* (2012) and Roy and Dass (2014), as shown in Table 6.44 in Appendix 50. Figures 6.224 and 6.225 show the plotted data of the measured and predicted friction angles obtained through the two models. The model by Chen *et al.* (2012) bears a good correlation, while the model by Roy and Dass (2014) exhibits a weak correlation. The observed differences may stem from the soil materials used to develop these models. The model by Chen *et al.* (2012) was developed for compacted expansive soil, whereas the model by Roy and Dass (2014) is based on cohesionless and non-swelling soil materials. The predicted values of the model by Chen *et al.* (2012) agree with the model developed in this study; however, the present model is more accurate and portrays a strong correlation. The precision of the model proposed in this study may come from the inclusion of a higher number of covariates. The models by Chen *et al.* (2012) and Roy and Dass (2014) incorporated 1 to 2 covariates. The models proposed by Chen *et al.* (2012) and Roy and Dass (2014) were derived from 4 and 10 soil specimens, respectively, whereas the present model is based on data from 60 soil specimens. Lastly, the model developed in this thesis is validated by the model proposed by Chen *et al.* (2012).

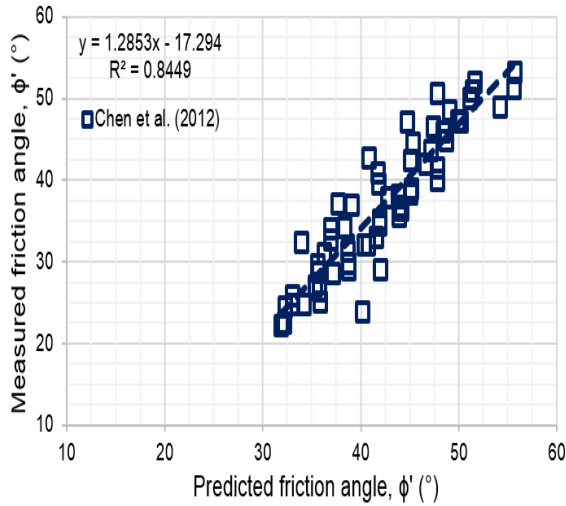


Figure 6.224: Measured vs predicted values of (ϕ') Model Chen *et al.* (2012)

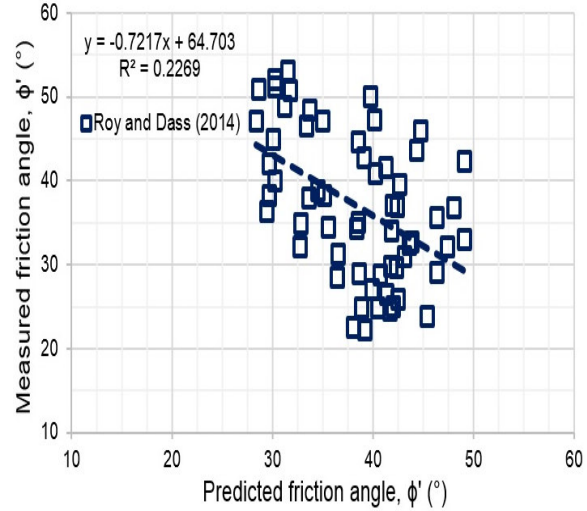


Figure 6.225: Measured vs predicted values of (ϕ') Model Roy and Dass (2014)

6.8.4.3 Estimated model (ϕ^b)

Section 6.7.5 meticulously described the relationship between the angle indicating the rate of increase in shear strength with respect to changes in matric suction (ϕ^b) and geotechnical index properties. The regression analysis is conducted based on the correlation matrices in Tables 6.1, 6.2, Table 6.10 in Appendix 8, Table 6.22 in Appendix 39, and Table 6.23 in Appendix 40. Additionally, the p-value is employed to select the model's covariates. When the p-value of the covariate coefficient is higher than 0.05, the coefficient is considered as 0, indicating that the covariate is irrelevant to the model. Table 6.43 in Appendix 50 shows the p-values. The covariates of the model consist of water content, specific gravity, fine-grained content, LL, and dry unit weight. The relation in Equation 6.106 describes the designed model, and Table 6.42 in Appendix 49 presents the model summary and coefficients.

$$(\phi^b)^\lambda = +\eta_0 - \eta_1 W + \eta_2 G_S - \eta_3 \text{Fine} + \eta_4 LL - \eta_5 \gamma_d \quad [6.106]$$

Where:

ϕ^b = angle indicating the rate of increase in shear strength with respect to a change in matric suction $[\circ]$

W = gravimetric water content [%]

G_S = specific gravity

LL = liquid limit [%]

$Fine$ = fine content [%]

γ_d = dry unit weight [$\text{kN} \cdot \text{m}^{-3}$]

λ = Box-cox transformation

η_0 = intercepts

η_i = multivariate coefficients, $i = 1, 2, \dots, 5$

6.8.4.4 Model validation (ϕ^b)

Figure 6.226 portrays the plotted data, showing the measured angle indicating the increased rate of shear strength with respect to a change in matric suction (ϕ^b), against the predicted values of ϕ^b obtained from compacted soil specimens at different water content. The scatter-plot data closely aligns with the trend line 1:1, showing a strong correlation between the measured and predicted values of ϕ^b , with a determination coefficient R^2 of 96.28 %.

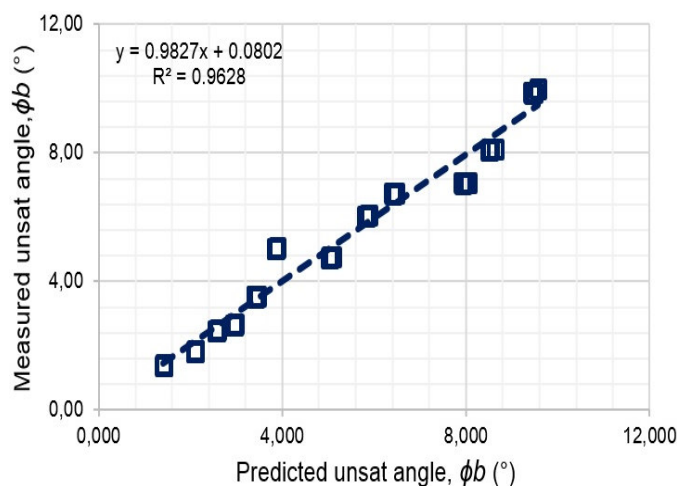


Figure 6.226: Measured vs predicted values of (ϕ^b)

The assessment of residuals (errors) is essential in determining the accuracy of a model. If the residuals show any pattern, it indicates that the model has not considered all the systematic data. For effective model execution, residuals should be irregular. The residuals must follow the specific distribution with a zero mean and constant difference (Goyal *et al.* 2006, Montgomery *et al.* 2021). Grivas and Chaloulakou (2006) as well as Papanastasiou *et al.* (2007) utilised the determination coefficient between predicted and measured values to assess the performance of the model. Nevertheless, Roy *et al.* (2011) applied this theory to assess the model's accuracy.

Figure 6.227 illustrates the standardised residuals of the angle indicating the increased rate of shear strength with respect to changes in matric suction (ϕ^b). The reported predicted determination coefficient for Equation 6.106 is 96.05 %, which holds statistical significance at 1 % level of importance. The present model demonstrates the capability to estimate ϕ^b with an acceptable level of accuracy.

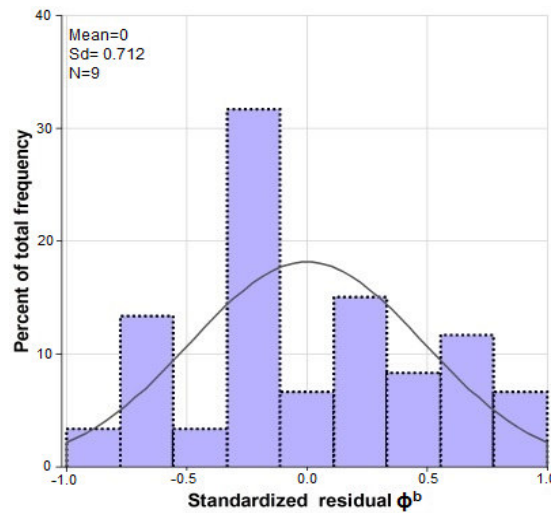


Figure 6.227: Frequency vs standardised residual (ϕ^b)

6.8.4.5 Estimated model (ϕ^s)

The response of the angle indicating the rate of decrease in shear strength with respect to changes in swelling stress to geotechnical index properties is meticulously presented in section 6.7.5. The regression analysis is performed based on the correlation matrices provided in Table 6.1, Table 6.2, Table 6.3, Table 6.28 in Appendix 43, and Table 6.29 in Appendix 44. Additionally, the p-value plays a critical role in selecting the model covariate. When the p-value of the covariate coefficient exceeds 0.05, the coefficient is considered as 0. Consequently, the covariate is deemed insignificant and excluded from the model. Table 6.43 in Appendix 50 displays p-values. The present model utilises the following covariates: water content, G_s , PI , FSI , and fine-grained content. Table 6.42 in Appendix 49 shows the model summary and coefficients of the model as described in Equation 6.107.

$$\phi^s = -\alpha_0 - \alpha_1 W + \alpha_2 G_s + \alpha_3 PI - \alpha_4 FSI - \alpha_5 Fine \quad [6.107]$$

Where:

ϕ^s = angle indicating the rate of decrease in shear strength with respect to a change in swelling stress [°]

W = gravimetric water content [%]

G_s = specific gravity

PI = plasticity index [%]

FSI = free swell index [%]

$Fine$ = fine content [%]

α_0 = intercepts

α_i = multivariate coefficients, $i = 1, 2, \dots .5$

6.8.4.6 Model validation (ϕ^s)

Figure 6.228 presents graph data depicting the measured ϕ^s and predicted ϕ^s values obtained from 60 compacted specimens at different water content. The variation observed in measured ϕ^s closely aligns with the predicted ϕ^s values. The scatter-plotted data exhibits minimal disparities, and the relation between the measured and predicted values of the ϕ^s describes a strong correlation with a determination coefficient of 98.04 %.

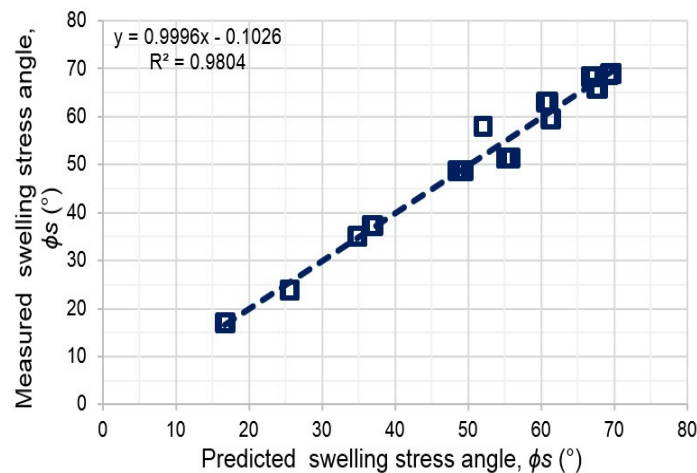


Figure 6.228: Measured vs predicted values of (ϕ^s)

The assessment of residuals (errors) is fundamental in determining the accuracy of a model. If the residuals demonstrate any arrangement, it suggests that the model has not considered all systematic data. For effective model execution, residuals should be irregular. The residuals should adhere to a typical distribution with a zero mean and constant difference (Goyal *et al.* 2006, Montgomery *et al.* 2021). Grivas and

Chaloulakou (2006) as well as Papanastasiou *et al.* (2007) utilised the determination coefficient between predicted and measured values to evaluate the model performance. However, Roy *et al.* (2011) implemented these theories to evaluate the precision of the model. Figure 6.229 displays the standardised residuals of the angle indicating the rate of decrease in shear strength with respect to change in swelling stress (ϕ^s). The reported predicted determination coefficient for Equation 1 is 97.73 %, and it holds statistical significance at 1 % level of importance. This present model demonstrates the capability to estimate ϕ^s with acceptable precision.

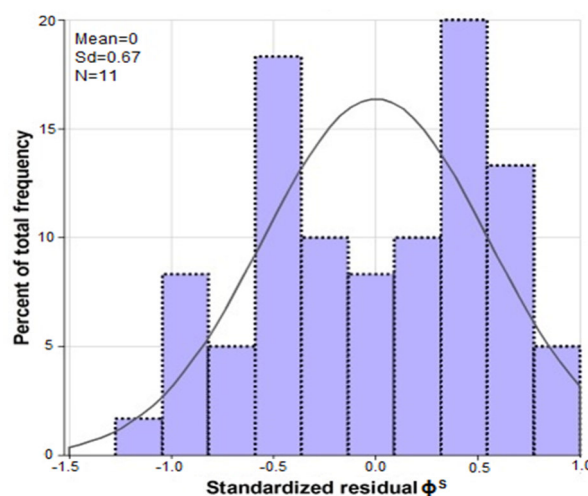


Figure 6.229: Frequency vs standardised residual (ϕ^s)

6.8.5 Geotechnical predictive models for soil cohesion (c')

6.8.5.1 Estimated model (c')

The response of c' to soil index properties, as presented in section 6.7.5, indicates a strong correlation between cohesion and water content, specific gravity, and clay content. The LL, PI, fine-grained content, and clay content describe a moderate influence on the cohesion of soil material. The multivariate analysis is conducted based on the correlation matrices outlined in Table 1, Table 2, Table 6.22 in Appendix 39, and Table 6.23 in Appendix 40. The model covariates include the parameters: water content, specific gravity, PI, LL, and fine-grained content. Table 6.43 in Appendix 50 shows the p-values, and Table 6.42 in Appendix 49 shows the summary and coefficients of the model as described in Equation 6.108.

$$(c')^\lambda = -\zeta_0 - \zeta_1 W - \zeta_2 \text{Fine} + \zeta_3 G_s - \zeta_4 LL + \zeta_5 PI \quad [6.108]$$

Where:

c' = cohesion [kPa]

W = gravimetric water content [%]

G_s = specific gravity

PI = plasticity index [%]

LL = liquid limit [%]

Fine = fine content [%]

λ = Box-cox transformation

ζ_0 = intercept

ζ_i = multivariate coefficients, $i = 1, 2, \dots, 5$

6.8.5.2 Model validation (c')

Figure 6.230 displays curve data illustrating the measured cohesion and predicted cohesion values obtained from 60 compacted specimens at different water content. The observed changes in estimated cohesion closely similar to the changes in predicted cohesion values. The scatter-plotted data displays minor disparities. The relation between the predicted and measured cohesion values demonstrate a strong correlation with a determination coefficient of 94.41 %.

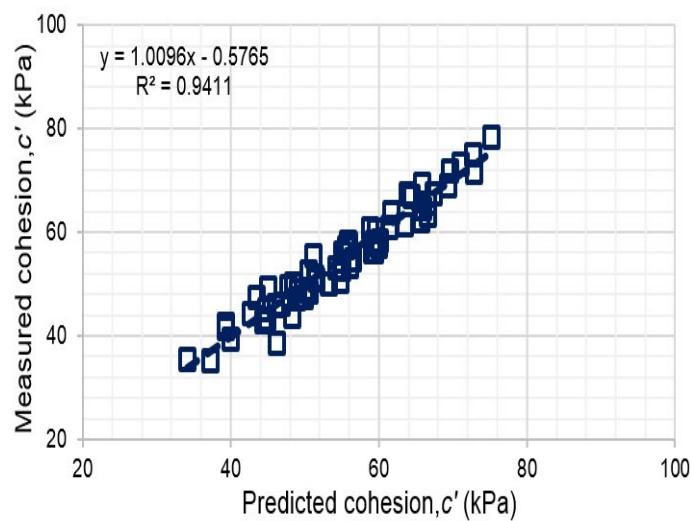


Figure 6.230: Measured vs predicted values of (c')

The model developed in this study is compared to the models proposed by Chen *et al.* (2012), Roy and Dass (2014). Figures 6.231 and 6.232 present the scatter-plotted data of the measured and predicted cohesion values obtained through the two models. While the model by Chen *et al.* (2012) portrays a strong correlation, the model proposed by Roy and Dass (2014) exhibits a weaker correlation. The observed differences may arise from the type of soil material studied in these models. The model by Chen *et al.* (2012) is developed based on compacted expansive soil, similar to the present model. However, the model proposed by Roy and Dass (2014) is designed based on cohesionless and non-swelling soil material. The predicted values from the model by Chen *et al.* (2012) align with the model developed in this study. Nevertheless, the present model can predict cohesion (c') with acceptable accuracy.

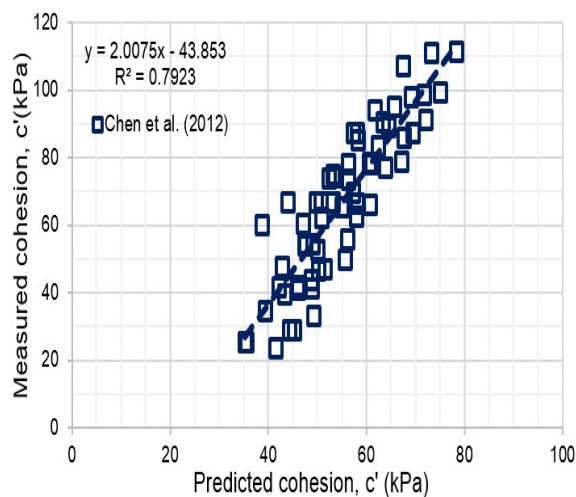


Figure 6.231: Measured vs predicted values of (c') Model Chen *et al.* (2012)

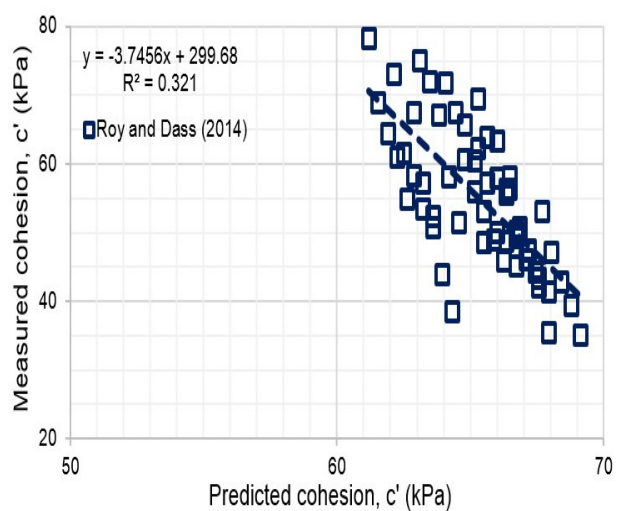


Figure 6.232: Measured vs predicted values of (c') Model Roy and Dass (2014)

CHAPTER 7: SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

The objective of this study was to evaluate the shear strength properties, formulate a predictive shear strength constitutive relation and associated models for compacted unsaturated expansive soils. Sedimentation analysis, particle size distribution, and consistency limits tests were conducted. The results indicated that BFS, WIS, and WES belong to the high plastic clay (CH) category, while BES falls under the low plastic clay (CL) category. Additionally, the X-ray diffraction test quantified the mineralogical constituents controlling the swelling behaviour of identified soils and validated the soil classification tests.

Geotechnical properties of soil samples were assessed through specific gravity tests, modified Proctor compaction tests, zero swell tests, soil suction measurements using the filter paper technique, soil-water characteristic curve modelling, and advanced triaxial testing. The study also explored the relationship between shear strength and matric suction, as well as the correlation between shear strength and swelling stress. The unsaturated shear strength parameters of the tested soils, their response to geotechnical index properties, and soil mineralogy were determined. A modified-unsaturated shear strength relation was formulated, and its response to geotechnical index properties and soil mineralogy was evaluated. Multivariate regression analysis was conducted on data collected from 60 tested soil specimens, using NCSS 12 software to design the geotechnical predictive models.

7.2 Conclusions

The outcomes of this research work were summarised as follows:

- The evaluation of various methods for measuring soil suction indicated that the filter paper technique and chilled-mirror hygrometer were cost-effective methods. However, the filter paper technique emerged as the preferred choice due to its simplicity and affordability. It provides an effective means to assess both total suction and matric suction across a wide range with careful attention to the testing process. This result justified the use of the filter paper technique in this study.

- The tested soils portrayed a range of liquid limits from 40 % to 79 %, plastic limits from 19 % to 57 %, free swell index ranging from 36 % to 133 %, and free swell ratio range varying between 1.17 to 2.50. Swelling stress ranged from 107 kPa to 827 kPa. The reported smectite clay mineral ranged from 32 % to 76 %. These results confirmed the classification of BFS, WIS, and WES as high plastic clay (CH), BES as low plastic clay (CL). The results also supported the swelling behaviour of the studied soils.
- The tested soils displayed total suction values ranging from 110 kPa to 11283 kPa, matric suction values ranging from 80 kPa to 8745 kPa, and air entry values ranging from 6 kPa to 20.20 kPa. Among various models, models Seki (BL) and Durner (DB) yielded the best fit for the soil-water characteristic curve (SWCC) for compacted expansive soil, surpassing Van Genuchten (VG) Fredlund and Xing (FX), and Kosugi (LN) models.
- Unsaturated shear strength values (τ_u) were found to range from 119 kPa to 1512 kPa, while saturated shear strength (τ_s) values ranged from 94 kPa to 941 kPa. The τ_u values were 1.1 to 1.78 times higher than τ_s values due to the contribution of the matric suction. Additionally, the tested soils displayed friction angles (ϕ') varying from 22° to 53°, angle related to matric suction (ϕ^b) ranging from 1.41° to 10°, angle (ϕ^s) ranging from 16.96° to 69°, (ϕ^s) indicated the rate of reduction in shear strength with respect to a variation of swelling stress (P_s), and cohesion (c') ranging from 35 kPa to 78 kPa.
- The parameters ϕ' , ϕ^b , ϕ^s , and c' were predominantly influenced by water content (%), specific gravity, and clay content (%). Specifically, ϕ' showed a strong correlation with the free swell ratio and free swell index. ϕ^b exhibited a strong relationship with liquid limit, plastic limit, and dry unit weight. ϕ^s displayed a strong correlation with free swell ratio and free swell index. ϕ' , ϕ^s , and c' demonstrated a moderate relationship with liquid limit, plastic limit, dry unit weight, and void ratio. ϕ' also exhibited a moderate correlation with smectite (%) and plagioclase (%). ϕ^b showed a strong relationship with smectite (%) and a moderate correlation with plagioclase and K-feldspar

(%). ϕ^s showed a moderate relationship with smectite (%) and K-feldspar (%), while c' displayed a moderate correlation with smectite (%), K-feldspar (%), and plagioclase (%).

- ϕ' , ϕ^b , ϕ^s , and c' increased when the dry unit weight increased and decreased when the plastic limit, liquid limit, void ratio, free swell index, free swell ratio, clay fraction, and fine-grained fraction increased. ϕ' decreased with an increase in smectite content and increased with an increase in plagioclase content. ϕ^b decreased with increasing smectite content and increased with increasing plagioclase and K-feldspar content. ϕ^s decreased with increasing smectite content and increased with increasing K-feldspar content. c' decreased with increasing smectite content and increased with increasing plagioclase and K-feldspar content.
- The modified-unsaturated shear strength (τ_{mu}) was established by the equilibrium of three stress variables: effective normal stress ($\sigma_n - u_a$), matric suction ($u_a - u_w$), and swelling stress (P_s). The contributions of matric suction and swelling stress were incorporated into the saturated shear strength of the soil material. The reported τ_{mu} values ranged from 31 kPa to 1162 kPa at the dry optimum. However, τ_{mu} values exhibited marginal values beyond the dry optimum. τ_{mu} values were found to be smaller than unsaturated shear strength (τ_u) values due to the development of swelling stress within the soil material, which reduced cohesion among soil particles, and ultimately leading to the reduction of shear strength.
- The evaluation of τ_{mu} was conducted by comparing τ_{mu} with three existing constitutive relations of τ_u established by Fredlund and Rahardjo (1993), Garven and Vanapalli (2006), and Bulolo and Leong (2019). Previous unsaturated shear strength constitutive relations might overestimated the shear strength of compacted unsaturated expansive soil due to their failure to account for the stress state induced by the swelling stress in these soils. Incorporating the influence of swelling stress on shear strength of expansive soil enhanced

the durability of constructions and reduced the cost associated with assessing and repairing damages to structures. Besides, the marginal values of τ_{mu} reported beyond the dry optimum can be enhanced using a proper soil treatment technique.

- A novel approach bridging classical soil mechanics (CSM) and unsaturated soil mechanics (USM) was developed for compacted unsaturated expansive soil. This approach utilised mathematical relations to determine unsaturated shear strength characteristics using basic geotechnical index properties. Geotechnical predictive models were designed for soil mineralogy, soil suction, compaction features, and unsaturated shear strength parameters. The predicted values were achieved with a satisfactory level of precision.
- Lastly, the research work clearly illustrates that the modified-unsaturated shear strength constitutive relation proposed in this study is an efficient tool for characterising the shear strength of expansive unsaturated soils. The influence of the swelling stress on the shear strength of expansive soils should not be overlooked in design. However, considering the influence of swelling stress on the shear strength of expansive soils improves the durability of constructions. The geotechnical predictive models designed in this study can effectively evaluate soil parameters and reduce the cost and time required for laboratory tests. The advancements explored in this research contribute to the progress in unsaturated soil mechanics and design enhancement.

- Modified-unsaturated shear strength constitutive relation:

$$\tau_{mu} = c' + (\sigma_n - u_a) \tan(\phi') + (u_a - u_w) \tan(\phi^b) + \delta P_s \tan(\phi^s) \quad [\text{model 1}]$$

- Geotechnical predictive models for soil mineralogy:

$$FSI \geq 35\% \text{ and } FSR \geq 1.10$$

$$Smectite = +\xi_0 + \xi_1 PI + \xi_2 FSR - \xi_3 G_S - \xi_4 FSI \quad [\text{model 2}]$$

$$Silica = +\eta_0 - \eta_1 PI - \eta_2 FSR - \eta_3 Gravel + \eta_4 FSI \quad [\text{model 3}]$$

$$k\text{-feldspar} = +\mu_0 + \mu_1 PI + \mu_2 FSR - \mu_3 G_S - \mu_4 FSI \quad [\text{model 4}]$$

$$Plagioclase = -\alpha_0 - \alpha_1 LL + \alpha_2 LS + \alpha_3 FSI + \alpha_4 G_S \quad [\text{model 5}]$$

- Geotechnical predictive models for compaction features:

$$OMC = +\xi_0 - \xi_1 LL + \xi_2 Fine + \xi_3 Sand + \xi_4 Gravel - \xi_5 G_S \quad [\text{model 6}]$$

$$\gamma_{dmax} = +\mu_0 - \mu_1 LL + \mu_2 Fine + \mu_3 Sand + \mu_4 Gravel - \mu_5 G_S \quad [\text{model 7}]$$

- Geotechnical predictive models for soil suction:

$$(\psi_m)^\lambda = +\xi_0 - \xi_1 W + \xi_2 Fine - \xi_3 \gamma_d - \xi_4 e + \xi_5 G_S - \xi_6 LL \quad [\text{model 8}]$$

$$(\psi_t)^\lambda = +\eta_0 - \eta_1 W + \eta_2 Fine - \eta_3 \gamma_d - \eta_4 e - \eta_5 G_S - \eta_6 LL \quad [\text{model 9}]$$

- Geotechnical predictive model for saturated friction angle:

$$(\phi')^\lambda = -\xi_0 - \xi_1 W + \xi_2 G_S + \xi_3 PI - \xi_4 LL - \xi_5 Clay - \xi_6 Fine \quad [\text{model 10}]$$

- Geotechnical predictive model for matric suction angle:

$$(\phi^b)^\lambda = +\eta_0 - \eta_1 W + \eta_2 G_S - \eta_3 Fine + \eta_4 LL - \eta_5 \gamma_d \quad [\text{model 11}]$$

- Geotechnical predictive model for swelling stress angle:

$$\phi^s = -\alpha_0 - \alpha_1 W + \alpha_2 G_S + \alpha_3 PI - \alpha_4 FSI - \alpha_5 Fine \quad [\text{model 12}]$$

- Geotechnical predictive model for soil cohesion:

$$(c')^\lambda = -\zeta_0 - \zeta_1 W - \zeta_2 Fine + \zeta_3 G_S - \zeta_4 LL + \zeta_5 PI \quad [\text{model 13}]$$

7.3 Recommendations

Further studies should be conducted for a good understanding of the mechanism and origin of reported weak correlations between the silica content and soil properties such as free swell ratio, free swell index, Atterberg limits, sand content, and gravel content.

The axis-translation technique (ATT) utilised for measuring matric suction during triaxial testing has a suction range of 0 to 1500 kPa. However, the reported matric suction values for compacted unsaturated expansive soils typically range from 40 kPa to 8000 kPa. Therefore, there is a need to extend the matric suction range of the ATT to accurately measure high matric suction values during triaxial testing.

Over the last few decades, significant advancements have been made in unsaturated soil mechanics. It is now widely recognised that the basic principles of soil mechanics should encompass both unsaturated and saturated soils. As a result, the geotechnical engineering curriculum in South African universities should incorporate the study of unsaturated soil mechanics.

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APPENDIX 1

Table 5.3: Modified compaction test data sheet

Geotechnical and Material Laboratory, Department of Civil Engineering , CUT					
COMPACTION TEST DATA SHEET: SANS 3001-GR30					
Sample ID:					
Date:					
Operator					
Mass taken (Kg)					
Description:					
I- APPROXIMATE VALUES					
a) Water added					
Basin number	1	2	3	4	5
Initial water content (W_i) %					
Added water in (Millilitre)					
Added water in (kg)					
Mass of the soil, M_{soil} (kg)					
Target moisture content % (W_t)					
b) Dry density					
Mould No	1	2	3	4	5
Mould+ Base plate+ Glass lid in (kg)					
Mould+ Base plate+ Glass lid+ Water in (kg)					
Mass of water in (kg)					
Temperature t° Test					
Rd of water @ t° Test see chart					
Volume of mould in millilitre : V_m					
Volume of mould (M^3)					
Mass of mould + Wet soil (M_1), (kg)					
Mass of mould (M_m), (kg)					
Mass of wet soil : $M_1 - M_m$, (kg)					
Total density, $Y_m = (M_1 - M_m) / V_m$, in (kg / m^3)					
Dry density , (kg/ m^3)					
II- ACTUAL VALUES					
a) Moisture					
Container number	1	2	3	4	5
Mass of container + wet soil (M_1) in (Gramme)					
Mass of container + dry soil (M_2) in (Gramme)					
Mass of container (M_c) in (Gramme)					
Mass of water ($M_1 - M_2$) in (Gramme)					
Mass of dry soil ($M_2 - M_c$) in (Gramme)					
Moisture content (%) $W = (M_1 - M_2) / (M_2 - M_c) * 100$					
Dry density , (kg / m^3)					
III- SUMMARY					
Maximum dry density, (Kg/ m^3)					
Optimum moisture content, W_{opt} (%)					

APPENDIX 2

Table 5.4: Soil suction test data sheet

Measurement of soil suction using filter paper- Data sheet
ASTM D 5298 (1994)
 Geotechnical and Material Laboratory, Department of Civil
 Engineering, CUT
 Sample ID
 Date tested:
 Tested by:

Sample	1-1		1-2		1-3		1-4		1-5	
Gravimetric water content of soil sample, W , (%)										
Tin No										
Top filter paper / Bottom filter paper	Top	Bot	Top	Bot	Top	Bot	Top	Bot	Top	Bot
Cold Tare Mass, g	Tc									
Mass of wet Filter paper + Cold Tare Mass, g	m1									
Mass of Dry Filter paper + Hot Tare Mass, g	m2									
Hot Tare Mass, g	Th									
Mass of water in Filter Paper, g $M_2 - T_h$	Mf									
Mass of water in Filter Paper, g $M_1 - M_2 - T_c + T_h$	Mw									
Water content of filter Paper, g $(M_w/M_f) \%$	Wf									
Suction, kPa	ψ									
Suction, logkPa	ψ									
Suction, $PF = \log kPa + 1$	ψ									

APPENDIX 3

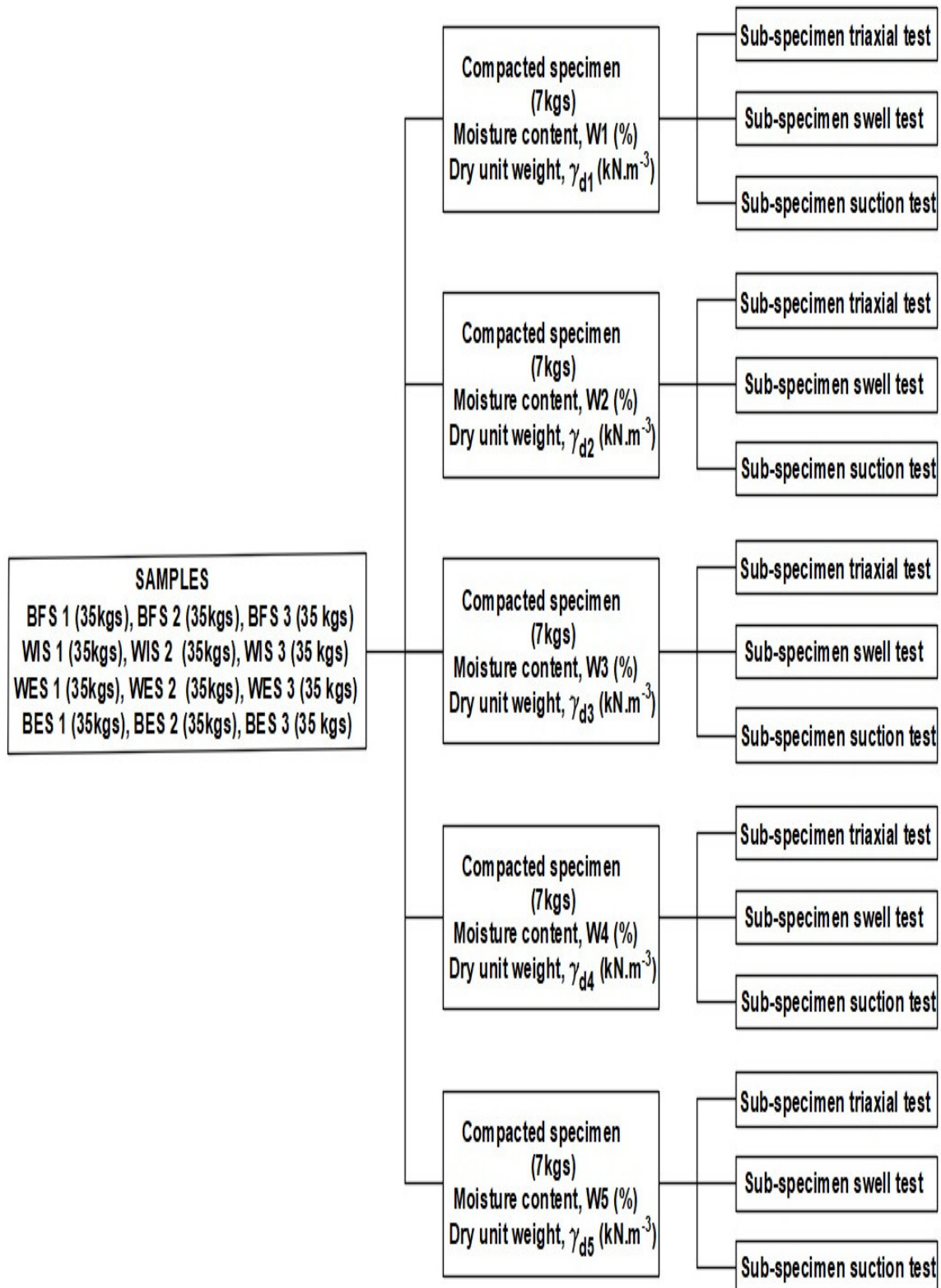


Figure 5.7: Diagram of specimen for zero swelling, suction, and triaxial tests

APPENDIX 4

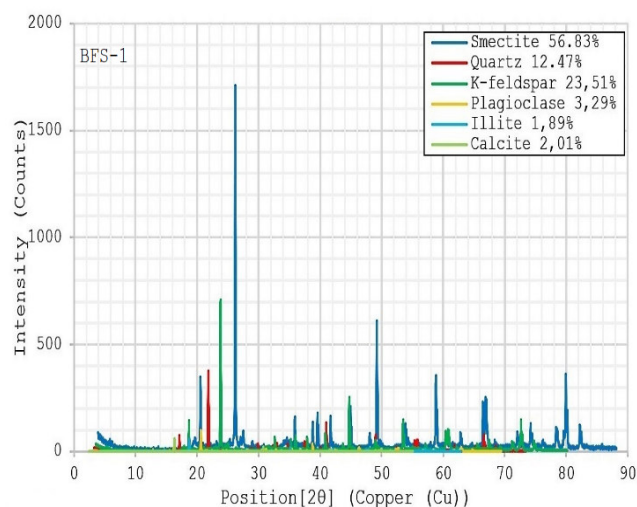


Figure 6.9: X-ray diffraction graph of BFS-1

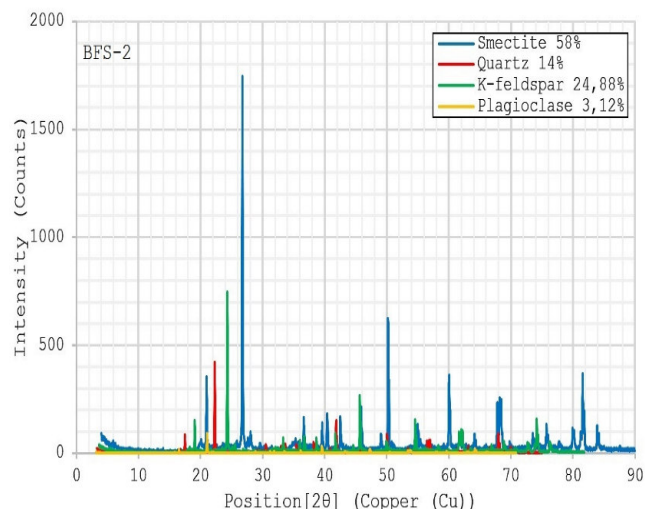


Figure 6.10: X-ray diffraction graph of BFS-2

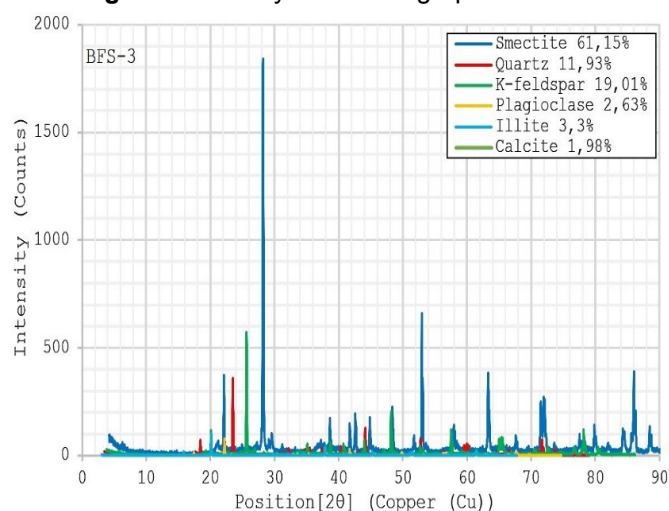


Figure 6.11: X-ray diffraction graph of BFS-3

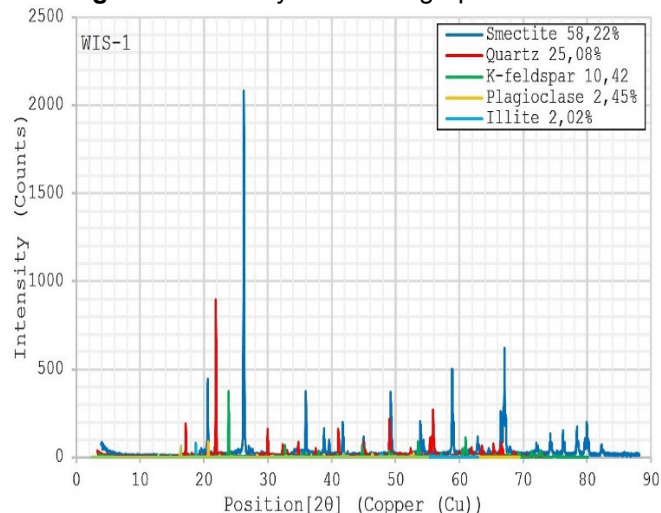


Figure 6.12: X-ray diffraction graph of WIS-1

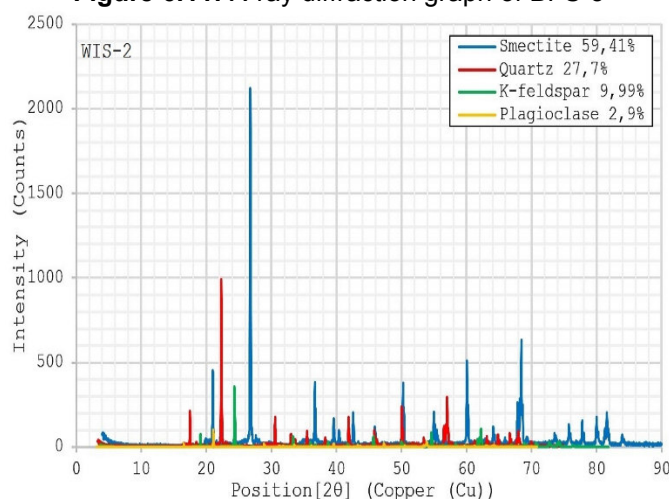


Figure 6.13: X-ray diffraction graph of WIS-2

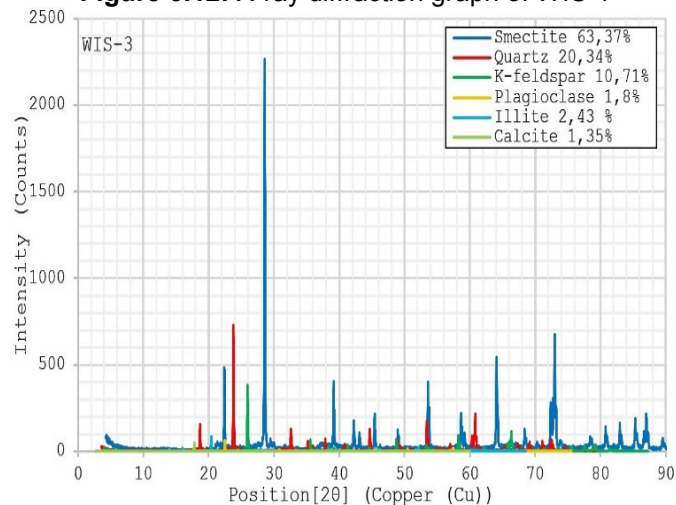


Figure 6.14: X-ray diffraction graph of WIS-3

APPENDIX 5

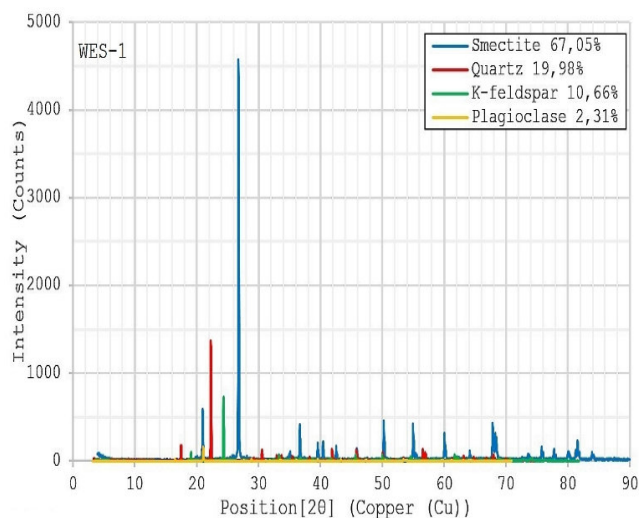


Figure 6.15: X-ray diffraction graph of WES-1

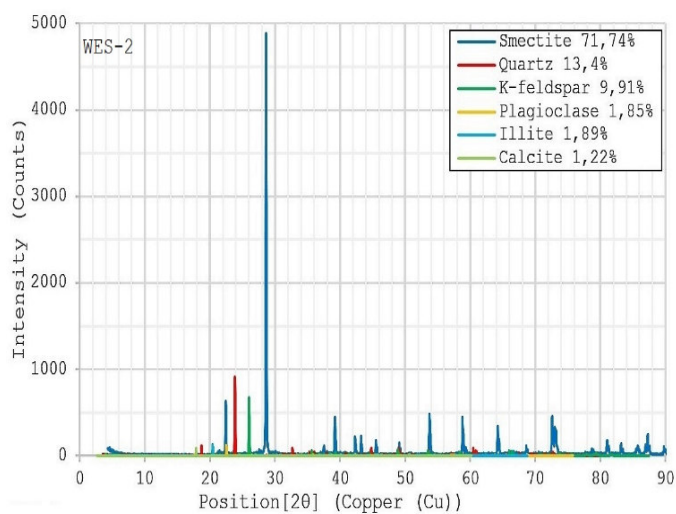


Figure 6.16: X-ray diffraction graph of WES-2

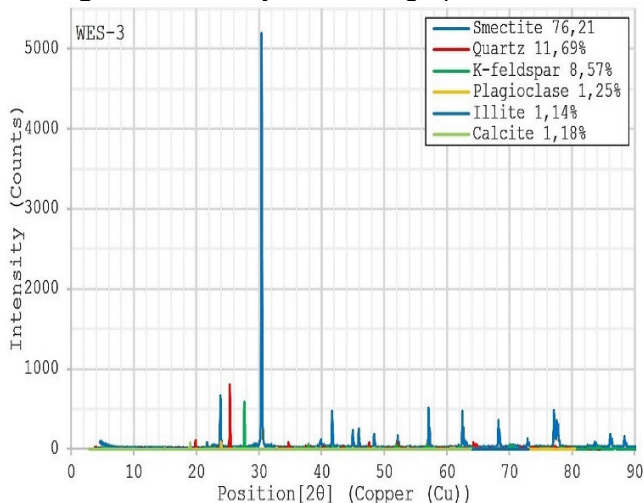


Figure 6.17: X-ray diffraction graph of WES-3

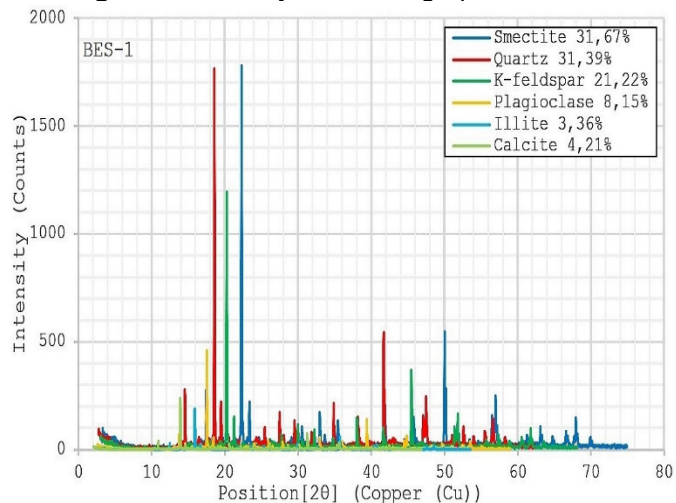


Figure 6.18: X-ray diffraction graph of BES-1

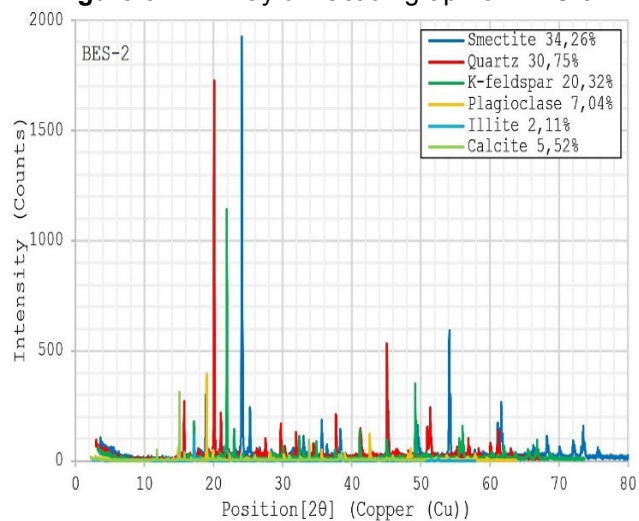


Figure 6.19: X-ray diffraction graph of BES-2

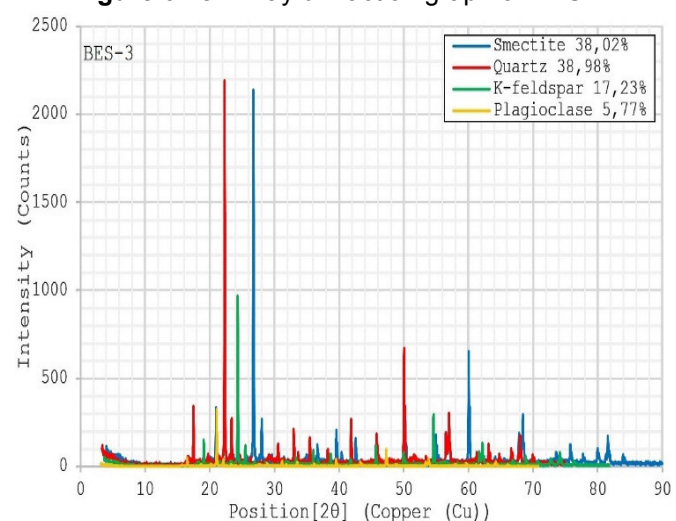


Figure 6.20: X-ray diffraction graph of BES-3

APPENDIX 6

Table 6.5: Smectite response

Variables	R ²	R ² (Adj)	R ² (Pred)	PI (%)	FSR	FSI (%)	Gs	Gravel (%)	Sand (%)
1	96.6	96.4	95.6	x					
1	95.8	95.5	94.8						x
2	97.8	97.4	96.8			x			
2	97.6	97.2	96.5	x				x	
3	99.1	98.9	98.5	x	x	x			
3	98.7	98.3	97.1		x	x			x
4	99.9	99.8	99.8	x	x	x	x		
4	99.8	99.8	99.6	x	x	x		x	
5	99.9	99.9	99.8	x	x	x	x	x	
5	99.9	99.9	99.8	x	x	x	x		x

Table 6.6: Silica response

Variables	R ²	R ² (Adj)	R ² (Pred)	PI (%)	FSR	FSI (%)	Gs	Gravel (%)	Sand (%)
1	43.1	38.7	27.9						x
1	28.8	23.4	8.8	x					
2	74.2	69.9	63.3	x				x	
2	72.3	67.4	59.1					x	x
3	79.9	74.4	65.4	x				x	
3	76.6	70.2	48.3		x	x			x
4	91.4	88.8	83.5	x	x	x		x	
4	91.4	88.1	75.6		x	x	x		x
5	94.7	91.8	77.3	x	x	x		x	x
5	94.1	90.8	78.8	x	x	x	x	x	

Table 6.7: K-feldspar response

Variables	R ²	R ² (Adj)	R ² (Pred)	PI (%)	FSR	FSI (%)	Gs	LL (%)
1	65.2	62.6	54.3				x	
1	60.4	57.3	51.1			x		
2	70.8	65.9	56.9				x	x
2	70.7	65.8	58.1		x	x		
3	86.2	82.4	75.0	x		x	x	
3	82.8	78.2	72.9		x	x	x	
4	95.1	93.6	90.2	x	x	x	x	
4	90.7	87.0	81.9	x	x	x		x
5	96.5	94.6	91.3	x	x	x	x	x

APPENDIX 7

Table 6.8: Plagioclase response

Variable	R ²	R ² (Adj)	R ² (Pred)	LL (%)	LS (%)	FSI (%)	Gs	Clay (%)
1	92.7	92.2	90.4		x			
1	89.1	88.2	85.4	x				
2	93.2	92.1	90.5		x	x		
2	92.8	91.6	89.6		x		x	
3	93.3	91.5	89.7	x	x	x		
3	93.3	91.5	88.3		x	x		x
4	95.2	93.7	92.6	x	x	x	x	
4	93.4	90.8	86.9		x	x	x	x
5	93.6	90.0	85.3	x	x	x	x	x

Table 6.9: Soil mineralogy response to soil properties correlation equations

Correlations equations	R ² (%)	Equation number
Smectite (%) = 11.68 + 1.13PI	96.66	6.3
Smectite (%) = -3.34 + 33.37FSR	90.07	6.4
Smectite (%) = -367.09 + 156.77Gs	85.32	6.5
Smectite (%) = 23.98 + 0.41FSI	87.03	6.6
Silica (%) = 42.07 - 0.523PI	50.44	6.7
Silica (%) = 48.72 - 15.235FSR	46.03	6.8
Silica (%) = 35.19 - 0.174FSI	37.97	6.9
Silica (%) = 16.461Gravel ^{0.133}	6.06	6.10
K-feldspar (%) = 26.76 - 0.142FSI	58.72	6.11
K-feldspar (%) = 28.98 - 0.342PI	49.6	6.12
K-feldspar (%) = 170.1 - 57.21Gs	63.57	6.13
K-feldspar (%) = 34.08 - 10.37FSR	50.76	6.14
Plagioclase (%) = 14.52 - 0.178LL	94.09	6.15
Plagioclase (%) = -1.73 + 0.566LS	92.72	6.16
Plagioclase (%) = 8.08 - 0.057FSI	72.17	6.17
Plagioclase (%) = 68.70 - 24.13Gs	85.16	6.18

APPENDIX 8

Table 6.10: Compaction features of twelve samples

Soils	Optimum moisture content (%)	Maximum dry unit weight (kN.m ⁻³)
BFS-1	20.07	17.58
BFS-2	22.61	17.16
BFS-3	23	16.95
WIS-1	24.03	16.85
WIS-2	24.58	16.71
WIS-3	26.05	16.45
WES-1	26.14	16.29
WES-2	26.52	16.05
WES-3	27.75	15.65
BES-1	17.23	19.60
BES-2	18.13	19.20
BES-3	18.24	18.76

Table 6.11: Compaction features response to soil properties correlation equations

Correlation equations	R ² (%)	Equation Number
$\gamma_{dmax} = 24.89e^{-0.006LL}$	98.28	6.19
$\gamma_{dmax} = 21.59e^{-0.006PI}$	98.85	6.20
$OMC = 9.86e^{0.0134LL}$	95.4	6.21
$OMC = 13.55e^{0.013PI}$	96.1	6.22
$OMC = 0.30Fine + 3.42$	97.48	6.23
$OMC = 37.12e^{-0.017Sand}$	91.6	6.24
$\gamma_{dmax} = -0.104Fine + 24.03$	96.44	6.25
$\gamma_{dmax} = 0.131Sand + 13.38$	95.11	6.26
$\gamma_{dmax} = 0.278Gravel + 15.87$	59.1	6.27
$\gamma_{dmax} = 158e^{-0.82Gs}$	92.48	6.28
$OMC = -0.858Gravel + 27.19$	68.51	6.29
$OMC = 41.71Gs - 89.79$	93.87	6.30
$\gamma_{dmax} = 1.025Cc + 16.68$	70	6.31
$\gamma_{dmax} = 15.27e^{0.0009Cu}$	76.81	6.32
$OMC = 24.22e^{-0.121Cc}$	57.06	6.33
$OMC = -0.044Cu + 29$	77.64	6.34

APPENDIX 9

Table 6.12: Suction test results for BFS and WIS

Soils	Soil water content (%)	Dry unit weight, (kN/m ³)	Total suction (kPa)	Matric suction (kPa)	Osmotic suction (kPa)
BFS-1	12.06	15.60	5883.92	4741.62	1142.3
	14.02	16.21	4064.22	3327.18	737.04
	17.03	17.01	1923.09	1388.22	534.87
	20.07	17.58	1036.11	671.89	364.22
	24.8	17.20	340.034	200	140.034
BFS-2	12.25	15.15	6112.321	4925.68	1186.641
	15.32	15.95	4221.982	3456.34	765.642
	20.12	16.98	1997.745	1442.11	555.635
	22.61	17.16	1076.324	697.98	378.344
	25.5	16.92	353.234	207.79	145.444
BFS-3	13.11	14.97	6444.53	5193.39	1251.14
	17.12	15.98	4451.45	3644.19	807.26
	19.98	16.59	2106.33	1520.49	585.84
	23	16.95	1134.82	735.9	398.92
	25.97	16.60	372.43	219.08	153.35
WIS-1	14.98	15.60	7439.15	5984.56	1454.59
	17.50	16.05	5410.66	4332.678	1077.982
	21	16.58	3580.89	2748.30	832.59
	24.03	16.85	1699.05	1199.35	499.70
	28.06	16.70	816.77	450.227	366.543
WIS-2	14.25	15.25	7723.408	6213.234	1510.174
	18.32	15.98	5617.411	4498.234	1119.177
	24.58	16.71	2517.727	1853.32	664.407
	26.21	16.65	1763.982	1245.199	518.783
	29.10	16.32	847.98	467.431	380.549
WIS-3	15	15.19	8239.036	6628.04	1610.996
	20	15.87	5992.43	4798.544	1193.886
	23	16.20	3965.92	3043.812	922.108
	26.05	16.45	1881.75	1328.33	553.42
	29.97	16.02	904.59	498.64	405.95

APPENDIX 10

Table 6.13: Suction test results for WES and BES

Soils	Soil water content (%)	Dry unit weight (kN/m ³)	Total suction (kPa)	Matric suction (kPa)	Osmotic suction (kPa)
WES-1	15.93	14.94	9926.183	7693.666	2232.517
	19.25	15.48	6922.321	5227.777	1694.544
	23.37	16.09	4011.482	2986.456	1025.026
	26.14	16.29	2475.62	1778.651	696.969
	29.10	15.85	1397.745	890.47	507.275
WES-2	17.20	14.77	10621.01	8232.22	2388.79
	19.98	15.20	7406.88	5593.72	1813.16
	23.05	15.70	4292.28	3195.51	1096.77
	26.52	16.05	2648.91	1903.156	745.754
	30.43	15.41	1495.59	952.8	542.79
WES-3	18	14.61	11283.21	8745.48	2537.73
	21.47	15	7868.687	5942.48	1926.207
	24.56	15.49	4559.901	3394.741	1165.16
	27.75	15.65	2814.07	2021.814	792.256
	30	15.21	1588.83	1012.20	576.63
BES-1	9.18	16.35	4163.35	3312.26	851.09
	12.42	18.01	2510.04	1982.31	527.73
	14.54	18.95	830.926	538.11	292.816
	17.23	19.60	323.818	225.609	98.209
	20.30	19.10	109.66	79.70	29.96
BES-2	9.21	16.01	4503.56	3582.93	920.63
	13.35	17.97	2715.16	2144.29	570.87
	15.50	18.70	898.827	582.082	316.745
	18.13	19.20	350.28	250.72	99.51
	21.05	18.58	118.62	86.22	32.4
BES-3	10.21	16.11	4997.235	3975.678	1021.557
	15	18	3012.787	2379.348	633.439
	18.24	18.76	997.354	645.888	351.466
	19.93	18.50	388.676	222.785	165.891
	21.6	18.01	131.631	95.666	35.965

APPENDIX 11

Table 6.14: Soil suction response to soil properties correlation equations

Correlation equations	R ² (%)	Equation Number
$\psi_m = 111.26e^{0.0607\text{Clay}}$	79.5	6.35
$\psi_m = 24.27 e^{0.0584\text{LL}}$	86.96	6.36
$\psi_m = 7081.5\text{Gs} - 18017$	83.82	6.37
$\psi_m = 18.805\text{FSI} - 377$	87.67	6.38
$\psi_m = -147.04\text{LS} + 2481$	74.33	6.39
$\psi_m = 3763\text{Void ratio} - 1148.2$	82.05	6.40
$\psi_m = 1447.2\text{FSR} - 1478.7$	81.70	6.41
$\psi_m = -455.34\gamma_d + 8973.5$	77.77	6.42

APPENDIX 12

Table 6.16: SWCC fitting parameters (BFS)

Soils	Model	Parameters	R ²	AIC
BFS-1	Van Genuchten	$\theta_s = 0.42989$; $\theta_r = 5.647 \times 10^{-6}$ $\alpha = 0.017725$; $n = 1.1659$	0.8539	-90.30
	Kosugi	$\theta_s = 0.46789$; $\theta_r = 1.8706 \times 10^{-7}$ $\psi_m = 1584.9$; $\sigma = 3.6659$	0.8867	-106.43
	Fredlund and Xing	$\theta_s = 0.4649$; $\theta_r = 3.8109 \times 10^{-7}$ $a = 1455.9$; $m = 2.436$; $n = 0.4657$	0.8877	-105.46
	Durner	$\theta_s = 0.44745$; $\theta_r = 1.6822 \times 10^{-5}$ $w_1 = 0.22098$; $\alpha_1 = 0.049397$ $n_1 = 2.3851$; $\alpha_2 = 0.001225$; $n_2 = 1.3945$	0.9965	-118.08
	Seki	$\theta_s = 0.44608$; $\theta_r = 7.2554 \times 10^{-6}$ $w_1 = 0.15043$; $\psi_{m1} = 21.100$ $\sigma_1 = 0.40657$; $\psi_{m2} = 3608.7$; $\sigma_2 = 2.2019$	0.9970	-119.81
BFS-2	Van Genuchten	$\theta_s = 0.44658$; $\theta_r = 5.866 \times 10^{-6}$ $\alpha = 0.017062$; $n = 1.1659$	0.8539	-89.313
	Kosugi	$\theta_s = 0.48606$; $\theta_r = 1.9426 \times 10^{-7}$ $\psi_m = 1646.5$; $\sigma = 3.6659$	0.8867	-105.44
	Fredlund and Xing	$\theta_s = 0.48295$; $\theta_r = 3.963 \times 10^{-7}$ $a = 1512.4$; $m = 2.4366$; $n = 0.46574$	0.8876	-104.47
	Durner	$\theta_s = 0.46481$; $\theta_r = 1.7461 \times 10^{-5}$ $w_1 = 0.22098$; $\alpha_1 = 0.047551$ $n_1 = 2.3851$; $\alpha_2 = 0.0011793$; $n_2 = 1.3945$	0.9965	-117.05
	Seki	$\theta_s = 0.46340$; $\theta_r = 1.8199 \times 10^{-6}$ $w_1 = 0.15047$; $\psi_{m1} = 21.917$ $\sigma_1 = 0.40629$; $\psi_{m2} = 3746.0$; $\sigma_2 = 2.2014$	0.9970	-118.82
BFS-3	Van Genuchten	$\theta_s = 0.47085$; $\theta_r = 6.185 \times 10^{-6}$ $\alpha = 0.0161183$; $n = 1.1659$	0.8539	-87.937
	Kosugi	$\theta_s = 0.51247$; $\theta_r = 2.0482 \times 10^{-7}$ $\psi_m = 1735.9$; $\sigma = 3.6659$	0.8867	-104.06
	Fredlund and Xing	$\theta_s = 0.50920$; $\theta_r = 4.181 \times 10^{-7}$ $a = 1594.6$; $m = 2.4366$; $n = 0.46574$	0.8876	-103.10
	Durner	$\theta_s = 0.49008$; $\theta_r = 1.8397 \times 10^{-5}$ $w_1 = 0.22098$; $\alpha_1 = 0.045100$ $n_1 = 2.3851$; $\alpha_2 = 0.0011185$; $n_2 = 1.3945$	0.9965	-115.68
	Seki	$\theta_s = 0.48859$; $\theta_r = 1.4799 \times 10^{-5}$ $w_1 = 0.15037$; $\psi_{m1} = 23.104$ $\sigma_1 = 0.40582$; $\psi_{m2} = 3950.9$; $\sigma_2 = 2.2023$	0.997	-117.45

APPENDIX 13

Table 6.17: SWCC fitting parameters (WIS)

Soils	Model	Parameters	R2	AIC
WIS-1	Van Genuchten	$\theta_s = 0.44316$; $\theta_r = 2.1792 \times 10^{-5}$ $\alpha = 0.0065788$; $n = 1.2187$	0.8663	-93.442
	Kosugi	$\theta_s = 0.47419$; $\theta_r = 3.7343 \times 10^{-6}$ $h_m = 2543.8$; $\sigma = 3.3720$	0.88390	-103.05
	Fredlund and Xing	$\theta_s = 0.47205$; $\theta_r = 2.8037 \times 10^{-7}$ $a = 7185.1$; $m = 3.6862$; $n = 0.47531$	0.88850	-105.42
	Durner	$\theta_s = 0.46594$; $\theta_r = 0.12655$ $w_1 = 0.50804$; $\alpha_1 = 0.049893$ $n_1 = 1.3857$; $\alpha_2 = 4.3103 \times 10^{-4}$; $n_2 = 2.9135$	0.99946	-141.11
	Seki	$\theta_s = 0.46616$; $\theta_r = 0.14876$ $w_1 = 0.43478$; $\psi_{m1} = 67.842$ $\sigma_1 = 1.5890$; $\psi_{m2} = 2780.2$; $\sigma_2 = 0.65859$	0.99941	-139.94
WIS-2	Van Genuchten	$\theta_s = 0.46010$; $\theta_r = 2.262 \times 10^{-5}$ $\alpha = 0.0063367$; $n = 1.2187$	0.8663	-92.467
	Kosugi	$\theta_s = 0.49231$; $\theta_r = 3.8773 \times 10^{-6}$ $\psi_m = 2641.0$; $\sigma = 3.3720$	0.8839	-102.07
	Fredlund and Xing	$\theta_s = 0.49008$; $\theta_r = 2.9051 \times 10^{-7}$ $a = 7459.7$; $m = 3.6862$ $n = 0.47531$	0.8885	-104.45
	Durner	$\theta_s = 0.48375$; $\theta_r = 0.13139$ $w_1 = 0.50804$; $\alpha_1 = 0.048056$ $n_1 = 1.3857$; $\alpha_2 = 4.1517 \times 10^{-4}$; $n_2 = 2.9135$	0.99946	-140.14
	Seki	$\theta_s = 0.48397$; $\theta_r = 0.15445$ $w_1 = 0.43478$; $\psi_{m1} = 70.434$ $\sigma_1 = 1.5890$; $\psi_{m2} = 2886.5$; $\sigma_2 = 0.65859$	0.99941	-138.96
WIS-3	Van Genuchten	$\theta_s = 0.49082$; $\theta_r = 2.413 \times 10^{-5}$ $\alpha = 0.0059401$; $n = 1.2187$	0.8663	-90.786
	Kosugi	$\theta_s = 0.52517$; $\theta_r = 4.1350 \times 10^{-6}$ $\psi_m = 2817.3$; $\sigma = 3.3720$	0.8839	-100.39
	Fredlund and Xing	$\theta_s = 0.52280$; $\theta_r = 3.1027 \times 10^{-7}$ $a = 7957.7$; $m = 3.6862$; $n = 0.47531$	0.8885	-102.77
	Durner	$\theta_s = 0.51604$; $\theta_r = 0.14016$ $w_1 = 0.50804$; $\alpha_1 = 0.045049$ $n_1 = 1.3857$; $\alpha_2 = 3.8919 \times 10^{-4}$; $n_2 = 2.9135$	0.99894	-138.46
	Seki	$\theta_s = 0.51628$; $\theta_r = 0.16476$ $w_1 = 0.43478$; $\psi_{m1} = 75.136$ $\sigma_1 = 1.5890$; $\psi_{m2} = 3079.2$; $\sigma_2 = 0.65859$	0.99941	-137.28

APPENDIX 14

Table 6.18: SWCC fitting parameters (WES)

Soils	Model	Parameters	R ²	AIC
WES-1	Van Genuchten	$\theta_s = 0.50894$; $\theta_r = 1.5927 \times 10^{-6}$ $\alpha = 0.0018093$; $n = 1.2824$	0.8483	-85.438
	Kosugi	$\theta_s = 0.56075$; $\theta_r = 1.9665 \times 10^{-7}$ $\psi_m = 4171.4$; $\sigma = 3.4437$	0.8658	-90.794
	Fredlund and Xing	$\theta_s = 0.55326$; $\theta_r = 2.4283 \times 10^{-7}$ $a = 1.3235 \times 10^{-4}$; $m = 3.9520$ $n = 0.49182$	0.87575	-93.263
	Durner	$\theta_s = 0.56265$; $\theta_r = 0.10309$; $w_1 = 0.25377$; $\alpha_1 = 0.067191$; $n_1 = 2.0389$; $\alpha_2 = 4.9483 \times 10^{-4}$ $n_2 = 1.8006$	0.99858	-126.12
	Seki	$\theta_s = 0.56368$; $\theta_r = 0.14911$; $w_1 = 0.27315$ $\psi_{m1} = 24.204$; $\sigma_1 = 1.0285$; $\psi_{m2} = 3380.7$ $\sigma_2 = 1.0714$	0.99876	-127.89
WES-2	Van Genuchten	$\theta_s = 0.54456$; $\theta_r = 1.704 \times 10^{-5}$ $\alpha = 0.00116909$; $n = 1.2824$	0.8658	-89.035
	Kosugi	$\theta_s = 0.6000$; $\theta_r = 2.1042 \times 10^{-5}$ $\psi_m = 4463.4$; $\sigma = 3.4437$	0.86580	-89.035
	Fredlund and Xing	$\theta_s = 0.59198$; $\theta_r = 2.5965 \times 10^{-6}$ $a = 1.4162 \times 10^{-4}$; $m = 3.9520$ $n = 0.49182$	0.87575	-91.504
	Durner	$\theta_s = 0.60204$; $\theta_r = 0.11030$; $w_1 = 0.25377$ $\alpha_1 = 0.062795$; $n_1 = 2.0389$; $\alpha_2 = 4.624 \times 10^{-4}$ $n_2 = 1.8006$	0.99858	-124.36
	Seki	$\theta_s = 0.60314$; $\theta_r = 0.15954$; $w_1 = 0.27315$ $\psi_{m1} = 25.898$; $\sigma_1 = 1.0284$; $\psi_{m2} = 3617.4$ $\sigma_2 = 1.0714$	0.9987	-126.13
WES-3	Van Genuchten	$\theta_s = 0.57852$; $\theta_r = 1.810 \times 10^{-5}$ $\alpha = 0.0015921$; $n = 1.2824$	0.8483	-82.106
	Kosugi	$\theta_s = 0.63741$; $\theta_r = 2.235 \times 10^{-5}$ $\psi_m = 4740.4$; $\sigma = 3.4437$	0.86580	-87.462
	Fredlund and Xing	$\theta_s = 0.62889$; $\theta_r = 2.7598 \times 10^{-6}$ $a = 1.5041 \times 10^{-4}$; $m = 3.9520$ $n = 0.49182$	0.87575	-89.931
	Durner	$\theta_s = 0.63957$; $\theta_r = 0.11718$; $w_1 = 0.25377$ $\alpha_1 = 0.059125$; $n_1 = 2.0389$; $\alpha_2 = 4.3543 \times 10^{-4}$ $n_2 = 1.8006$	0.99858	-122.79
	Seki	$\theta_s = 0.64075$; $\theta_r = 0.16949$; $w_1 = 0.27315$ $\psi_{m1} = 27.506$; $\sigma_1 = 1.0285$; $\psi_{m2} = 3841.9$ $\sigma_2 = 1.0714$	0.99876	-124.55

APPENDIX 15

Table 6.19: SWCC fitting parameters (BES)

Soils	Model	Parameters	R ²	AIC
BES-1	Van Genuchten	$\theta_s = 0.26834$; $\theta_r = 2.972 \times 10^{-7}$ $\alpha = 0.010376$; $n = 1.2528$	0.8855	-114.40
	Kosugi	$\theta_s = 0.28011$; $\theta_r = 0.033192$ $\psi_m = 813.52$; $\sigma = 2.7694$	0.8923	-122.55
	Fredlund and Xing	$\theta_s = 0.28535$; $\theta_r = 0.059885$ $a = 5439.3$; $m = 7.1447$ $n = 0.52624$	0.8937	-123.23
	Durner	$\theta_s = 0.27908$; $\theta_r = 0.066162$; $w_1 = 0.22182$ $\alpha_1 = 0.086889$; $n_1 = 2.4494$; $\alpha_2 = 0.0023956$ $n_2 = 1.7393$	0.9984	-136.98
	Seki	$\theta_s = 0.27909$; $\theta_r = 0.076861$; $w_1 = 0.21144$ $\psi_{m1} = 14.536$; $\sigma_1 = 0.75790$; $\psi_{m2} = 847.83$ $\sigma_2 = 1.3262$	0.9977	-132.30
BES-2	Van Genuchten	$\theta_s = 0.31141$; $\theta_r = 3.461 \times 10^{-7}$ $\alpha = 0.0089406$; $n = 1.2528$	0.8855	-110.53
	Kosugi	$\theta_s = 0.32507$; $\theta_r = 0.038520$ $\psi_m = 944.10$; $\sigma = 2.7694$	0.8923	-118.68
	Fredlund and Xing	$\theta_s = 0.33115$; $\theta_r = 0.069496$ $a = 6312.4$; $m = 7.1447$ $n = 0.52624$	0.8937	-119.36
	Durner	$\theta_s = 0.32388$; $\theta_r = 0.076781$; $w_1 = 0.2218$ $\alpha_1 = 0.074872$; $n_1 = 2.4494$; $\alpha_2 = 0.0020643$ $n_2 = 1.7393$	0.9984	-133.11
	Seki	$\theta_s = 0.32389$; $\theta_r = 0.089197$; $w_1 = 0.21145$ $\psi_{m1} = 16.869$; $\sigma_1 = 0.75790$; $\psi_{m2} = 983.91$ $\sigma_2 = 1.3262$	0.9977	-128.43
BES-3	Van Genuchten	$\theta_s = 0.33083$; $\theta_r = 3.680 \times 10^{-7}$ $\alpha = 0.0084159$; $n = 1.2528$	0.8855	-108.96
	Kosugi	$\theta_s = 0.34534$; $\theta_r = 0.040922$ $\psi_m = 1003.0$; $\sigma = 2.7694$	0.89226	-117.11
	Fredlund and Xing	$\theta_s = 0.35179$; $\theta_r = 0.073829$ $a = 6705.9$; $m = 7.1447$ $n = 0.52624$	0.89370	-117.78
	Durner	$\theta_s = 0.34407$; $\theta_r = 0.081568$; $w_1 = 0.22182$ $\alpha_1 = 0.070478$; $n_1 = 2.4494$; $\alpha_2 = 0.0019432$ $n_2 = 1.7393$	0.99839	-131.54
	Seki	$\theta_s = 0.34408$; $\theta_r = 0.094758$; $w_1 = 0.21145$ $\psi_{m1} = 17.921$; $\sigma_1 = 0.75790$; $\psi_{m2} = 1045.3$ $\sigma_2 = 1.3262$	0.9977	-126.86

APPENDIX 16

Table 6.20: Swelling stress test results for BFS and WIS as compacted

Soils	Soil water content (%)	Dry unit weight, (kN/m ³)	Surcharge load (kN)	Ring area (m ²)	Swelling stress (kPa)
BFS-1	12.06	15.60	0.25	0.00196	127.551
	14.02	16.21	0.35	0.00196	178.571
	17.03	17.01	0.46	0.00196	234.694
	20.07	17.58	0.60	0.00196	306.122
	24.8	17.20	0.67	0.00196	341.837
BFS-2	12.25	15.15	0.27	0.00196	137.755
	15.32	15.95	0.37	0.00196	188.776
	20.12	16.98	0.45	0.00196	229.592
	22.61	17.16	0.64	0.00196	326.531
	25.50	16.92	0.71	0.00196	362.245
BFS-3	13.11	14.97	0.29	0.00196	147.960
	17.12	15.98	0.40	0.00196	204.082
	19.98	16.59	0.53	0.00196	270.408
	23	16.95	0.69	0.00196	352.041
	25.97	16.60	0.77	0.00196	392.857
WIS-1	14.98	15.60	0.21	0.00196	107.143
	17.50	16.05	0.46	0.00196	234.694
	21.00	16.58	0.57	0.00196	290.816
	24.03	16.85	0.73	0.00196	372.449
	28.06	16.70	0.82	0.00196	418.367
WIS-2	14.25	15.25	0.22	0.00196	112.245
	18.32	15.98	0.49	0.00196	250
	24.85	16.71	0.61	0.00196	311.225
	26.21	16.65	0.78	0.00196	397.959
	29.10	16.32	0.88	0.00196	448.979
WIS-3	15	15.19	0.24	0.00196	122.449
	20	15.87	0.53	0.00196	270.408
	23	16.20	0.66	0.00196	336.735
	26.05	16.45	0.85	0.00196	433.674
	29.97	16.02	0.96	0.00196	489.796

APPENDIX 17

Table 6.21: Swelling stress test results for WES and BES as compacted

Soils	Soil water content (%)	Dry unit weight (kN/m ³)	Surcharge load (kN)	Ring area (m ²)	Swelling stress (kPa)
WES-1	15.93	14.94	0.27	0.00196	137.755
	19.25	15.48	0.51	0.00196	260.204
	23.37	16.09	0.78	0.00196	397.959
	26.14	16.29	0.98	0.00196	500
	29.10	15.85	1.18	0.00196	602.041
WES-2	17.20	14.77	0.33	0.00196	168.367
	19.98	15.20	0.65	0.00196	331.633
	23.05	15.70	0.95	0.00196	484.694
	26.52	16.05	1.18	0.00196	602.041
	30.43	15.41	1.42	0.00196	724.489
WES-3	18	14.61	0.37	0.00196	188.775
	21.47	15	0.71	0.00196	362.244
	24.56	15.49	1.08	0.00196	551.020
	27.75	15.65	1.35	0.00196	688.775
	30	15.21	1.62	0.00196	826.530
BES-1	9.18	16.35	0.28	0.00196	143.882
	12.42	18.01	0.30	0.00196	150.86
	14.54	18.95	0.34	0.00196	174.09
	17.23	19.60	0.53	0.00196	269.26
	20.30	19.10	0.70	0.00196	356.90
BES-2	9.21	16.01	0.28	0.00196	144.31
	13.35	17.97	0.30	0.00196	151.86
	15.50	18.70	0.35	0.00196	176.98
	18.13	19.20	0.55	0.00196	279.93
	21.05	18.58	0.73	0.00196	374.74
BES-3	10.21	16.11	0.28	0.00196	144.93
	15	18	0.30	0.00196	153.31
	18.24	18.76	0.36	0.00196	181.19
	19.93	18.50	0.58	0.00196	295.42
	21.60	18.01	0.79	0.00196	400.62

APPENDIX 18

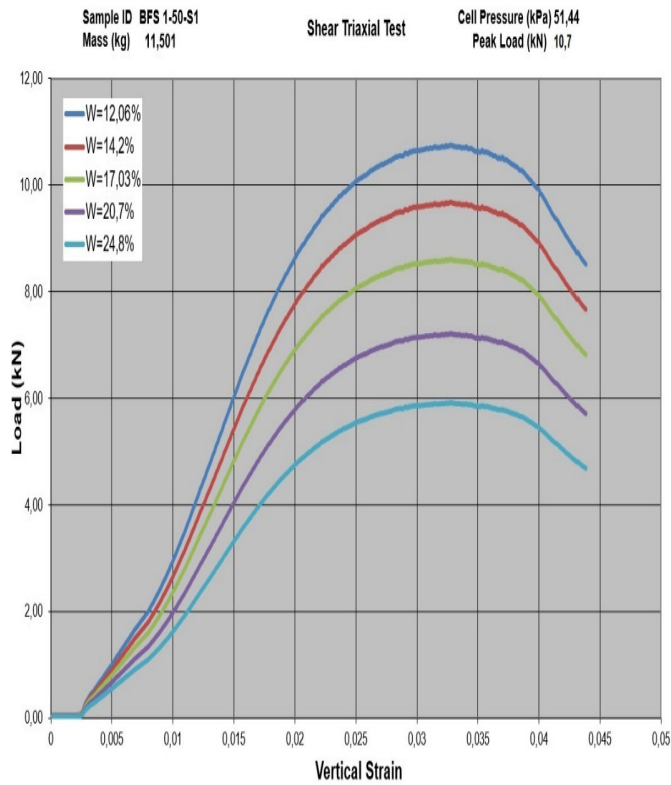


Figure 6.78: Stress-strain graph of BFS-1 at $\sigma_c = 50 \text{ kPa}$

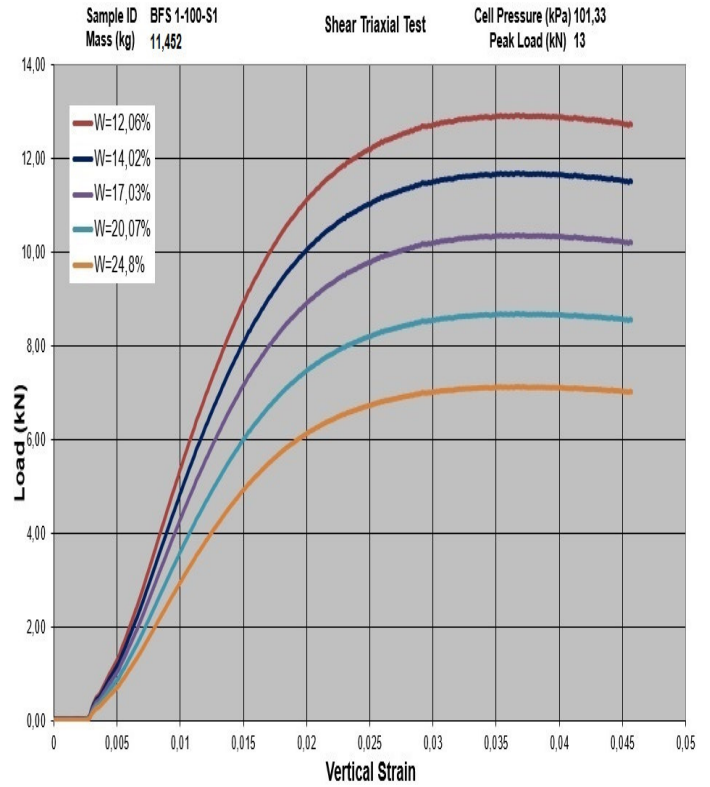


Figure 6.79: Stress-strain graph of BFS-1 at $\sigma_c = 100 \text{ kPa}$

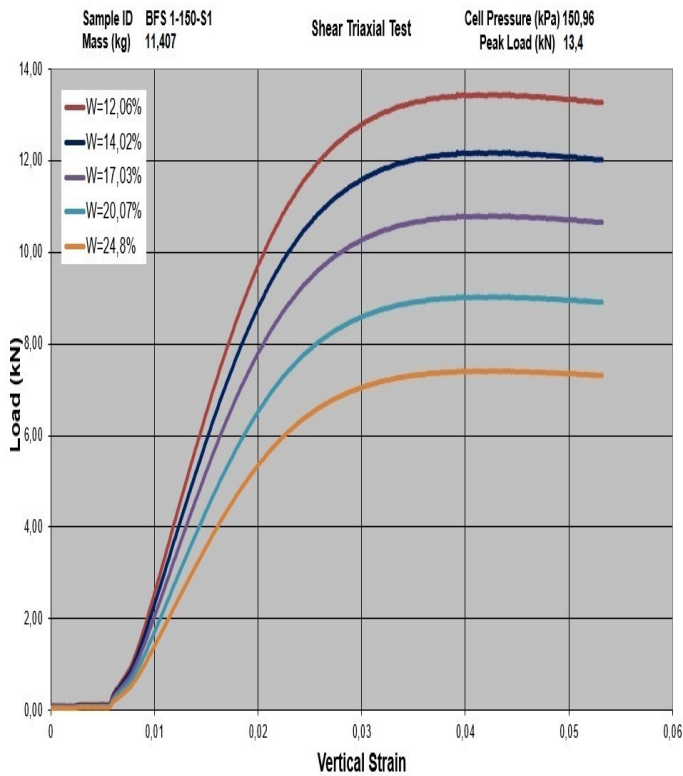


Figure 6.80: Stress-strain graph of BFS-1 at $\sigma_c = 150 \text{ kPa}$

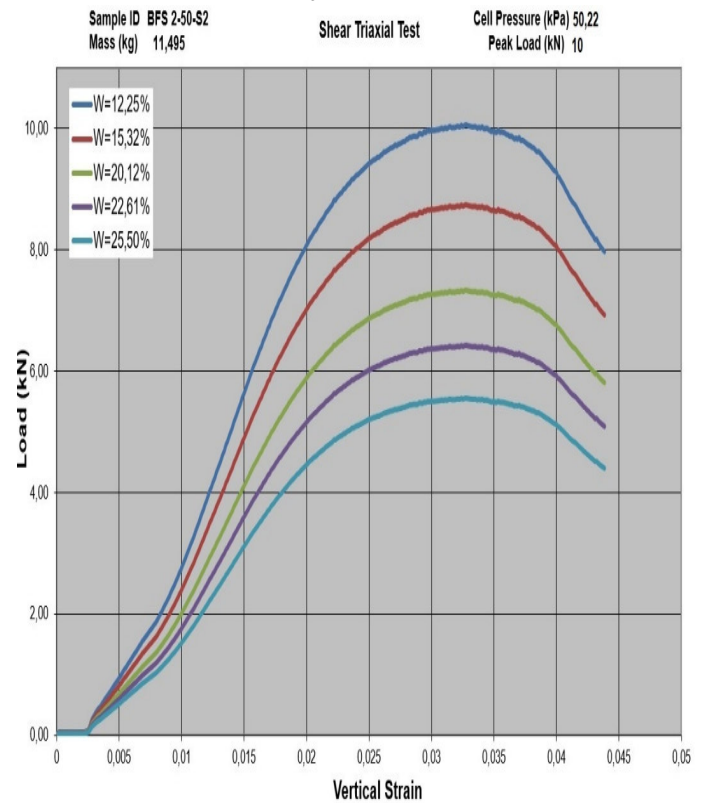


Figure 6.81: Stress-strain graph of BFS-2 at $\sigma_c = 50 \text{ kPa}$

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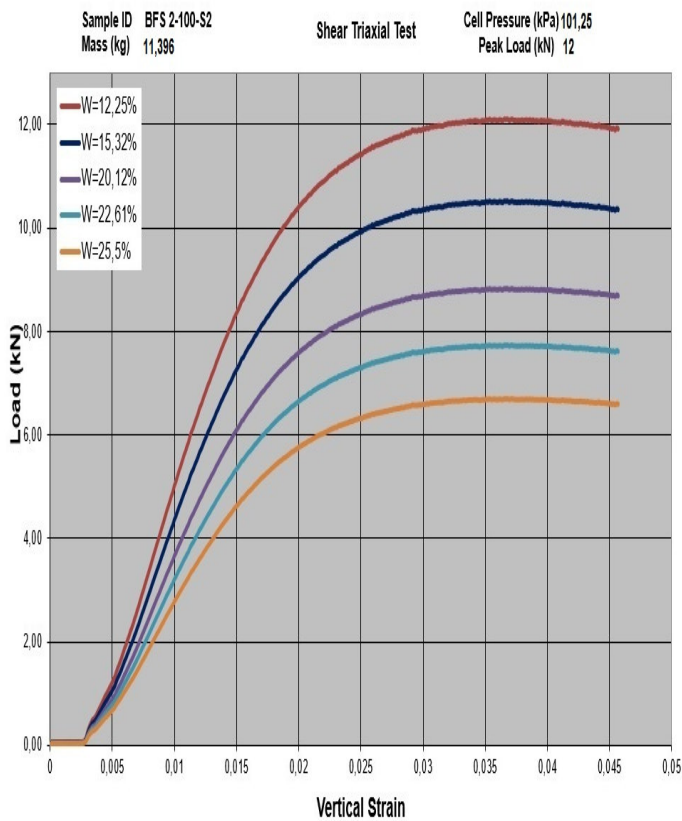


Figure 6.82: Stress-strain graph of BFS-2
at $\sigma_c = 100$ kPa

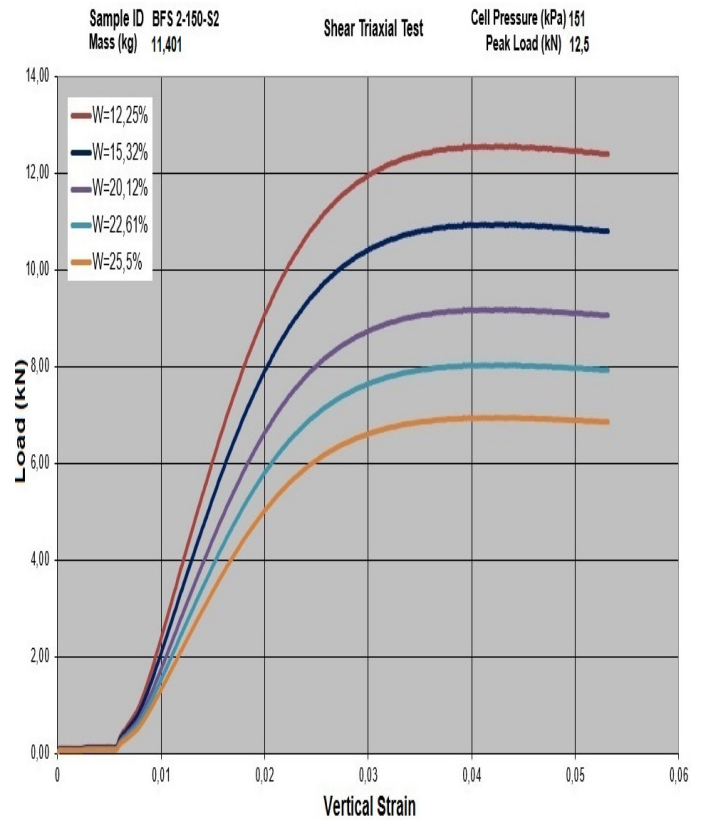


Figure 6.83: Stress-strain graph of BFS-2
at $\sigma_c = 150$ kPa

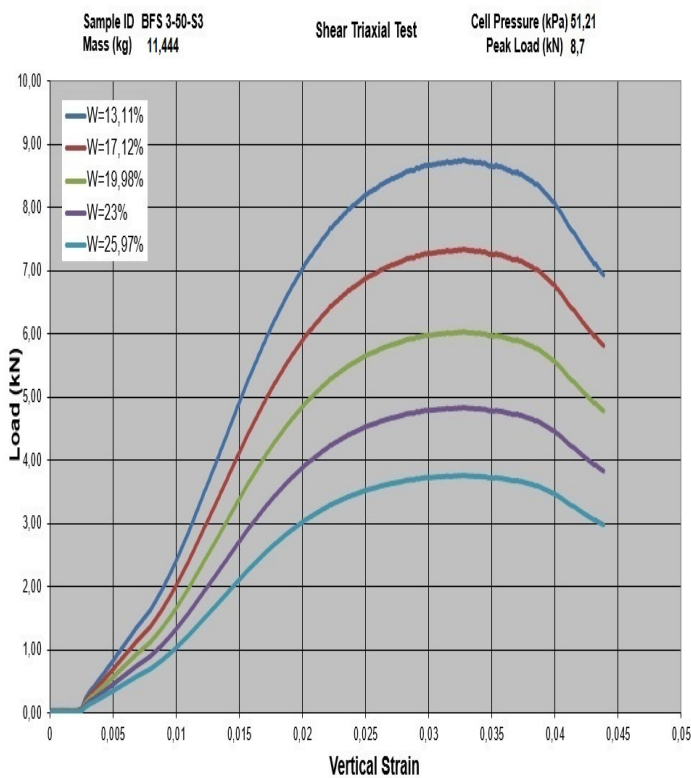


Figure 6.84: Stress-strain graph of BFS-3
at $\sigma_c = 50$ kPa

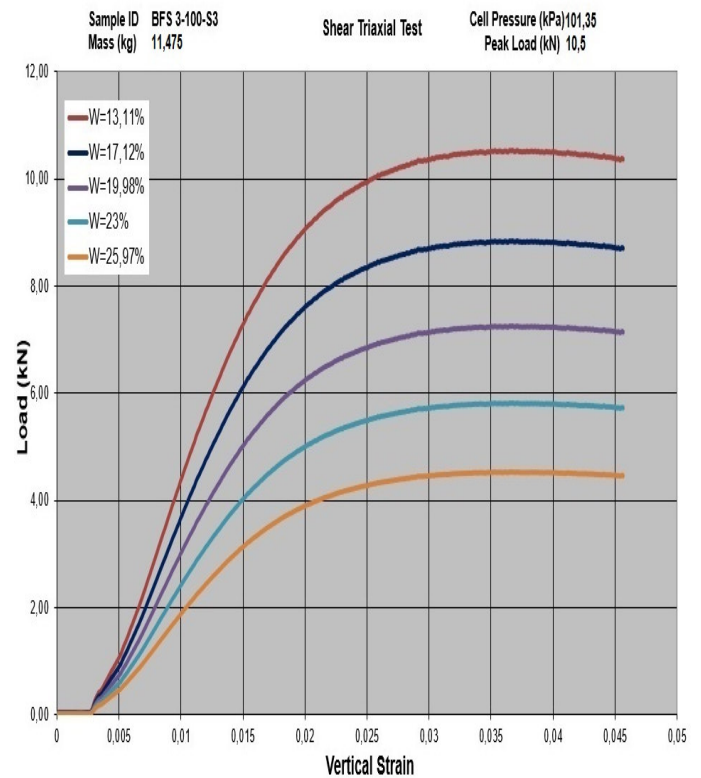


Figure 6.85: Stress-strain graph of BFS-3
at $\sigma_c = 100$ kPa

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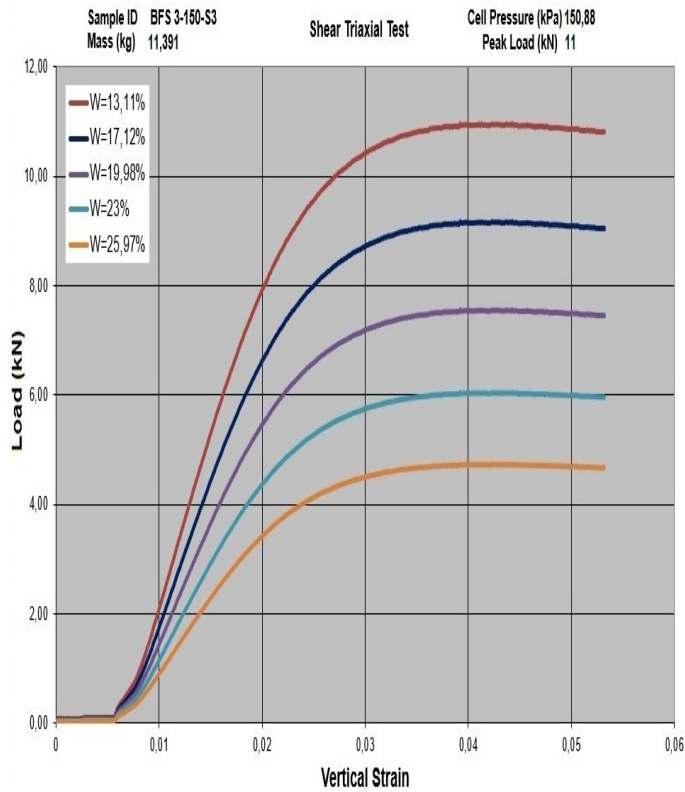


Figure 6.86: Stress-strain graph of BFS-3 at $\sigma_c = 150$ kPa

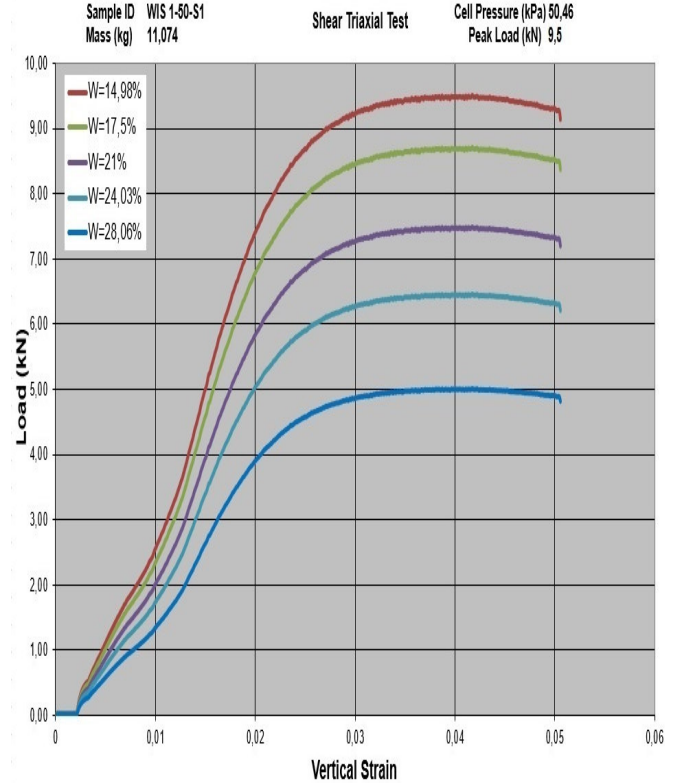


Figure 6.87: Stress-strain graph of WIS-1 at $\sigma_c = 50$ kPa

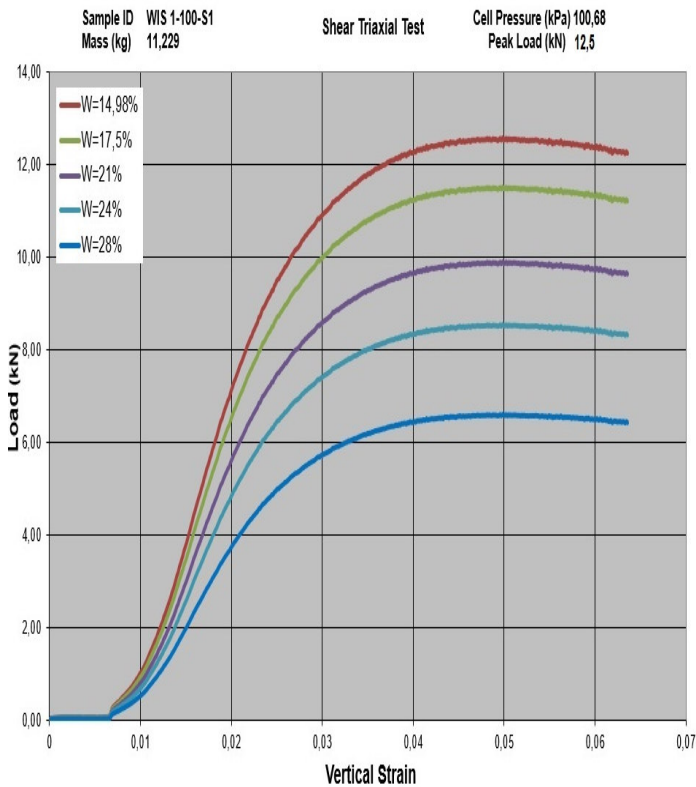


Figure 6.88: Stress-strain graph of WIS-1 at $\sigma_c = 100$ kPa

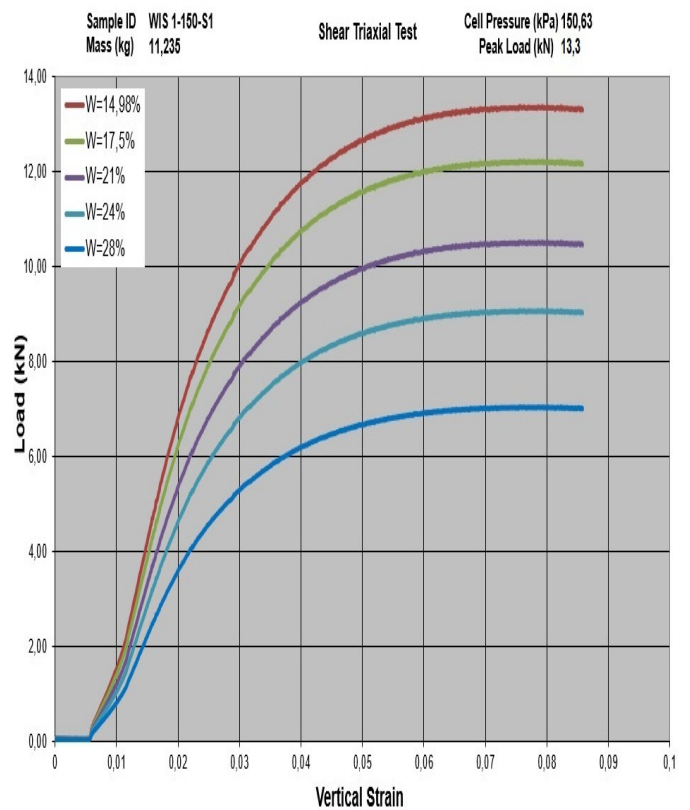


Figure 6.89: Stress-strain graph of WIS-1 at $\sigma_c = 150$ kPa

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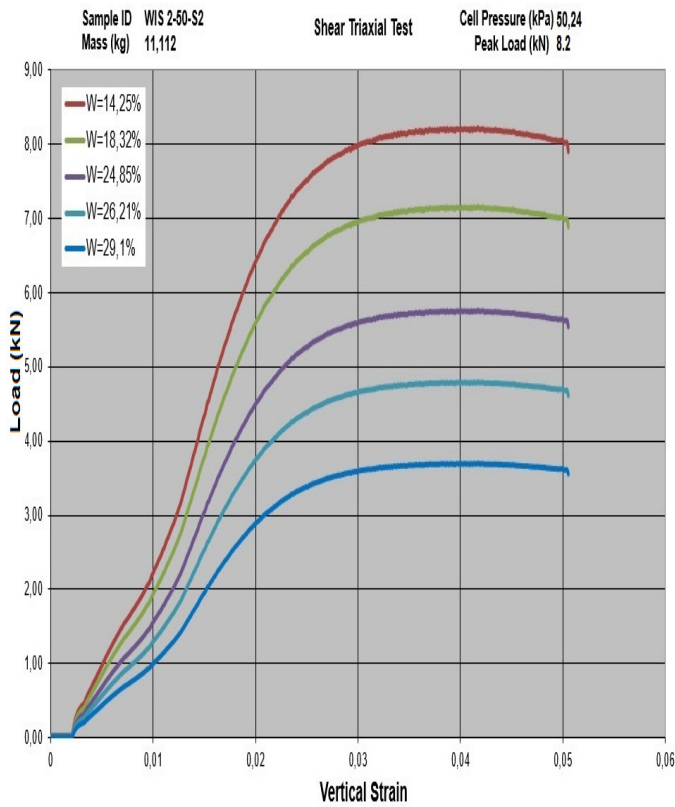


Figure 6.90: Stress-strain graph of WIS-2 at $\sigma_c = 50$ kPa

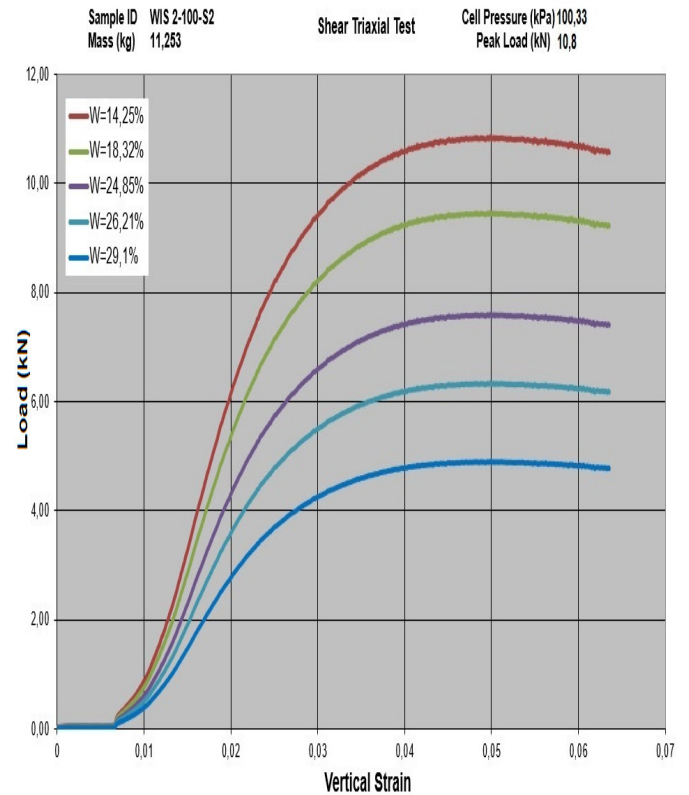


Figure 6.91: Stress-strain graph of WIS-2 at $\sigma_c = 100$ kPa

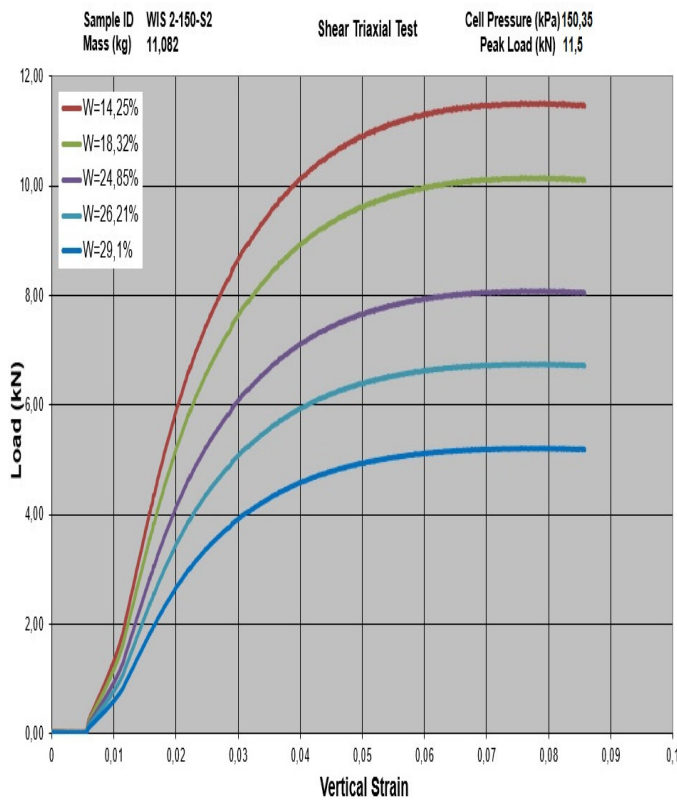


Figure 6.92: Stress-strain graph of WIS-2 at $\sigma_c = 150$ kPa

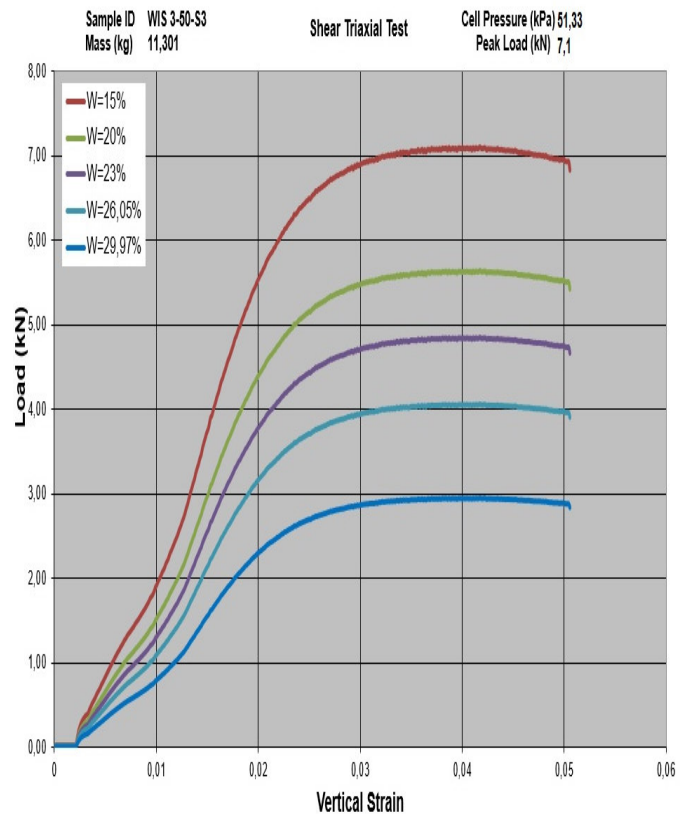


Figure 6.93: Stress-strain graph of WIS-3 at $\sigma_c = 50$ kPa

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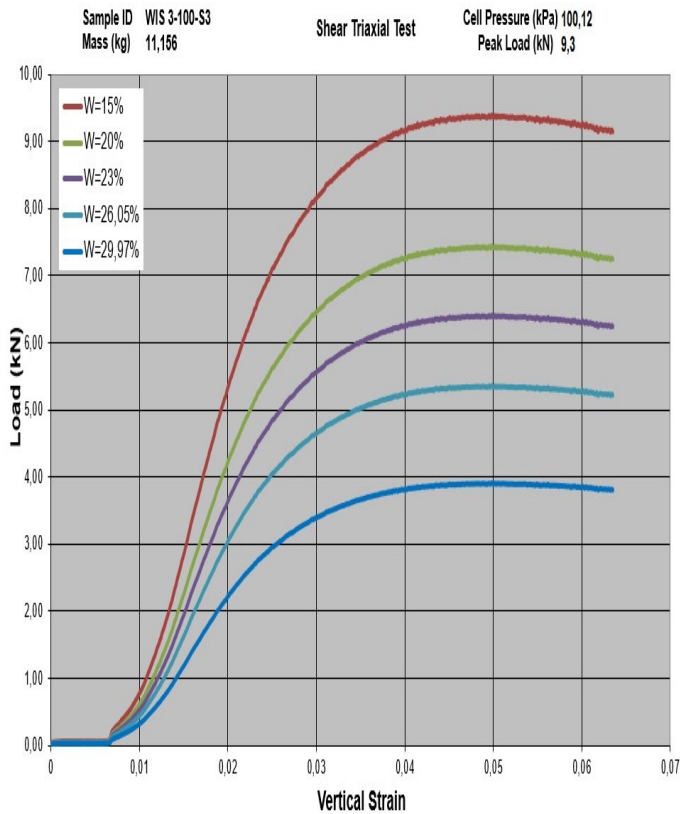


Figure 6.94: Stress-strain graph of WIS-3
at $\sigma_c = 100$ kPa

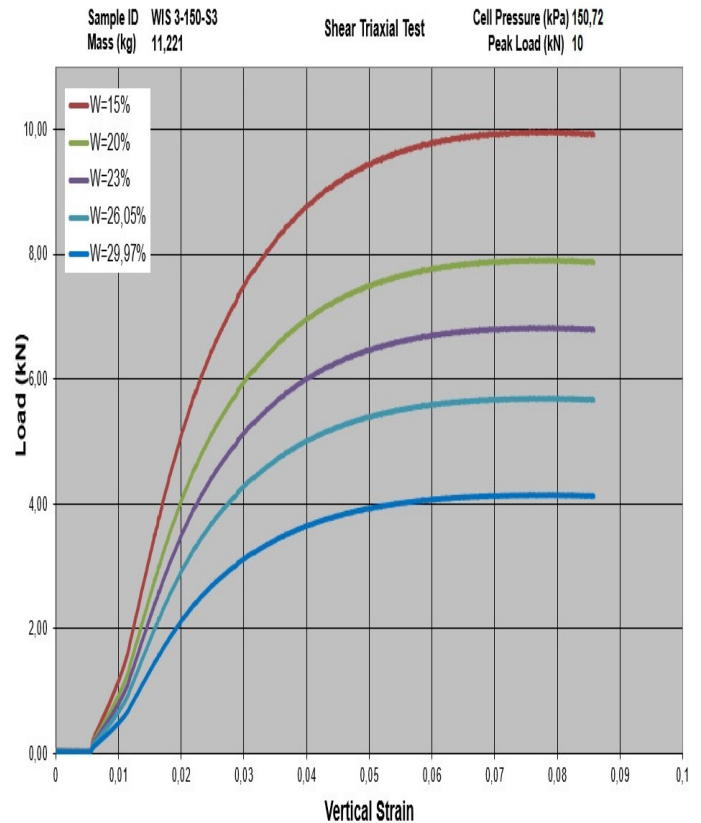


Figure 6.95: Stress-strain graph of WIS-3
at $\sigma_c = 150$ kPa

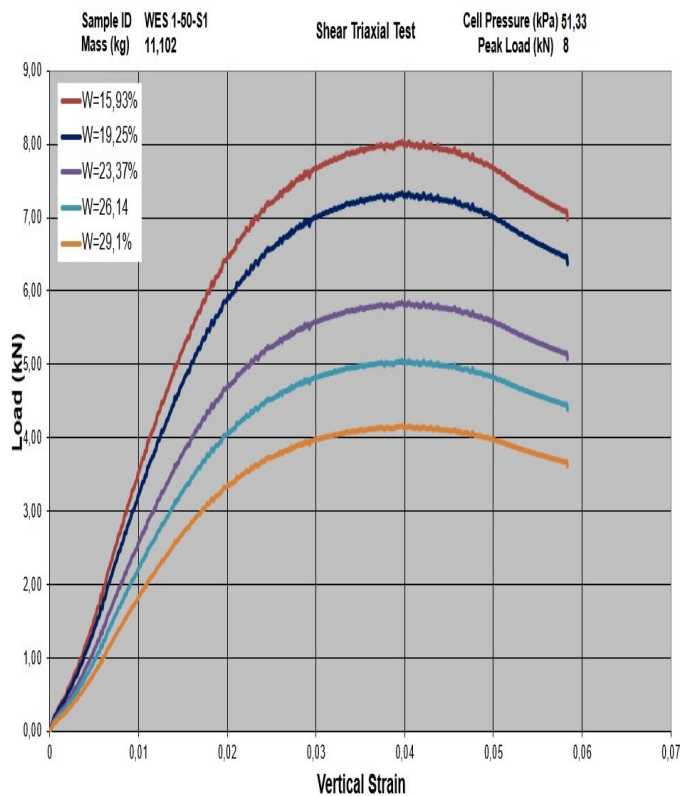


Figure 6.96: Stress-strain graph of WES-1
at $\sigma_c = 50$ kPa

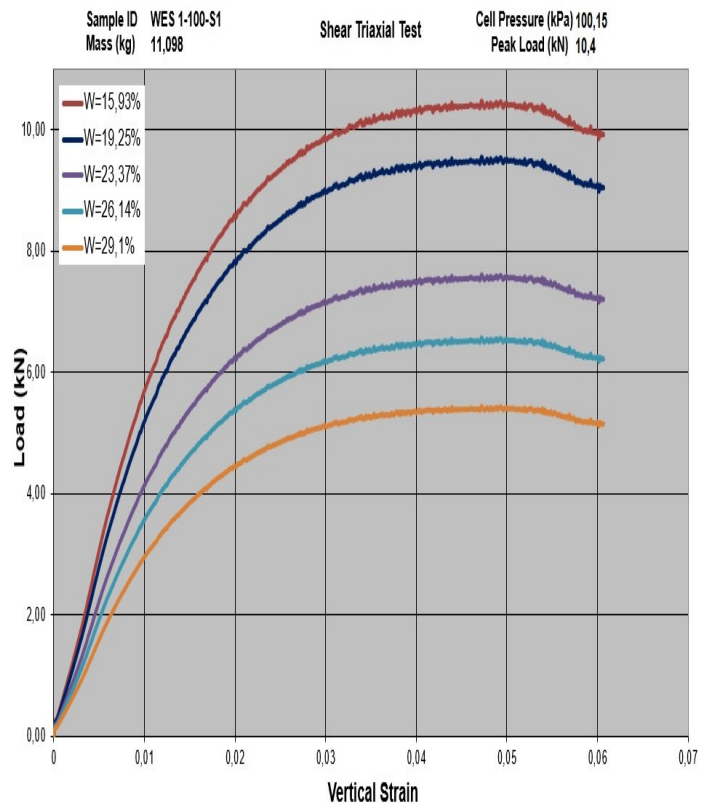


Figure 6.97: Stress-strain graph of WES-1
at $\sigma_c = 100$ kPa

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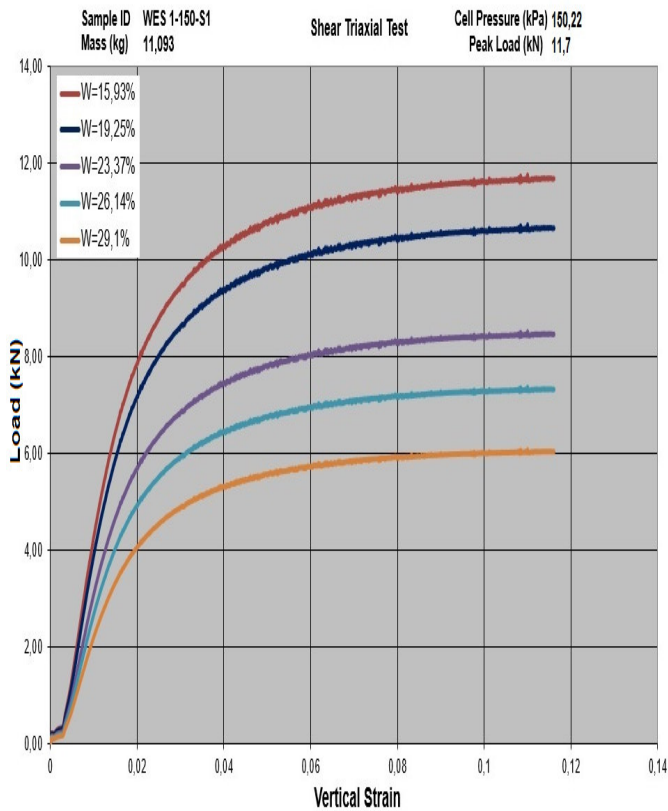


Figure 6.98: Stress-strain graph of WES-1
at $\sigma_c = 150$ kPa

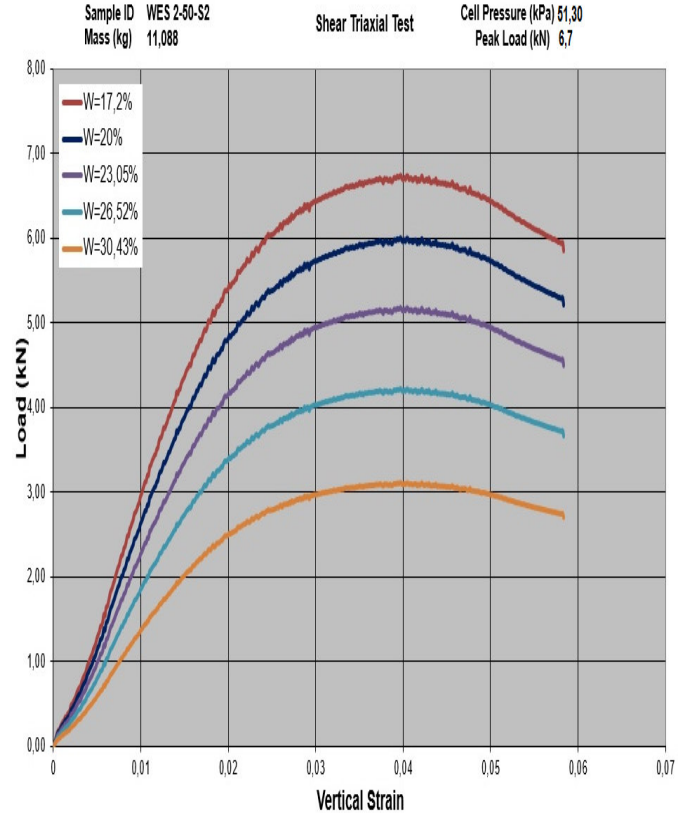


Figure 6.99: Stress-strain graph of WES-2
at $\sigma_c = 50$ kPa

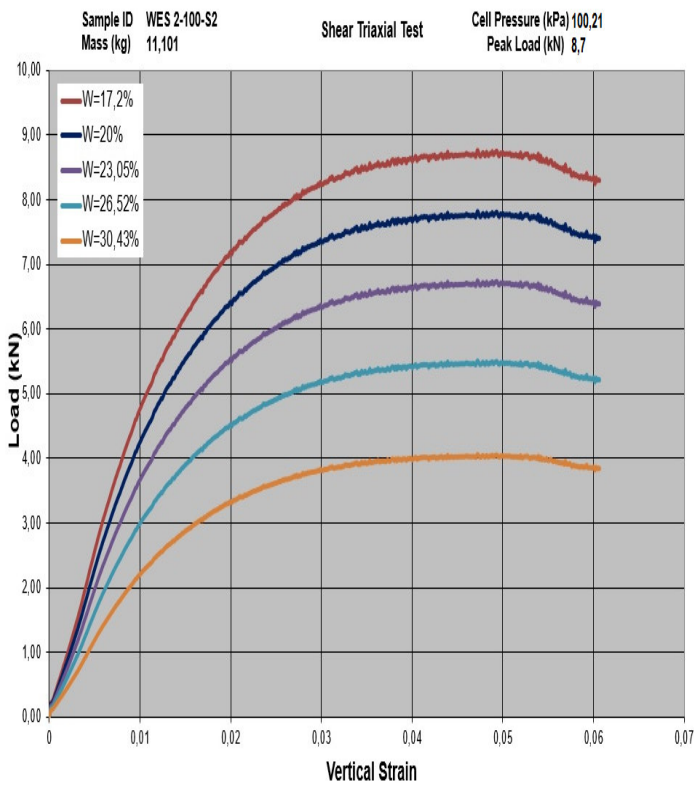


Figure 6.100: Stress-strain graph of WES-2
at $\sigma_c = 100$ kPa

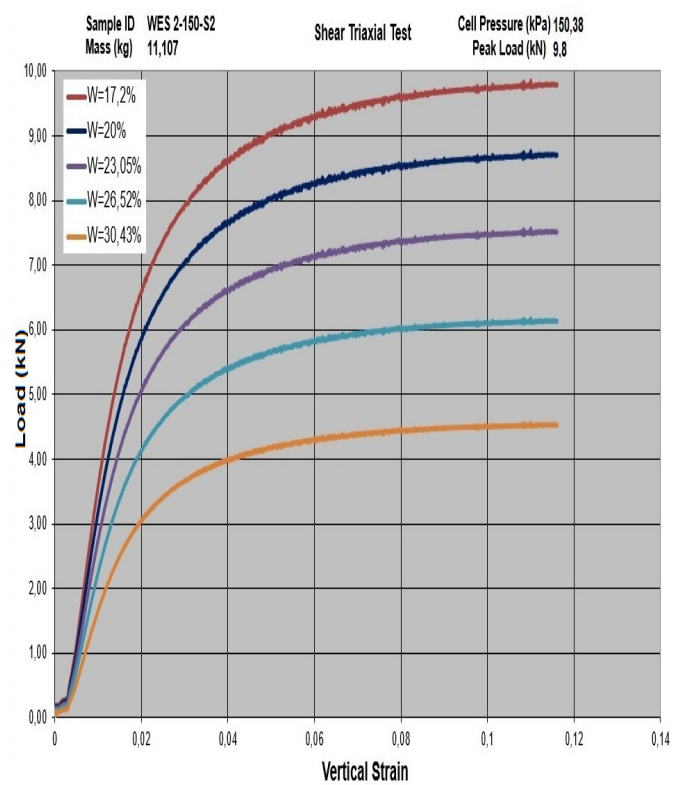


Figure 6.101: Stress-strain graph of WES-2 at
 $\sigma_c = 150$ kPa

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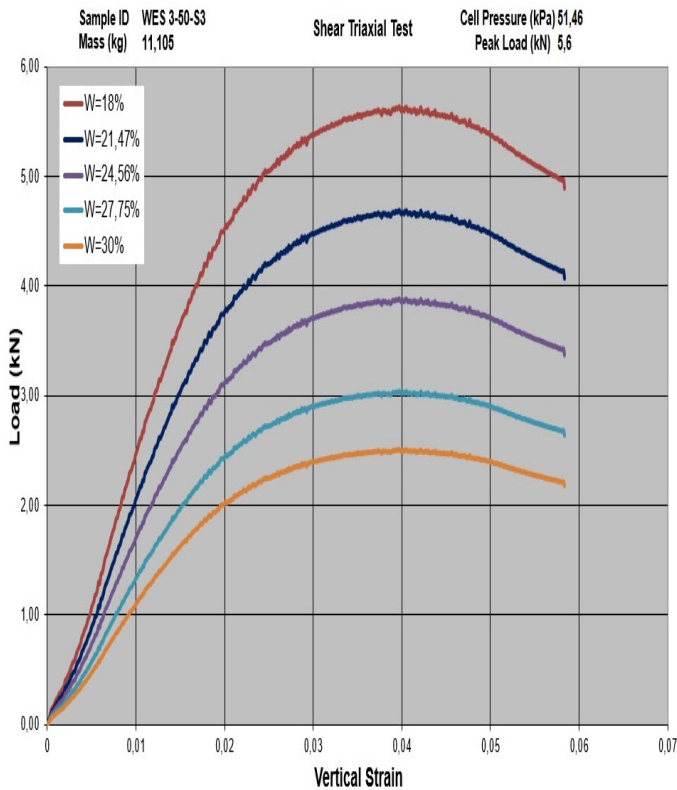


Figure 6.102: Stress-strain graph of WES-3 at $\sigma_c = 50$ kPa

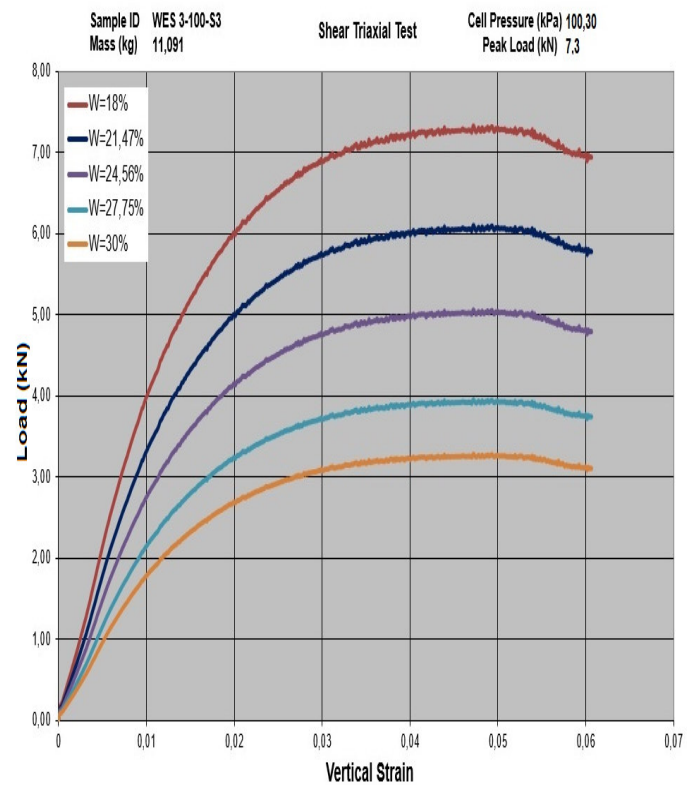


Figure 6.103: Stress-strain graph of WES-3 at $\sigma_c = 100$ kPa

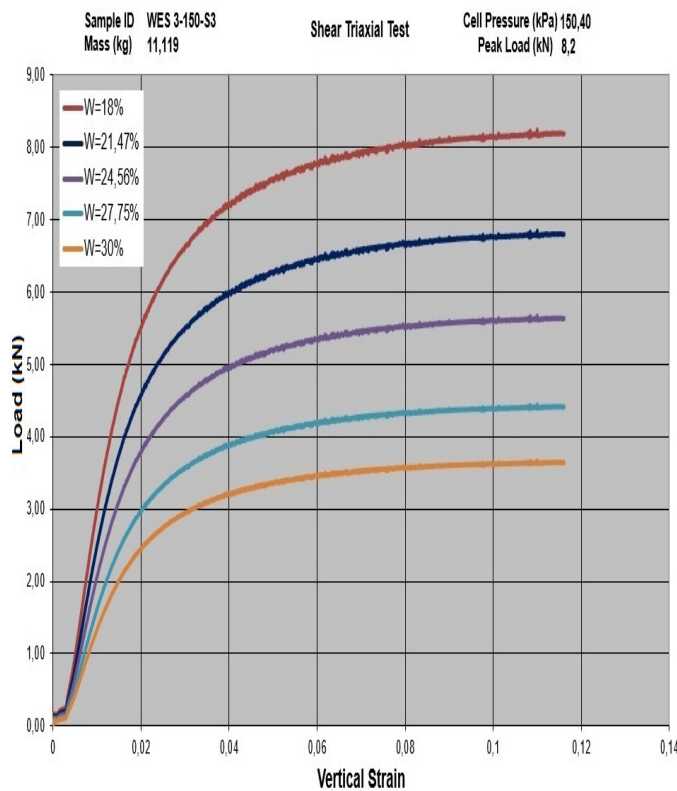


Figure 6.104: Stress-strain graph of WES-3 at $\sigma_c = 150$ kPa

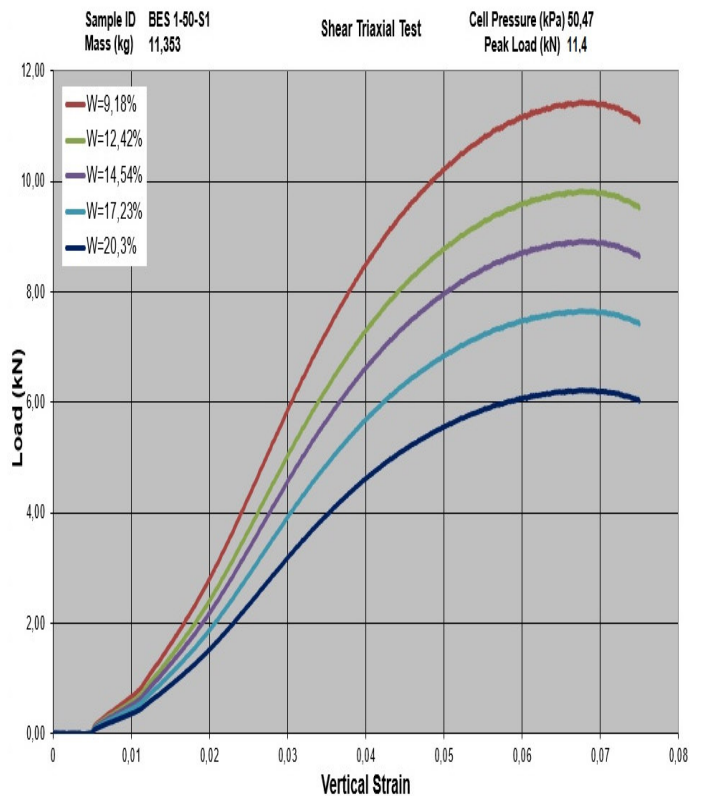


Figure 6.105: Stress-strain graph of BES-1 at $\sigma_c = 50$ kPa

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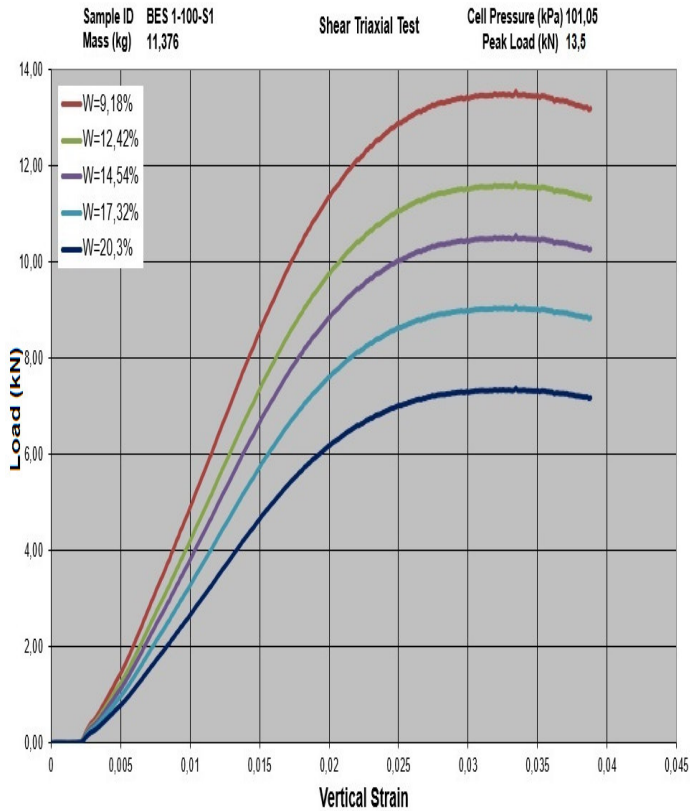


Figure 6.106: Stress-strain graph of BES-1
at $\sigma_c = 100$ kPa

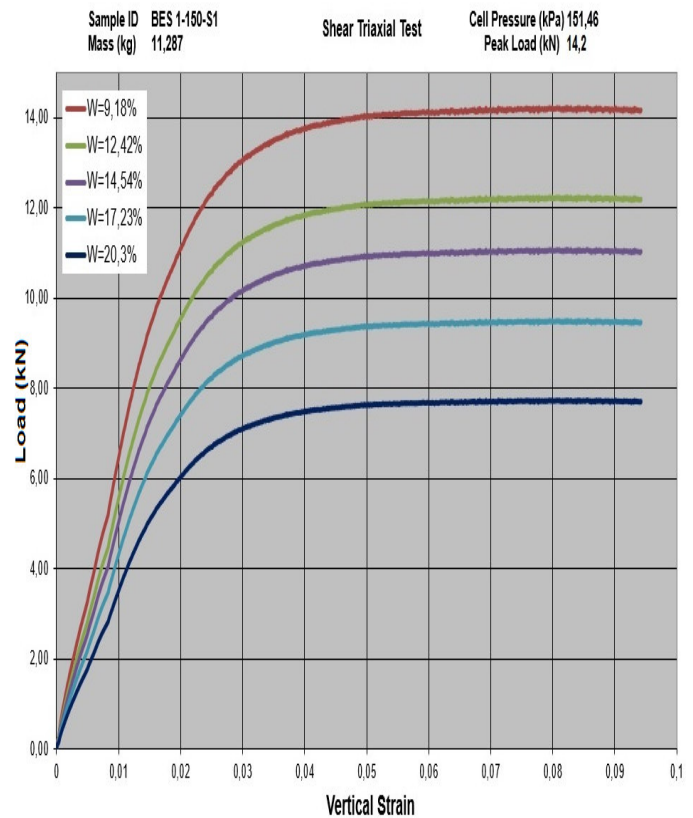


Figure 6.107: Stress-strain graph of BES-1
at $\sigma_c = 150$ kPa

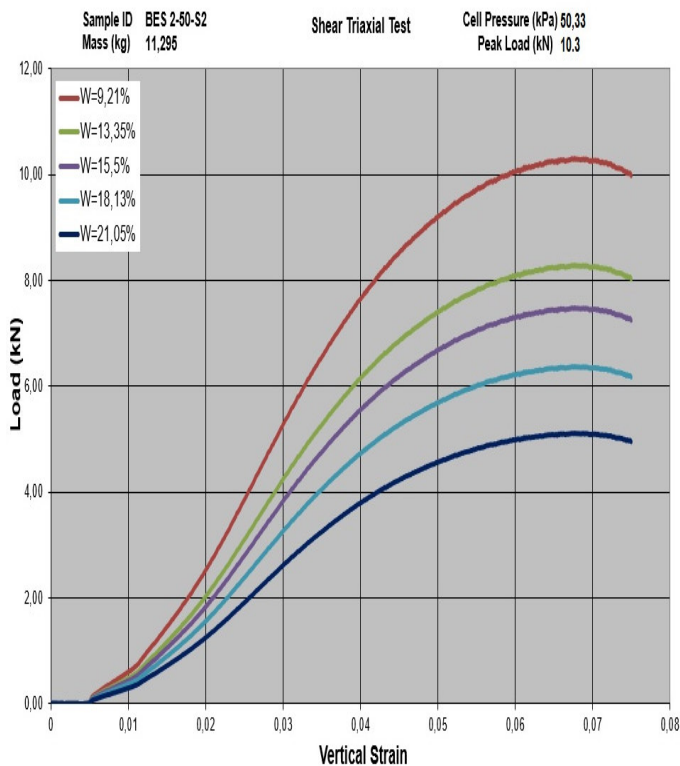


Figure 6.108: Stress-strain graph of BES-2
at $\sigma_c = 50$ kPa

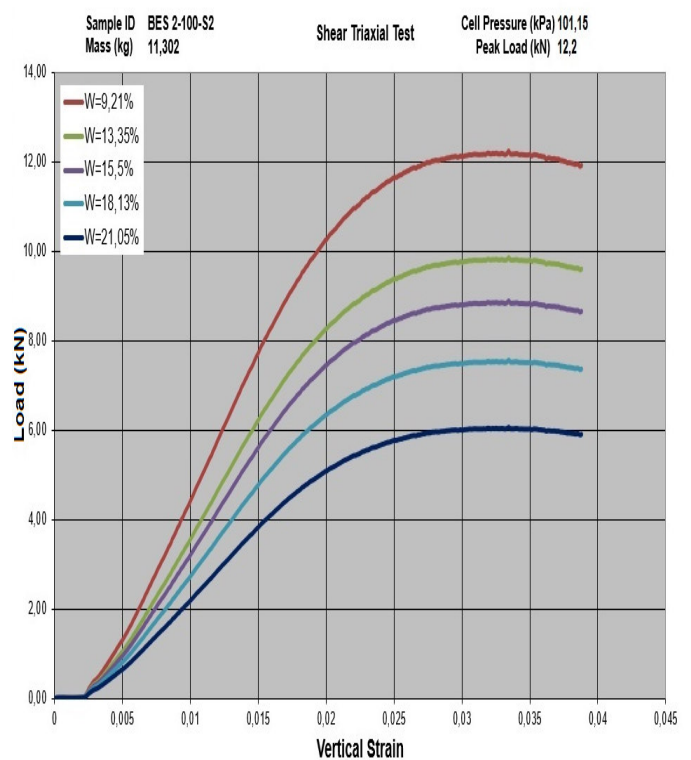


Figure 6.109: Stress-strain graph of BES-2
at $\sigma_c = 100$ kPa

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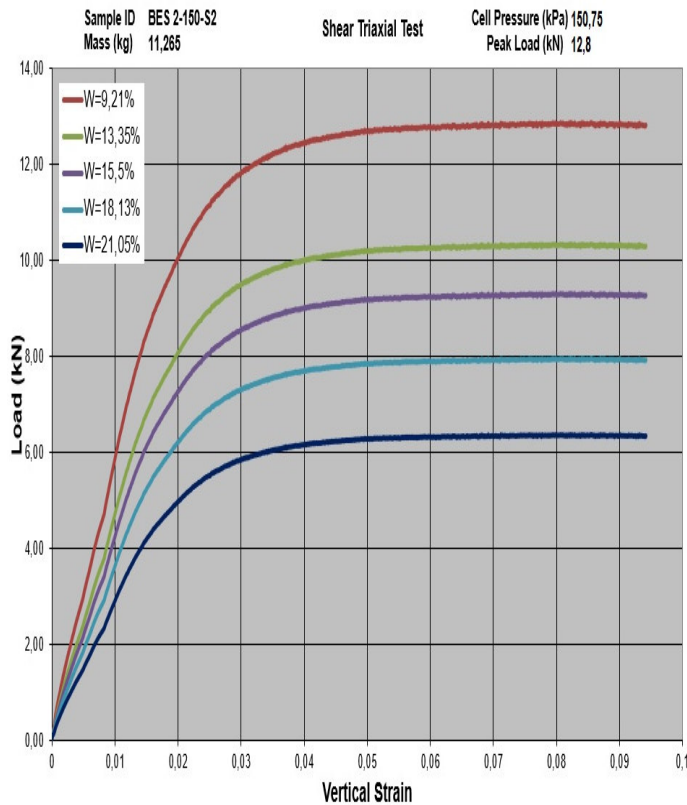


Figure 6.110: Stress-strain graph of BES-2
at $\sigma_c = 150$ kPa

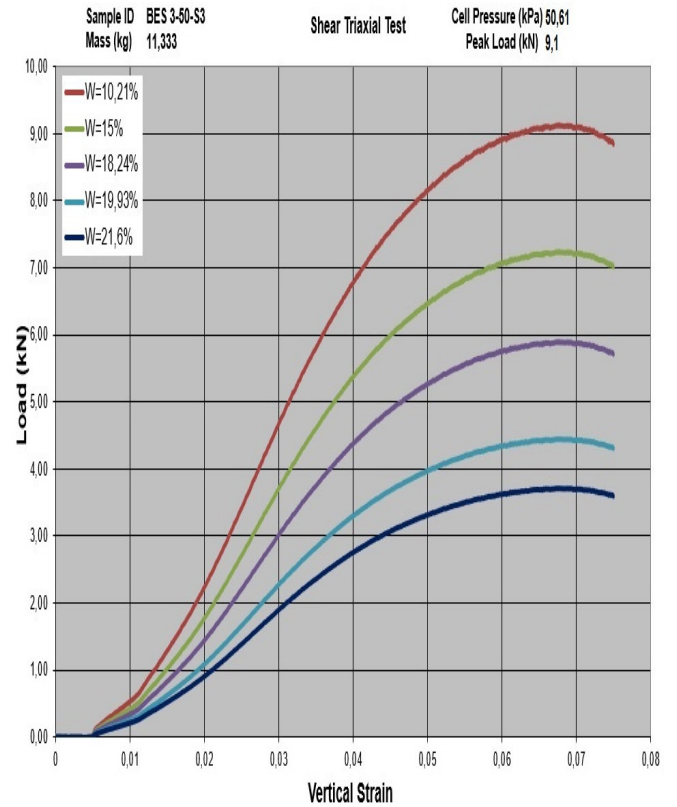


Figure 6.111: Stress-strain graph of BES-3
at $\sigma_c = 50$ kPa

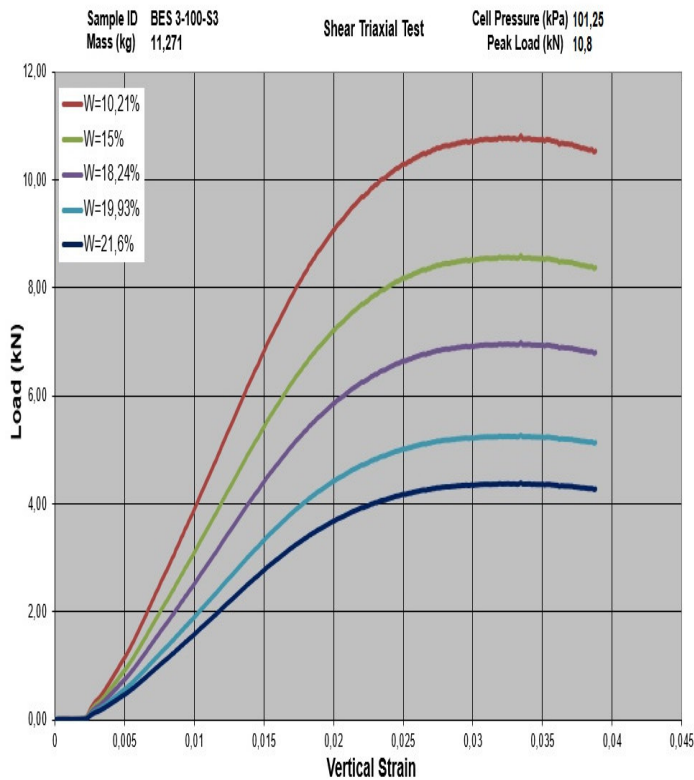


Figure 6.112: Stress-strain graph of BES-3
at $\sigma_c = 100$ kPa

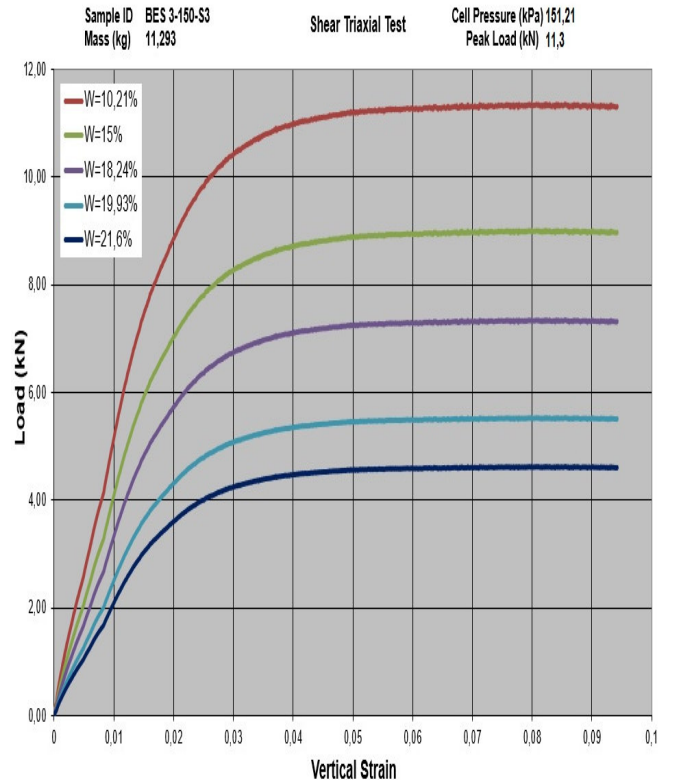


Figure 6.113: Stress-strain graph of BES-3
at $\sigma_c = 150$ kPa

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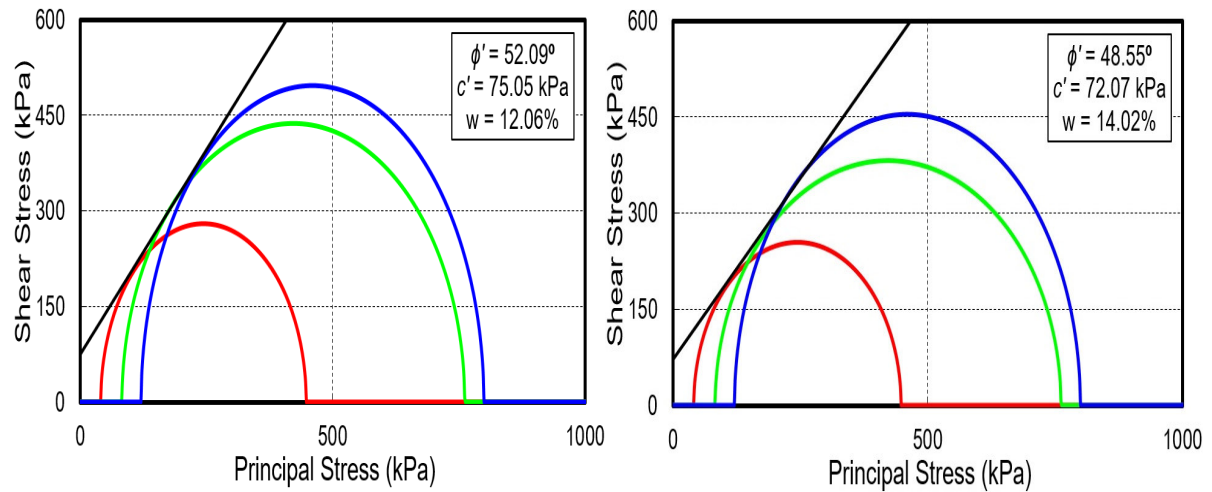


Figure 6.114: Failure envelopes of BFS-1 at 12.06% and 14.02% water content

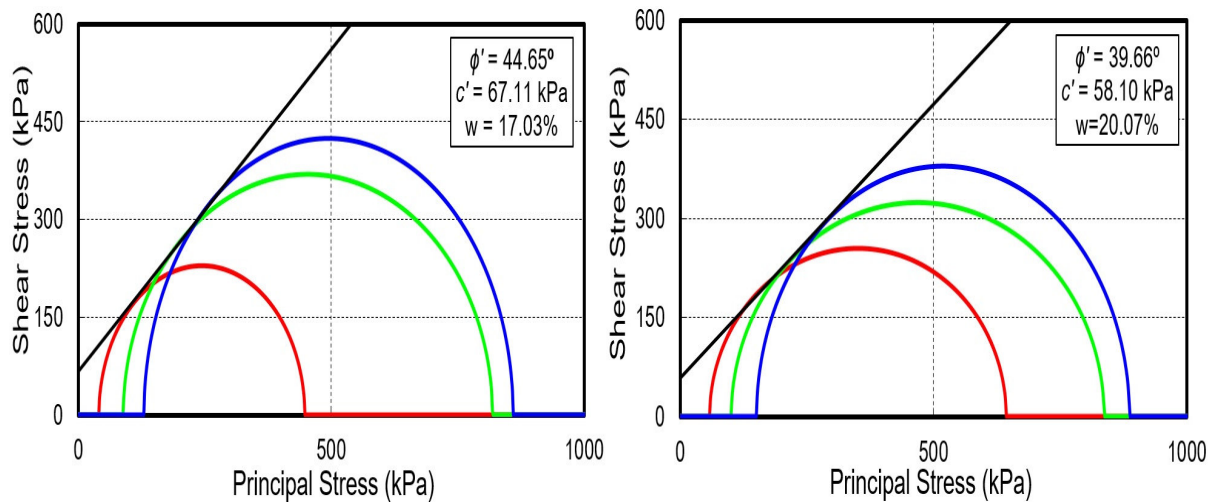


Figure 6.115: Failure envelopes of BFS-1 at 17.03% and 20.07% water content

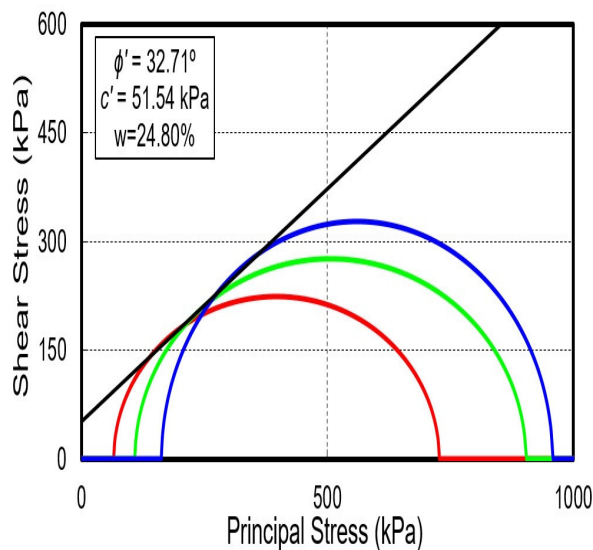


Figure 6.116: Failure envelope of BFS-1 at 24.80% water content

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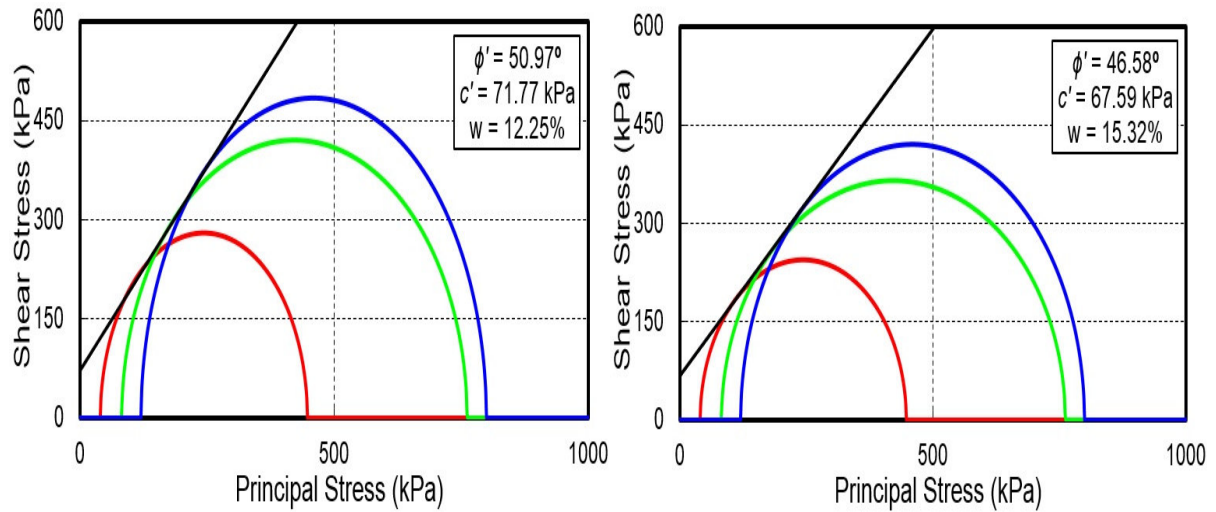


Figure 6.117: Failure envelopes of BFS-2 at 12.25% and 15.32% water content

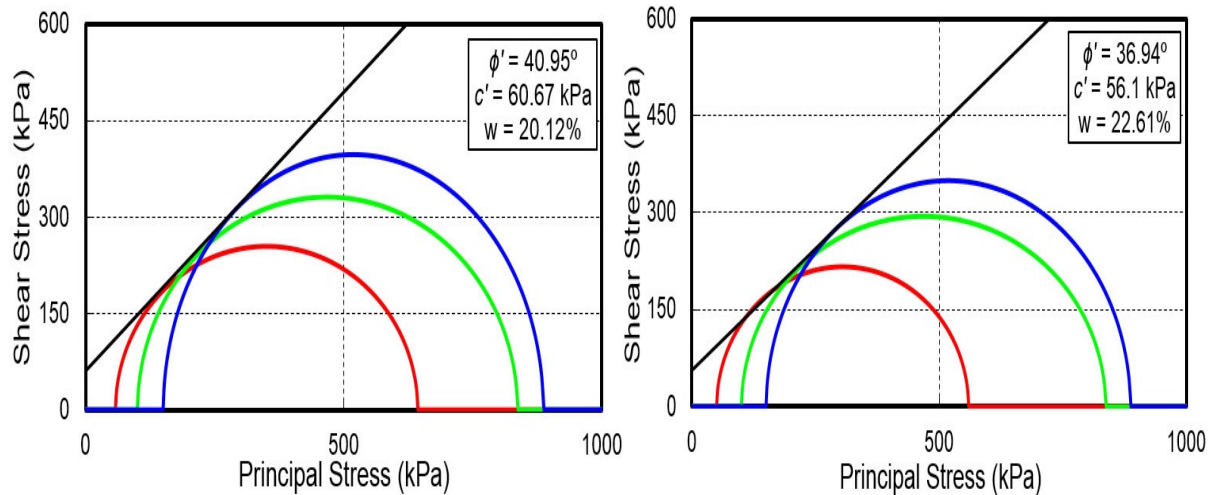


Figure 6.118: Failure envelopes of BFS-2 at 20.12% and 22.61% water content

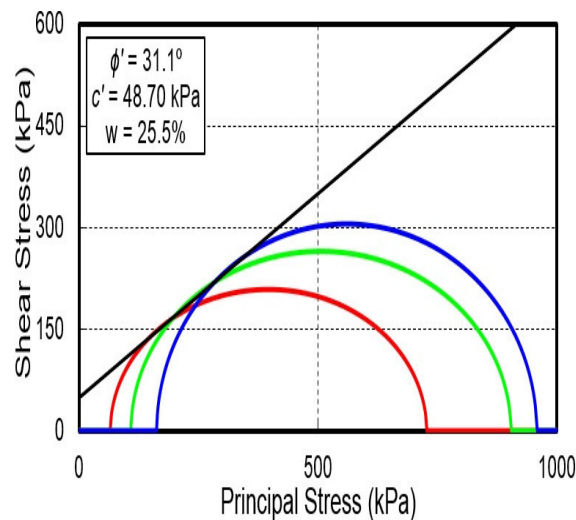


Figure 6.119: Failure envelope of BFS-2 at 25.50% water content

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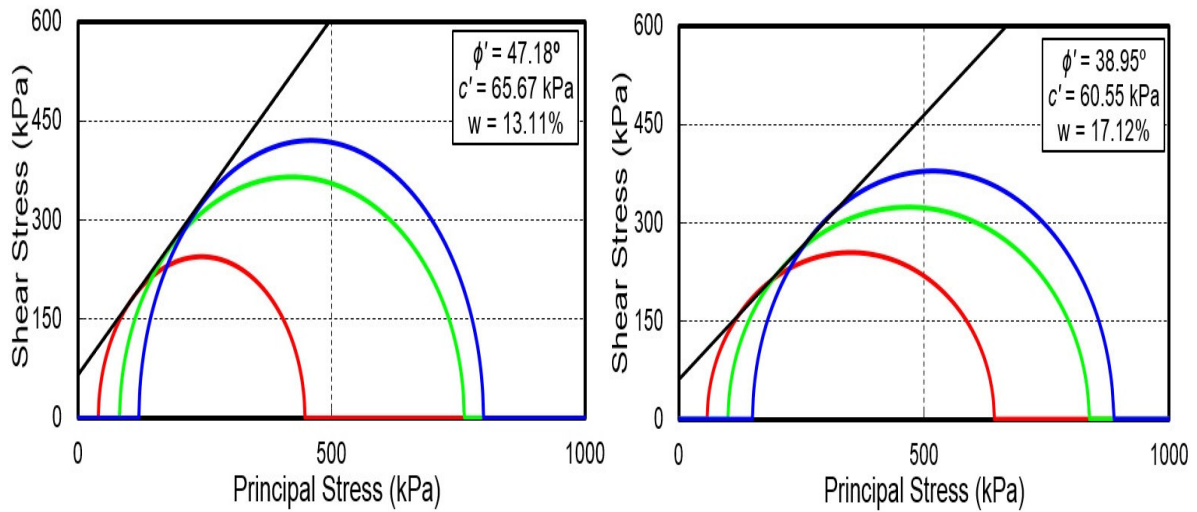


Figure 6.120: Failure envelopes of BFS-3 at 13.11% and 17.12% water content

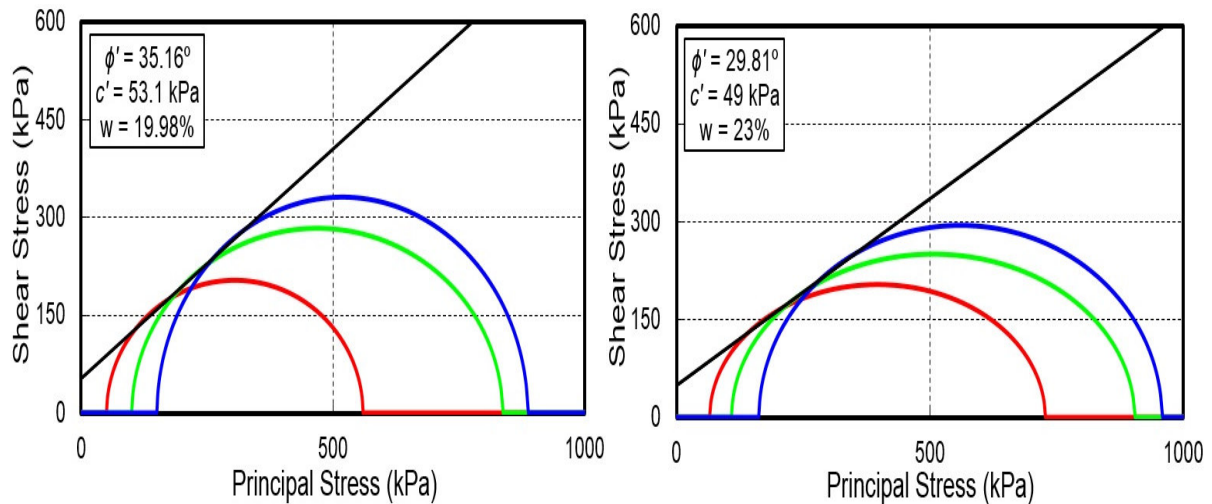


Figure 6.121: Failure envelopes of BFS-3 at 19.98% and 23% water content

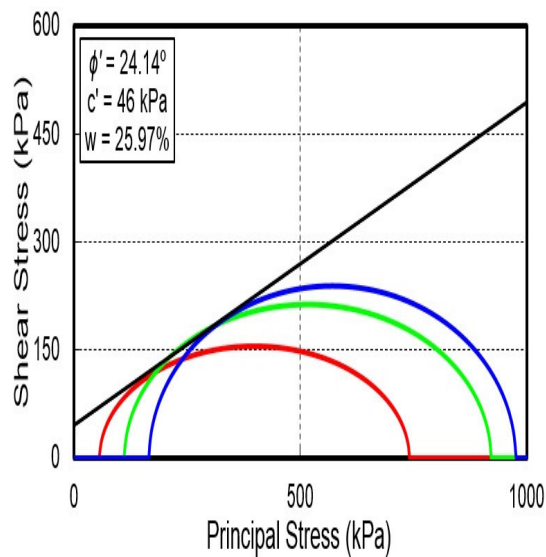


Figure 6.122: Failure envelope of BFS-3 at 25.97%

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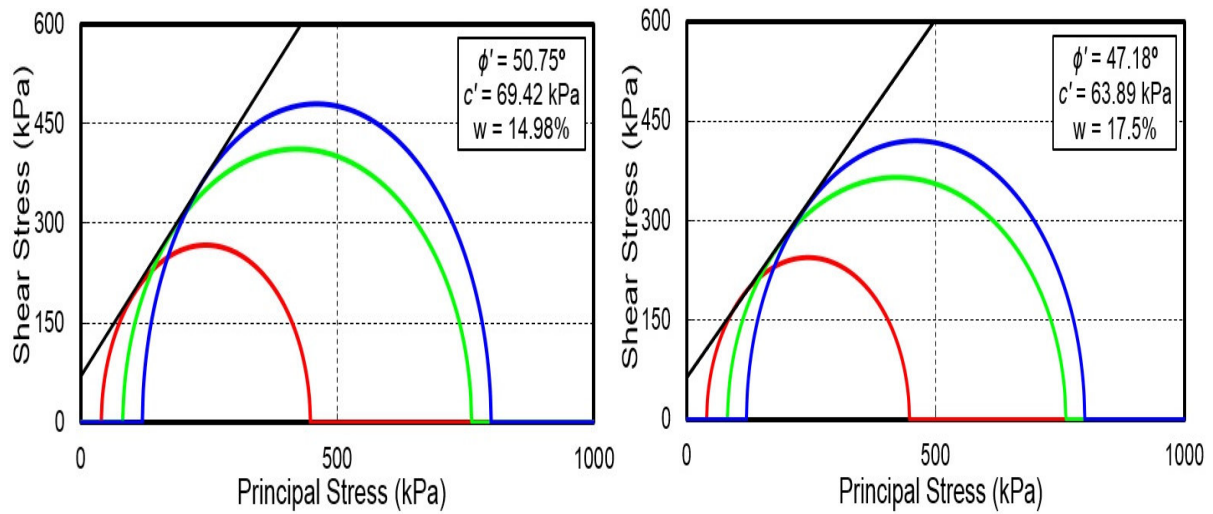


Figure 6.123: Failure envelopes of WIS-1 at 14.98% and 17.50% water content

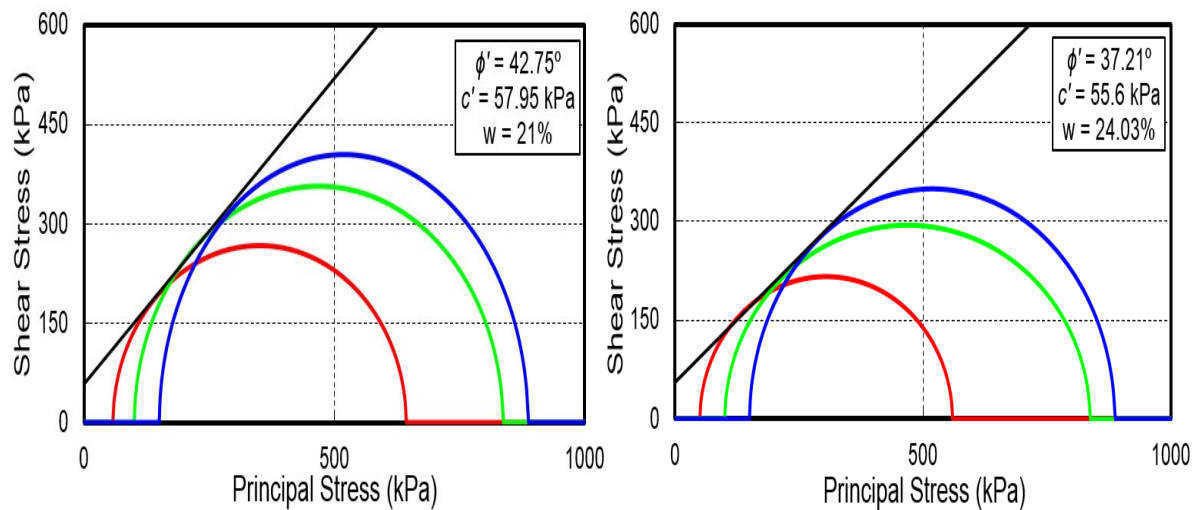


Figure 6.124: Failure envelopes of WIS-1 at 21% and 24.03% water content

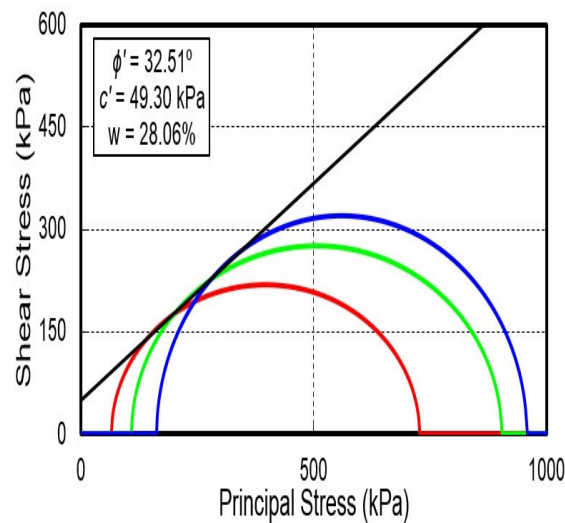


Figure 6.125: Failure envelope of WIS-1 at 28.06% water content

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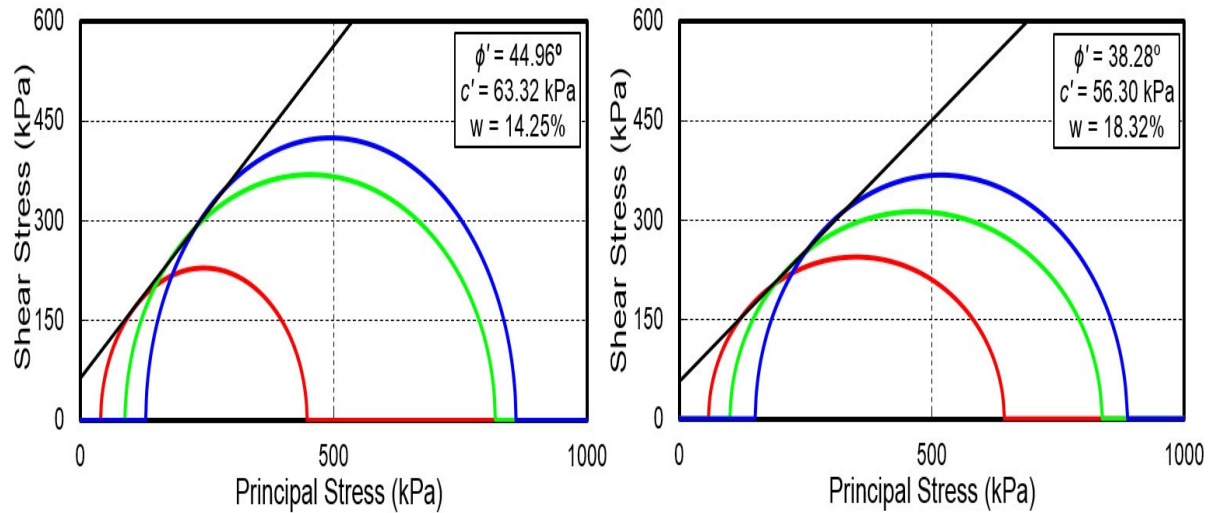


Figure 6.126: Failure envelopes of WIS-2 at 14.25% and 18.32% water content

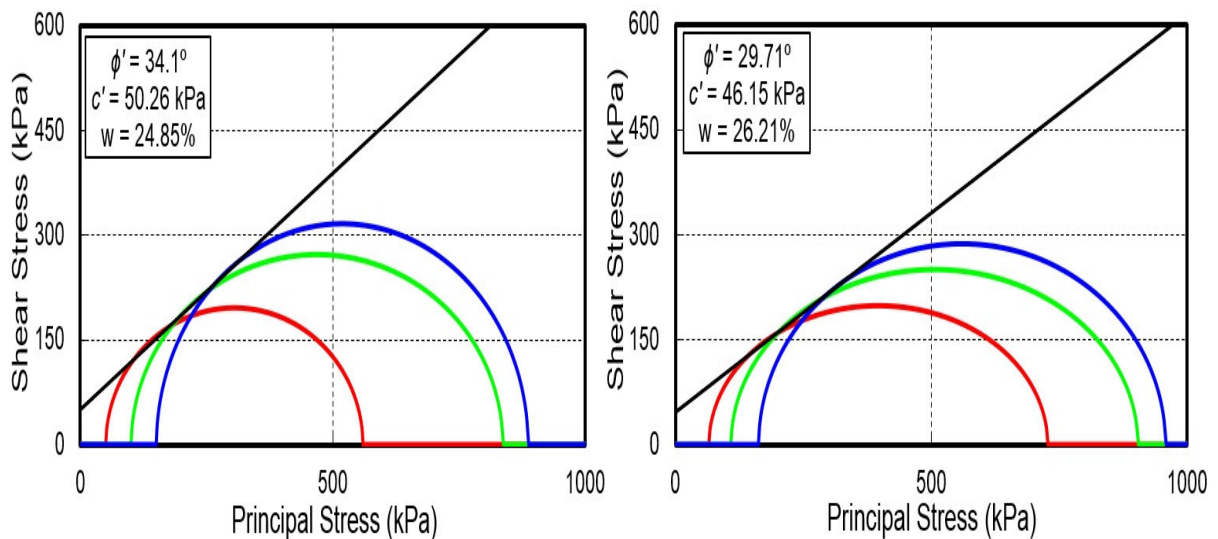


Figure 6.127: Failure envelopes of WIS-2 at 24.85% and 26.21% water content

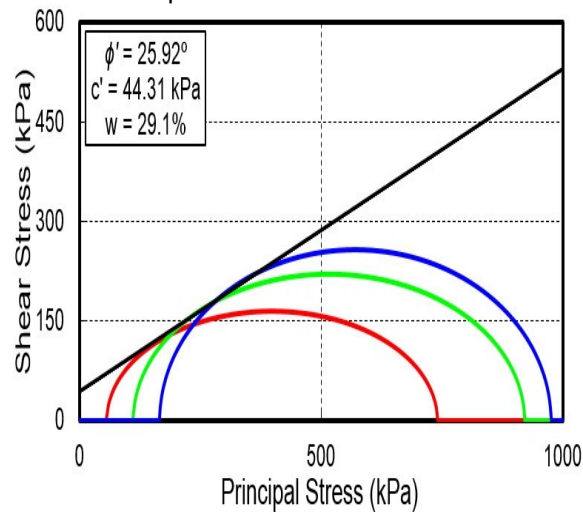


Figure 6.128: Failure envelope of WIS-2 at 29.10% water content

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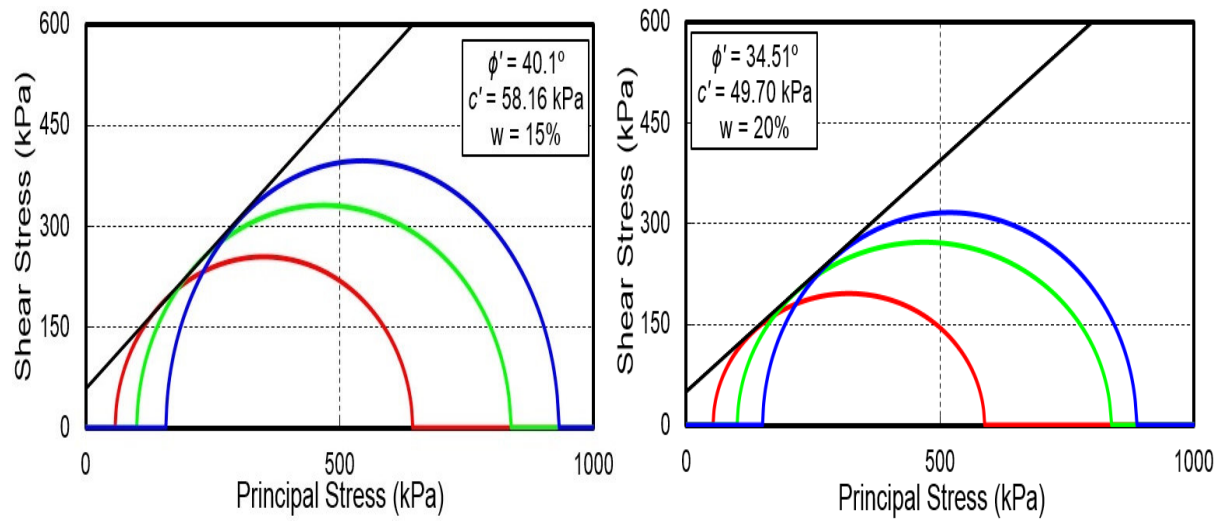


Figure 6.129: Failure envelopes of WIS-3 at 15.00% and 20% water content

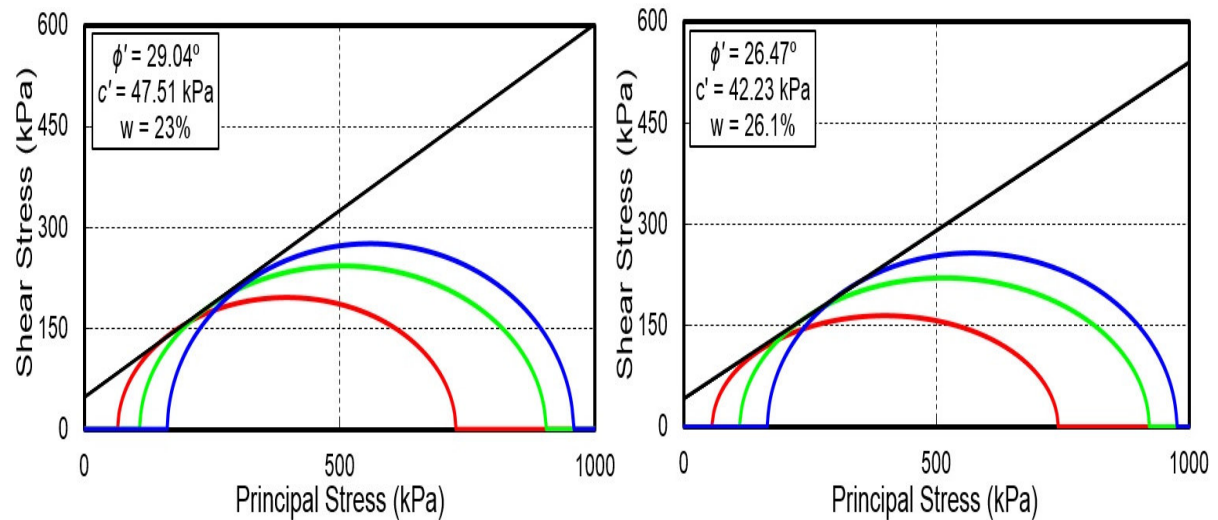


Figure 6.130: Failure envelopes of WIS-3 at 23% and 26.05% water content

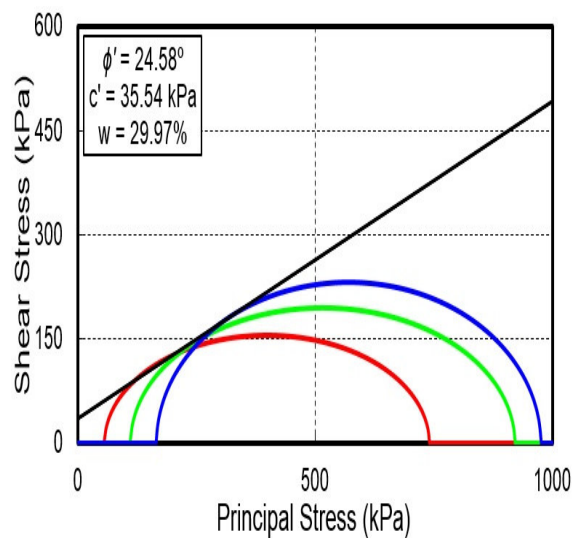


Figure 6.131: Failure envelope of WIS-3 at 29.97% water content

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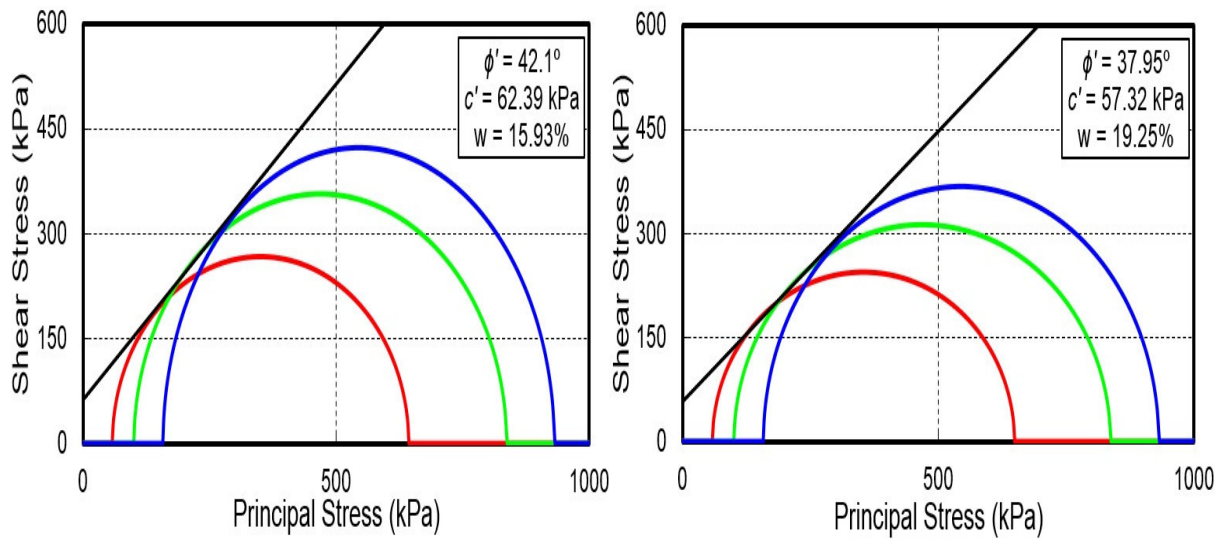


Figure 6.132: Failure envelopes of WES-1 at 15.93% and 19.25% water content

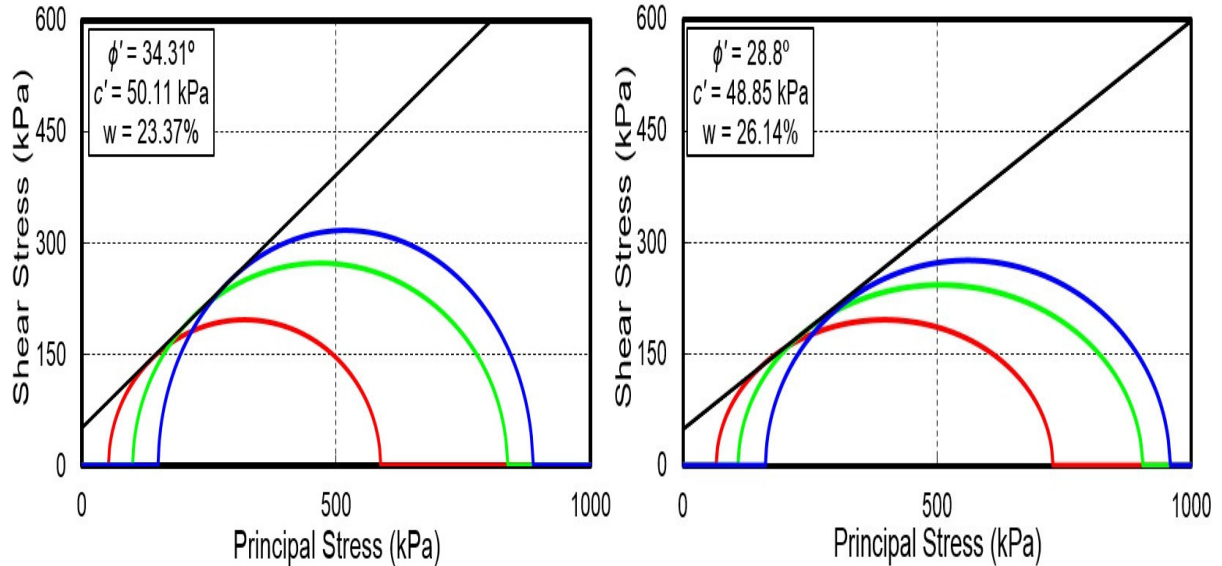


Figure 6.133: Failure envelopes of WES-1 at 23.37% and 26.14% water content

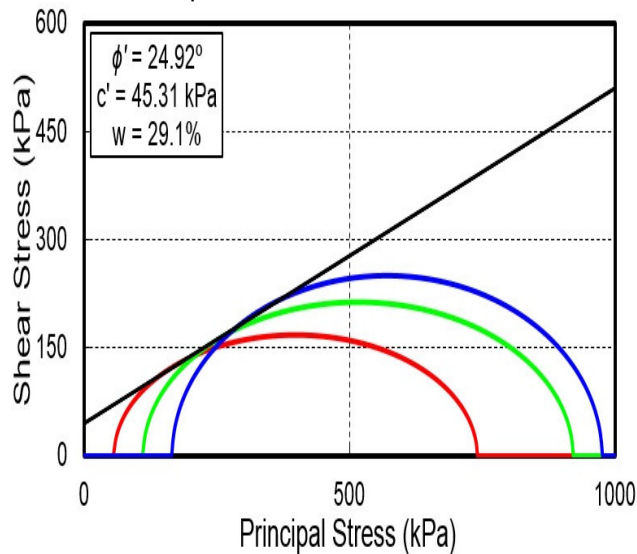


Figure 6.134: Failure envelope of WES-1 at 29.10%

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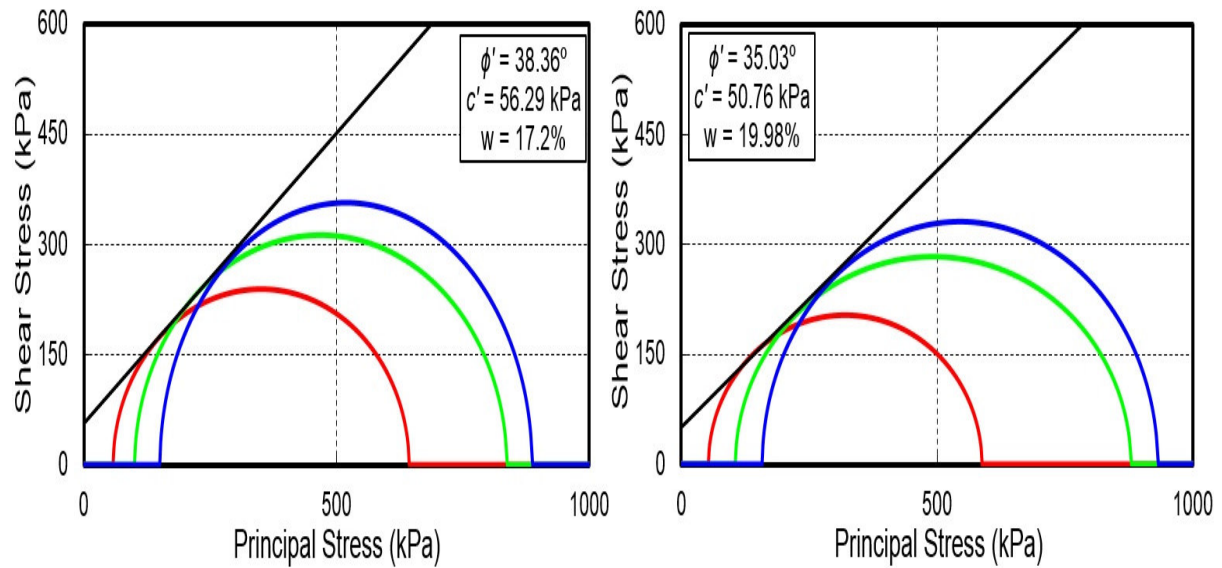


Figure 6.135: Failure envelopes of WES-2 at 17.20% and 19.98% water content

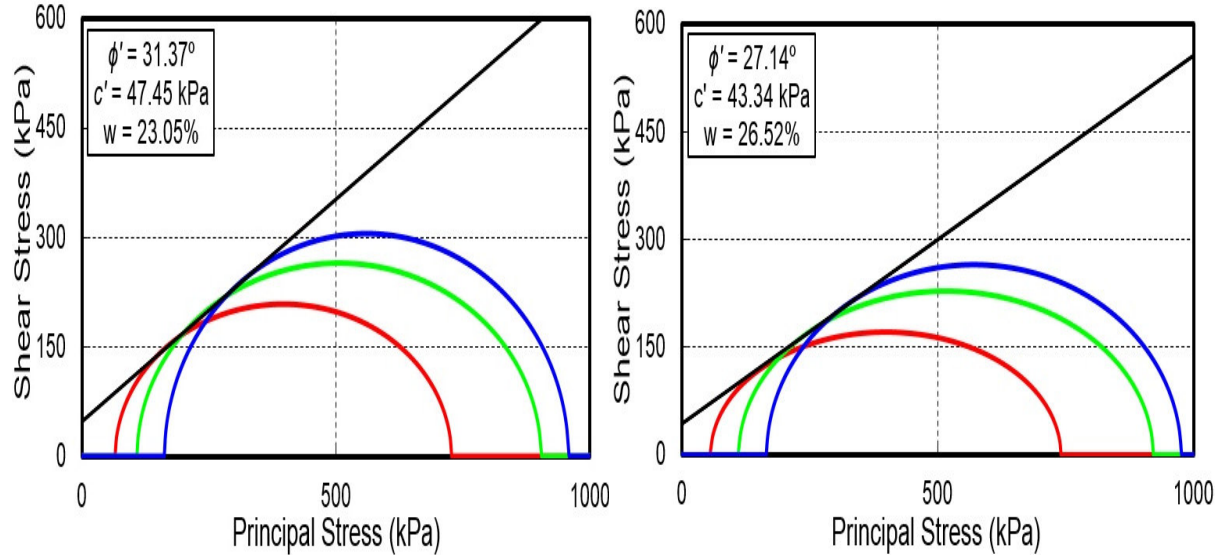


Figure 6.136: Failure envelopes of WES-2 at 23.05% and 26.52% water content

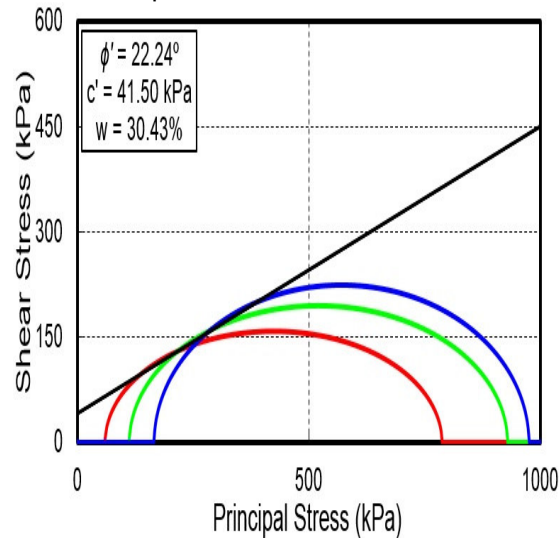


Figure 6.137: Failure envelope of WES-2 at 30.43% water content

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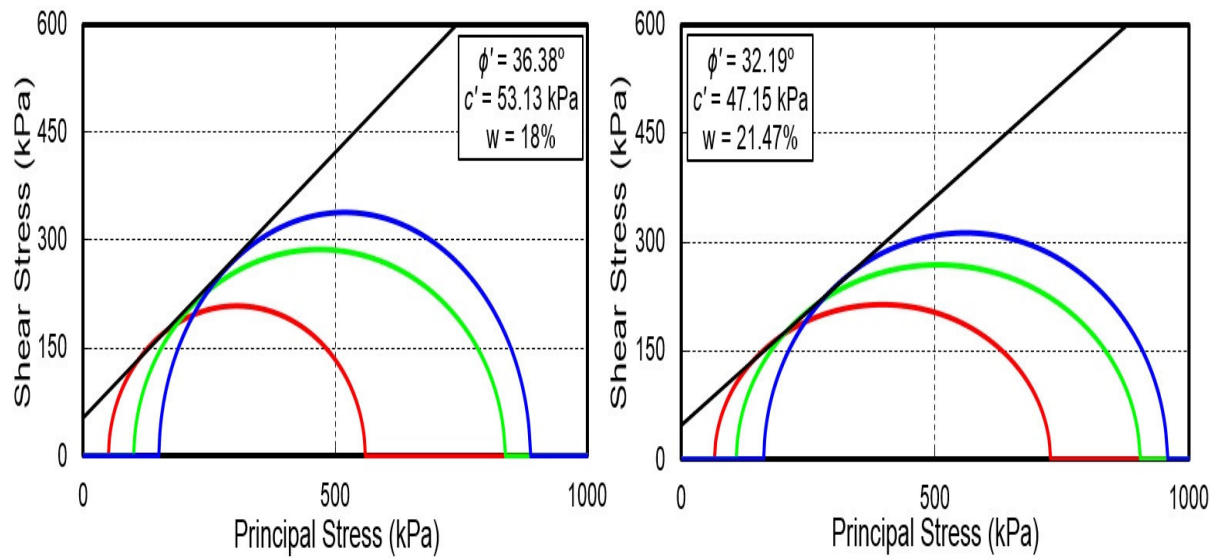


Figure 6.138: Failure envelopes of WES-3 at 18.00% and 21.47% water content

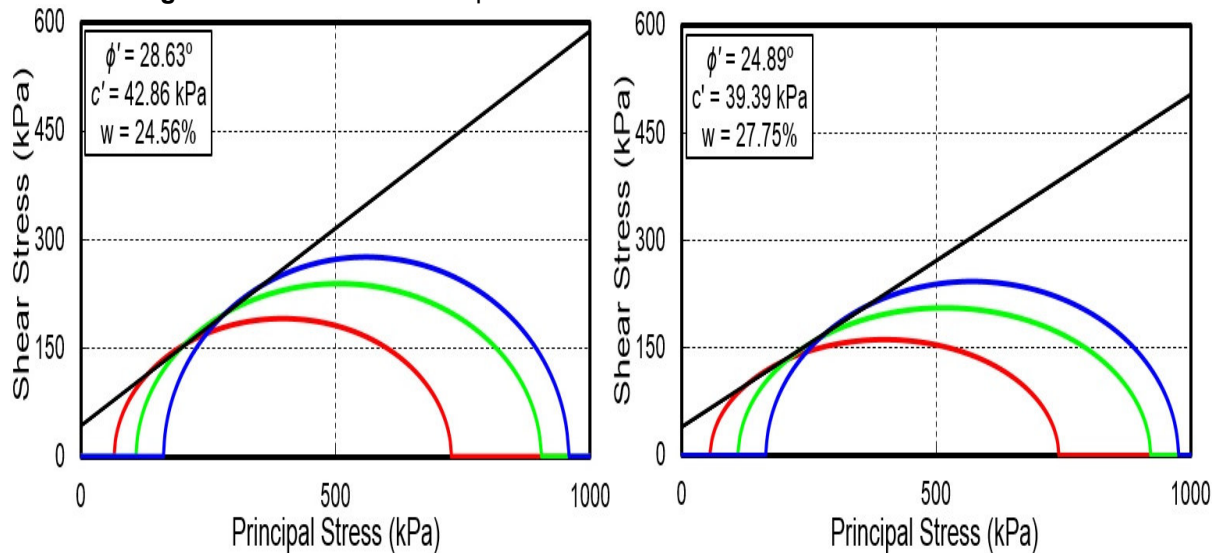


Figure 6.139: Failure envelopes of WES-3 at 24.56% and 27.75% water content

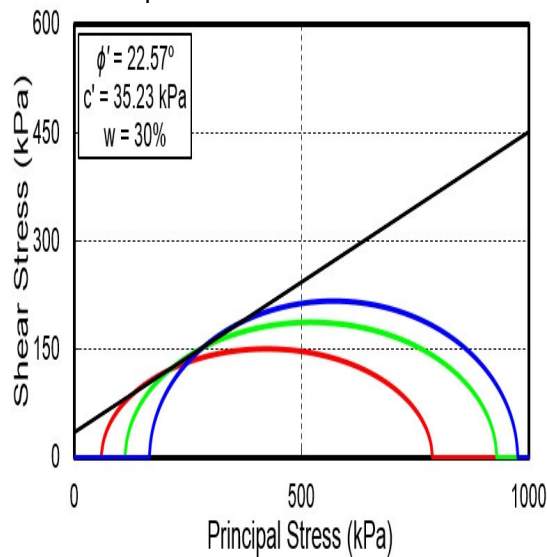


Figure 6.140: Failure envelope of WES-3 at 30% water content

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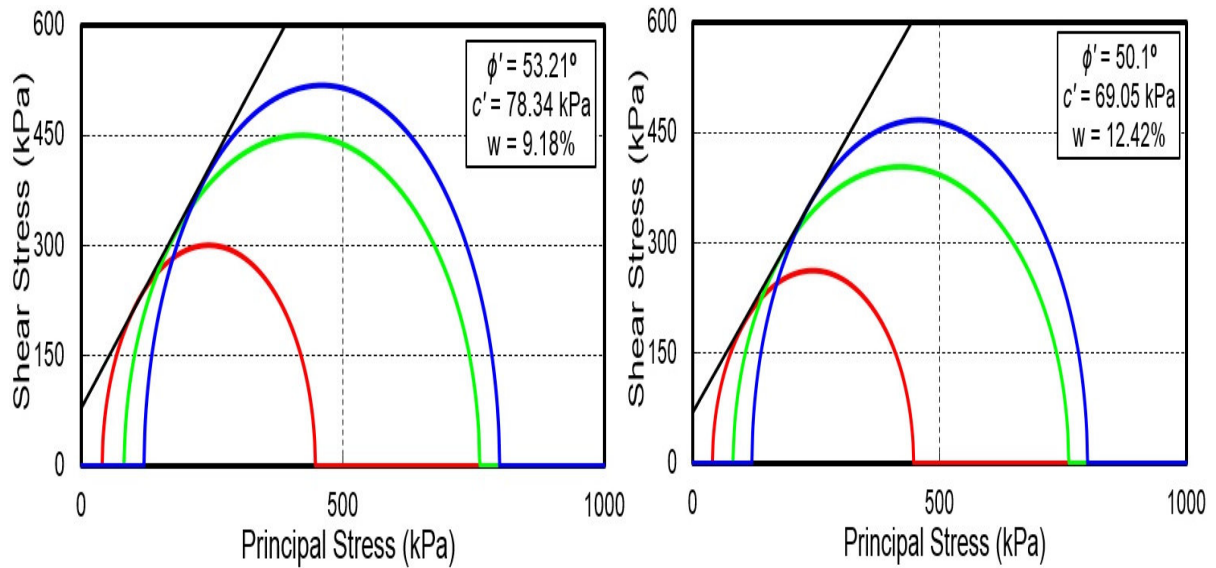


Figure 6.141: Failure envelopes of BES-1 at 9.18% and 12.42% water content

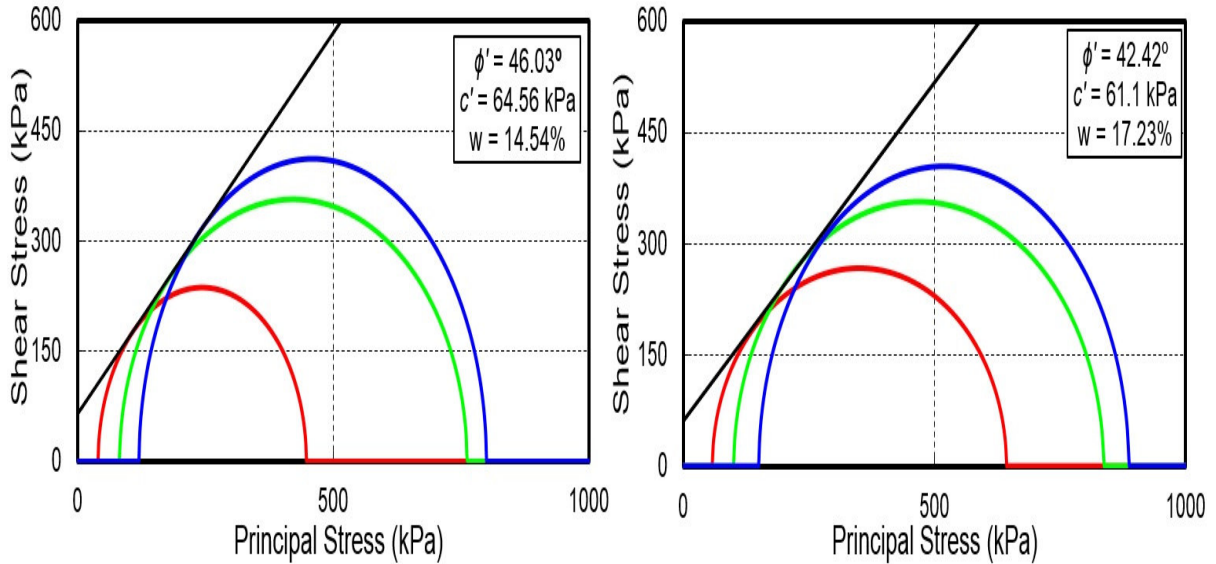


Figure 6.142: Failure envelopes of BES-1 at 14.54% and 17.23% water content

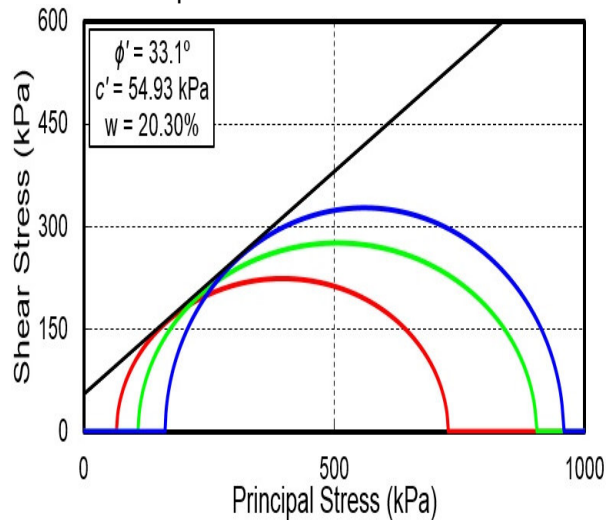


Figure 6.143: Failure envelope of BES-1 at 20.30% water content

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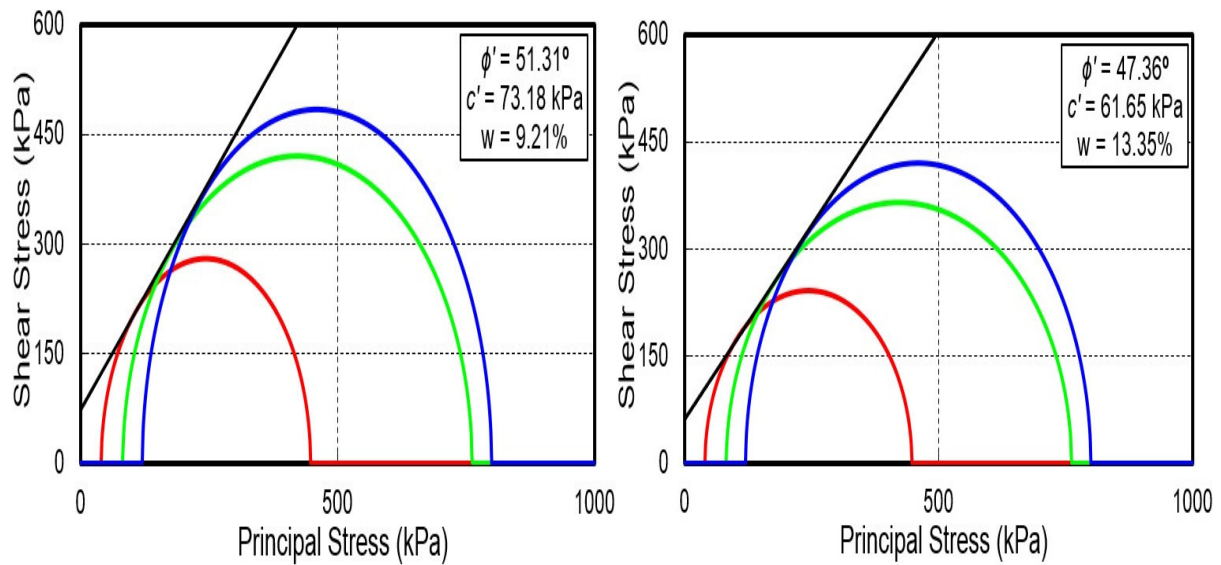


Figure 6.144: Failure envelopes of BES-2 at 9.21% and 13.35% water content

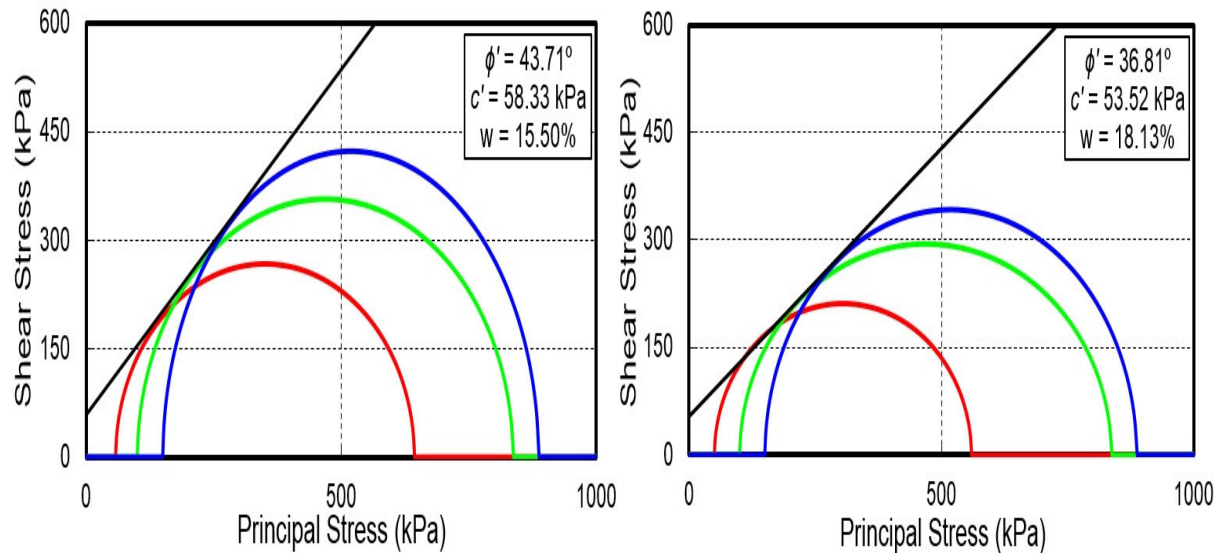


Figure 6.145: Failure envelopes of BES-2 at 15.50% and 18.13% water content

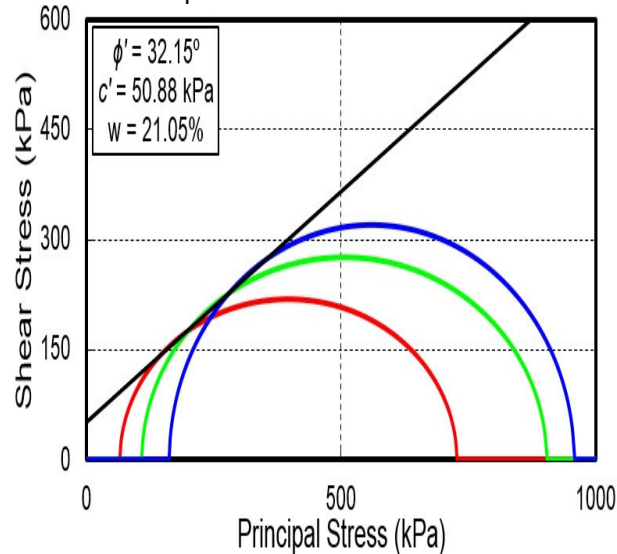


Figure 6.146: Failure envelope of BES-2 at 21.05% water content

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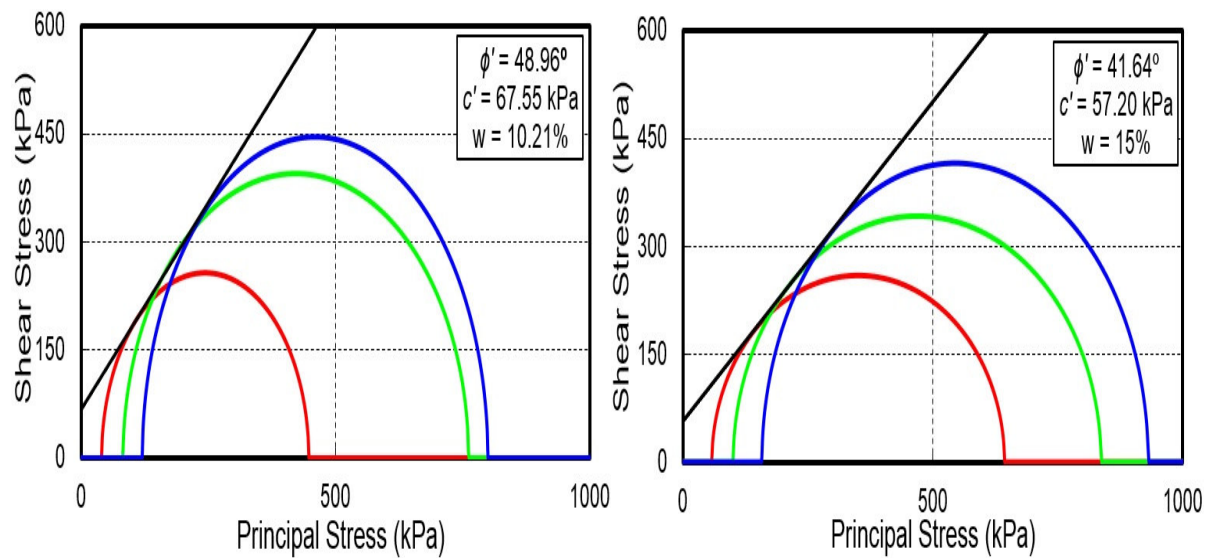


Figure 6.147: Failure envelopes of BES-3 at 10.21% and 15% water content

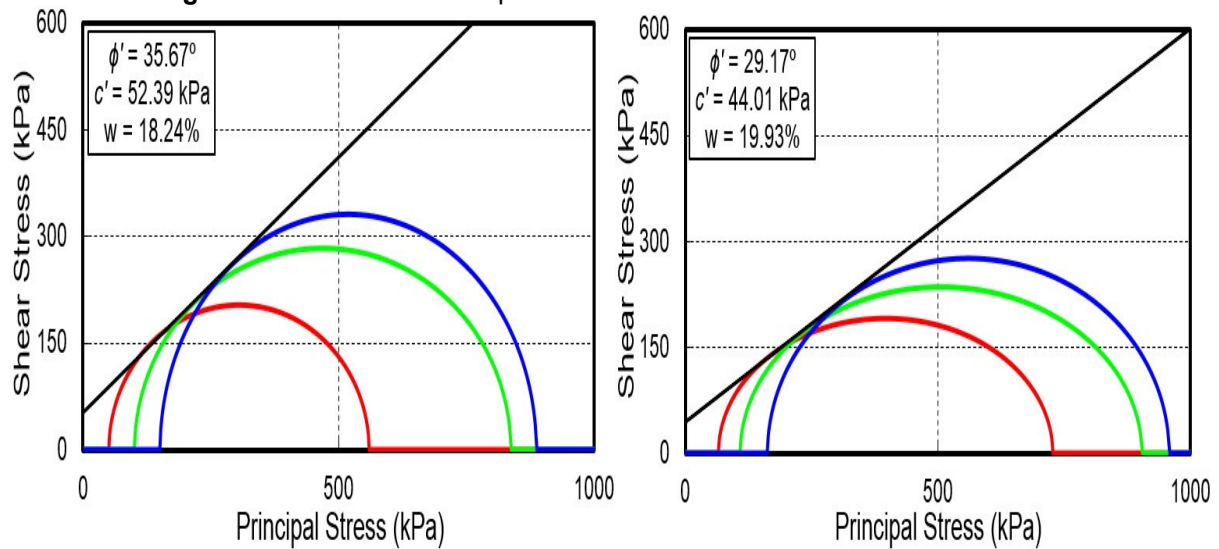


Figure 6.148: Failure envelopes of BES-3 at 18.24% and 19.93% water content

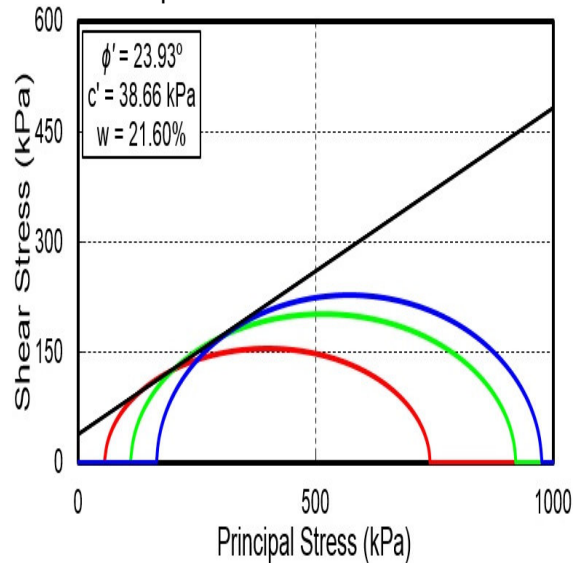


Figure 6.149: Failure envelope of BES-3 at 21.60% water content

APPENDIX 39

Table 6.22: Saturated and unsaturated shear strength for studied soils BFS, WIS

Soils	Water content (%)	c' (kPa)	$\tan(\phi')$ (Degree)	$(\sigma_n - u_a)_f$ (kPa)	τ_s (kPa)	$u_a - u_w$ (kPa)	$\tan(\phi^b)$ Degree	τ_u (kPa)	$\tau_u - \tau_s$ (kPa)
BFS-1	12.06	75.05	52.09	605.5	853	4741.62	6.77	1414.44	561.88
	14.02	72.05	48.55	547.78	692	3327.18	6.77	1086.56	394.27
	17.03	67.11	44.65	485.53	547	1388.22	6.77	711.25	164.50
	20.07	58.10	39.66	406.30	395	671.89	6.77	474.56	79.62
	24.80	51.54	32.71	333.87	266	200	6.77	289.67	23.70
BFS-2	12.25	71.77	50.97	565.88	770	4925.68	6.07	1292.45	522.62
	15.32	67.59	46.58	492.32	588	3456.34	6.07	954.56	366.72
	20.12	60.67	40.95	413.1	419	1442.11	6.07	572.14	153.01
	22.61	56.10	36.94	362.17	328	697.98	6.07	402.47	74.06
	25.50	48.70	31.07	312.93	237	207.79	6.07	259.29	22.05
BFS-3	13.11	65.67	47.18	492.32	597	5193.39	4.78	1033.20	436.25
	17.12	60.55	38.95	413.10	394	3644.19	4.78	700.58	306.11
	19.98	53.10	35.16	339.53	292	1520.49	4.78	419.98	127.72
	23	49.00	29.81	271.62	205	735.90	4.78	266.44	61.82
	25.97	46.00	25.14	212.21	146	219.08	4.78	163.99	18.40
WIS-1	14.98	69.42	50.75	537.59	727	5984.56	5.02	1253.44	526.04
	17.50	63.89	47.18	492.32	595	4332.68	5.02	976.02	380.84
	21	57.95	42.75	423.28	449	2748.30	5.02	690.80	241.58
	24.03	55.60	37.21	365.56	333	1199.35	5.02	438.60	105.42
	28.06	49.30	32.51	282.94	230	450.23	5.02	269.20	39.58
WIS-2	14.25	63.32	44.96	464.03	527	6213.23	3.56	912.54	385.84
	18.32	56.30	38.28	404.61	376	4498.23	3.56	654.95	279.34
	24.85	50.26	34.05	325.38	270	1853.32	3.56	385.24	115.09
	26.21	46.15	29.71	271.62	201	1245.20	3.56	278.47	77.33
	29.10	44.31	25.92	209.38	146	467.431	3.56	175.10	29.03
WIS-3	15	58.16	40.06	401.78	396	6628.04	2.49	682.34	286.33
	20	49.70	34.51	318.59	269	4798.54	2.49	476.04	207.30
	23	47.51	29.04	274.45	200	3043.81	2.49	331.39	131.49
	26.05	42.23	26.47	229.18	156	1328.33	2.49	213.73	57.38
	29.97	35.54	24.58	166.94	112	498.64	2.49	133.44	21.54

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Table 6.23: Saturated and unsaturated shear strength for studied soils WES, BES

Soils	Water content (%)	c' (kPa)	$\tan(\phi')$ (Degree)	$(\sigma_n - u_a)_f$ (kPa)	τ_s (kPa)	$u_a - u_w$ (kPa)	$\tan(\phi^b)$ Degree	τ_u (kPa)	$\tau_u - \tau_s$ (kPa)
WES-1	15.93	62.39	42.07	452.71	471	7693.67	2.67	830.31	359.29
	19.25	57.32	37.95	413.10	379	5227.78	2.67	623.62	244.14
	23.37	50.11	34.31	328.78	274	2986.46	2.67	413.94	139.47
	26.14	48.85	28.8	284.07	205	1778.65	2.67	288.08	83.06
	29.1	45.31	24.92	234.84	154	890.47	2.67	196.01	41.59
WES-2	17.2	56.29	38.36	379.14	356	8232.22	1.85	622.27	265.90
	19.98	50.76	35.03	337.83	288	5593.72	1.85	468.25	180.68
	23.05	47.45	31.37	291.43	225	3195.51	1.85	328.35	103.22
	26.52	43.34	27.14	237.67	165	1903.16	1.85	226.64	61.47
	30.43	41.50	22.24	175.42	113	952.80	1.85	144.01	30.78
WES-3	18	53.13	36.38	316.90	287	8745.48	1.41	498.24	211.64
	21.47	47.15	32.19	263.70	213	5942.48	1.41	356.96	143.81
	24.56	42.86	28.63	218.43	162	3394.74	1.41	244.25	82.15
	27.75	39.39	24.89	170.90	119	2021.81	1.41	167.61	48.93
	30	35.23	22.57	141.47	94	1012.20	1.41	118.53	24.50
BES-1	9.18	78.34	53.21	645.11	941	3312.26	10	1512.02	571.03
	12.42	69.05	50.08	554.57	732	1982.31	10	1073.58	341.75
	14.54	64.56	46.03	503.07	586	538.11	10	678.82	92.77
	17.23	61.09	42.42	431.77	456	225.61	10	494.52	38.90
	20.3	54.93	33.05	350.85	283	79.70	10	296.95	13.74
BES-2	9.21	73.18	51.31	582.86	801	3582.93	8.13	1313.69	512.72
	13.35	61.65	47.36	469.12	571	2144.29	8.13	877.95	306.85
	15.5	58.33	43.71	422.72	462	582.08	8.13	545.73	83.29
	18.13	53.52	36.81	360.47	323	250.77	8.13	359.17	35.89
	21.05	50.88	32.15	288.60	232	86.22	8.13	244.61	12.34
BES-3	10.21	67.55	48.96	514.96	659	3975.68	7.08	1150.89	491.79
	15	57.20	41.64	409.13	421	2379.35	7.08	715.28	294.33
	18.24	52.39	35.67	333.31	292	645.89	7.08	371.53	79.89
	19.93	44.01	29.17	250.69	184	222.79	7.08	211.5	27.56
	21.6	38.66	23.92	209.38	132	95.67	7.08	143.37	11.83

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Table 6.24: ϕ' response to soil properties correlation equations

Correlation equations	R ² (%)	Equation Number
$\phi' = 3739.6e^{-1.753Gs}$	71.54	6.43
$\phi' = 82.81e^{-0.04W}$	70.5	6.44
$\phi' = 68.87e^{-0.012LL}$	68.11	6.45
$\phi' = 73.63e^{-0.012Fine}$	72.39	6.47
$\phi' = 51.04e^{-0.011PI}$	63.79	6.46
$\phi' = 52.34e^{-0.014Clay}$	73.87	6.48
$\phi' = 5.059e^{0.108\gamma_d}$	61.89	6.49
$\phi' = 57.16e^{-0.932Void\ ratio}$	69.35	6.50
$\phi' = 63.49e^{-0.368FSR}$	74.39	6.51
$\phi' = 46.72e^{-0.004FSI}$	70.07	6.52
$\phi' = 56.134e^{-0.01Smectite}$	60.59	6.53
$\phi' = 26.81e^{0.0574\ Plagioclase}$	52.51	6.54

Table 6.25: ϕ^b response to soil properties correlations equations

Correlation equations	R ² (%)	Equation Number
$\phi^b = 106.51e^{-0.05Fine}$	91.96	6.55
$\phi^b = 27.04e^{-0.054Clay}$	92.17	6.56
$\phi^b = 84.48e^{-0.048LL}$	92.60	6.57
$\phi^b = 25.95e^{-0.046PI}$	91.63	6.58
$\phi^b = 5.E + 08e^{-6.883Gs}$	87.91	6.59
$\phi^b = 0.0022e^{0.438\gamma_d}$	80.84	6.60
$\phi^b = 166.47e^{-0.161W}$	88.48	6.61
$\phi^b = 20.29e^{-0.119AEV}$	91.54	6.62
$\phi^b = 18\ e^{-0.018FSI}$	92.58	6.63
$\phi^b = 37.03e^{-0.038Smectite}$	78.9	6.64
$\phi^b = 1.884e^{0.229Plagioclase}$	66.83	6.65
$\phi^b = 1.167e^{0.0833K-feldspar}$	65.79	6.66

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Table 6.26: ϕ^s response to soil properties correlation equations

Correlation equations	R ² (%)	Equation Number
$\phi^s = 407e^{-0.095W}$	70.44	6.67
$\phi^s = 4E + 06e^{-4.167Gs}$	70.05	6.68
$\phi^s = 259.71e^{-0.028LL}$	56.53	6.69
$\phi^s = 133.95e^{-0.027PI}$	57.05	6.70
$\phi^s = 327.53e^{-0.03Fine}$	64.99	6.71
$\phi^s = 152.74e^{-0.035Clay}$	74.52	6.72
$\phi^s = 0.623e^{0.248\gamma_d}$	50.38	6.73
$\phi^s = 171.6e^{-2.197Void\ ratio}$	59.54	6.74
$\phi^s = 255.35e^{-0.959FSR}$	76.43	6.75
$\phi^s = 119e^{-0.012FSI}$	76.45	6.76
$\phi^s = 161.07e^{-0.022Smectite}$	50.38	6.77
$\phi^s = 19.06e^{0.056K-feldspar}$	58.21	6.78

Table 6.27: c' response to soil properties correlation equations

Correlation equations	R ² (%)	Equation Number
$c' = 98.71e^{-0.03W}$	73.01	6.79
$c' = 1887e^{-1.341Gs}$	70.78	6.80
$c' = 86.53e^{-0.009LL}$	61.1	6.81
$c' = 69.26e^{-0.008PI}$	56.13	6.82
$c' = 91.84e^{-0.009Fine}$	67.65	6.83
$c' = 71.59e^{-0.01Clay}$	70.9	6.84
$c' = 75.759e^{-0.629Void\ ratio}$	63.49	6.85
$c' = 13.201e^{0.0776\gamma_d}$	53.6	6.86
$c' = 73.71e^{-0.007Smectite}$	51.57	6.87
$c' = 65.22e^{-0.003FSI}$	62.35	6.88
$c' = 39.19e^{0.0162K-feldspar}$	53.23	6.89
$c' = 43.63e^{0.0407Plagioclase}$	50.47	6.90

APPENDIX 43

Table 6.28: Modified-unsaturated shear strength for studied soils BFS, WIS

Soils	Water Content (%)	τ_u (kPa)	P_s (kPa)	$\tan(\phi^s)$ Degree	$\tau_u - \tau_{mu} = P_s \tan(\phi^s)$ (kPa)	τ_{mu} (kPa)
BFS-1	12.06	1414.45	127.55	69	332.28	1082.16
	14.02	1086.56	178.57	69	464.95	621.61
	17.03	711.25	234.69	69	610.76	100.48
	20.07	474.56	306.12	69	796.23	Marginal
	24.8	289.66	341.83	69	879.48	Marginal
BFS-2	12.25	1292.45	137.75	65.80	306.52	985.92
	15.32	954.56	188.77	65.80	419.85	534.70
	20.12	572.14	229.59	65.80	510.39	61.75
	22.61	402.47	326.53	65.80	725.54	Marginal
	25.5	259.30	362.24	65.80	804.52	Marginal
BFS-3	13.11	1033.20	147.96	59.53	251.48	781.71
	17.12	700.58	204.08	59.53	346.73	353.84
	19.98	419.98	270.40	59.53	459.24	39.26
	23	266.44	352.04	59.53	597.64	Marginal
	25.97	163.99	392.85	59.53	666.67	Marginal
WIS-1	14.98	1253.44	107.14	57.83	170.33	1083.10
	17.5	976.02	234.69	57.83	372.97	603.04
	21	690.80	290.81	57.83	461.98	228.81
	24.03	438.6	372.45	57.83	591.44	Marginal
	28.06	269.20	418.37	57.83	664.09	Marginal
WIS-2	14.25	912.54	112.25	48.74	127.94	784.59
	18.32	654.95	250	48.74	284.87	370.08
	24.85	385.24	311.23	48.74	354.51	30.72
	26.21	278.47	397.96	48.74	453.14	Marginal
	29.10	175.10	448.98	48.74	511.06	Marginal
WIS-3	15	682.34	122.45	37.16	92.81	589.53
	20	476.04	270.41	37.16	204.87	271.16
	23	331.39	336.74	37.16	255.04	76.34
	26.05	213.73	433.67	37.16	328.34	Marginal
	29.97	133.44	489.79	37.16	370.69	Marginal

APPENDIX 44

Table 6.29: Modified-unsaturated shear strength for studied soils WES, BES

Soils	Water Content (%)	τ_u (kPa)	P_s (kPa)	$\tan(\phi^s)$ Degree	$\tau_u - \tau_{mu} = P_s \tan(\phi^s)$ (kPa)	τ_{mu} (kPa)
WES-1	15.93	830.31	137.75	35	96.45	733.84
	19.25	623.62	260.20	35	182.13	441.49
	23.37	413.94	397.95	35	278.44	135.49
	26.14	288.08	500	35	349.71	Marginal
	29.10	196.01	602.04	35	420.92	Marginal
WES-2	17.20	622.27	168.36	23.72	73.97	548.28
	19.98	468.25	331.63	23.72	145.64	322.60
	23.05	328.35	484.69	23.72	212.76	115.57
	26.52	226.64	602.04	23.72	264.15	Marginal
	30.43	144.01	724.48	23.72	317.72	Marginal
WES-3	18	498.24	188.77	17	57.71	440.52
	21.47	356.96	362.24	17	110.68	246.27
	24.56	244.25	551.02	17	168.25	76.00
	27.75	167.61	688.77	17	210,18	Marginal
	30	118.53	826.53	17	252.06	Marginal
BES-1	9.18	1512.02	143.882	68.33	362,111	1161,834
	12.42	1073.58	150.86	68.33	379,485	701,236
	14.54	678.82	174.09	68.33	437,694	243,065
	17.23	494.52	269.26	68.33	676,631	Marginal
	20.30	296.95	356.90	68.33	896,415	Marginal
BES-2	9.21	1313.69	144.31	63	283,228	1030,459
	13.35	877.95	151.86	63	297,915	580,030
	15.50	545.72	176.98	63	347,061	198,662
	18.13	359.17	279.93	63	548,701	Marginal
	21.05	244.61	374.74	63	734,211	Marginal
BES-3	10.21	1150.89	144.93	51.36	181,296	969,599
	15	715.28	153.31	51.36	191,707	523,575
	18.24	371.53	181.19	51.36	226,491	145,036
	19.93	211.50	295.42	51.36	369,152	Marginal
	21.60	143.37	400.62	51.36	500,422	Marginal

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Table 6.31: Models summary and coefficients

Models	R^2	R^2 (adj)	R^2 (pred)	Intercepts	Coefficients
Smectite (%)	99.99	99.89	99.83	$\xi_0 = 92.70$	$\xi_1 = 1.594$; $\xi_2 = 28.11$ $\xi_3 = 42.75$; $\xi_4 = 0.428$
Silica (%)	91.43	90.79	90.45	$\eta_0 = 109.5$	$\eta_1 = 1.131$; $\eta_2 = 47.80$ $\eta_3 = 1.860$; $\eta_4 = 0.657$
K-feldspar (%)	95.13	93.63	90.21	$\mu_0 = 310.7$	$\mu_1 = 1.008$; $\mu_2 = 34.91$ $\mu_3 = 127.9$; $\mu_4 = 0.652$
Plagioclase	95.18	93.69	92.63	$\alpha_0 = 7.4$	$\alpha_1 = 0.104$; $\alpha_2 = 0.488$ $\alpha_3 = 0.0217$; $\alpha_4 = 4.13$

Table 6.32: P-values

Smectite		Silica		K-feldspar		Plagioclase	
Term	P-value	Term	P-value	Term	P-value	Term	P-value
PI (%)	0.000	PI (%)	0.003	PI (%)	0.000	LL (%)	0.004
FSR	0.000	FSR	0.001	FSR	0.001	LS (%)	0.014
Gs	0.000	Gravel (%)	0.000	Gs	0.000	Gs	0.030
FSI (%)	0.000	FSI (%)	0.000	FSI (%)	0.000	FSI (%)	0.034

Table 6.33: Models summary and coefficients

Models	R^2	R^2 (adj)	R^2 (pred)	Intercepts	Coefficients
OMC	97.50	96.80	93.11	$\xi_0 = 8.707$	$\xi_1 = 0.025$, $\xi_2 = 0.597$ $\xi_3 = 0.214$, $\xi_4 = 0.119$ $\xi_5 = 11.116$
$\gamma_{\max}$	99.37	99.02	97.79	$\eta_0 = 14.160$	$\mu_1 = 0.120$, $\mu_2 = 0.210$ $\mu_3 = 0.171$, $\mu_4 = 0.165$ $\mu_5 = 3.345$

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Table 6.34: Predictive models

Reference	Equations
Shien <i>et al.</i> (2015)	$\text{OMC} = -0.2929G - 0.1174S - 0.1515F + 0.7378LL$ $\gamma_{dmax} = +0.0219G + 0.0235S + 0.0246F - 0.0185LL$
Firomsa and Quezon (2019)	$\text{OMC} = +9.743 + 0.226LL + 0.114PL$ $\gamma_{dmax} = +1.788 - 0.003LL - 0.004PL$
Hussain and Atalar (2020)	$\text{OMC} = +0.1809LL + 8.5179$ $\gamma_{dmax} = +26.811 - 0.5198W_{Op}$

Table 6.35: P-values

γ_{dmax} (kN.m ⁻³)		OMC (%)	
Terms	P-value	Terms	P-value
LL, %	0.000	LL, %	0.000
PI, %	0.045	PI, %	0.037
PL, %	0.063	PL, %	0.088
Fine, %	0.000	Fine, %	0.001
Sand, %	0.011	Sand, %	0.038
Gravel, %	0.032	Gravel, %	0.025
C _c	0.331	C _c	0.452
C _u	0.016	C _u	0.022
G _s	0.000	G _s	0.000

Table 6.36: Validation of the developed models

Optimum water content OMC (%)				Maximum dry unit weight γ_{dmax} (kN.m ⁻³)			
Measured OMC	Predicted OMC	Absolute Error (AE)	Percent Error (PE)	Measured γ_{dmax}	Predicted γ_{dmax}	Absolute Error (AE)	Percent Error (PE)
20.07	20.94	0.87	4.33	17.58	17.44	0.14	0.79
22.61	21.64	0.97	4.29	17.16	17.25	0.09	0.52
23.00	22.95	0.05	0.22	16.95	16.96	0.01	0.06
24.03	23.42	0.61	2.54	16.85	16.94	0.09	0.53
24.58	24.37	0.21	0.85	16.71	16.76	0.05	0.30
26.05	26.00	0.05	0.19	16.45	16.39	0.06	0.36
26.14	25.58	0.56	2.14	16.29	16.47	0.18	1.10
26.52	26.86	0.34	1.28	16.05	15.88	0.17	1.06
27.75	28.39	0.64	2.30	15.65	15.55	0.10	0.64
17.23	17.02	0.21	1.21	19.6	19.59	0.01	0.05
18.13	18.00	0.13	1.60	19.20	19.20	0.00	0.00
18.24	19.01	0.77	4.22	18.76	18.64	0.12	0.64

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Table 6.37: Experimental data by Shien *et al.* (2015)

Soils	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Fine (%)	Liquid Limit (%)	Plastic Limit (%)	Specific gravity (Gs)	OMC (%)	γ_{dmax} (kN.m ⁻³)
Soil 1	7	39	41	13	54	43	27	2.55	18	16.6
Soil 2	9	46	36	9	45	53	34	2.45	24	14.7
Soil 3	3	28	50	19	69	51	33	2.56	24	14.8
Soil 4	1	8	64	27	91	47	30	2.57	19.5	16.0
Soil 5	20	46	31	3	34	44	28	2.56	14	17.2
Soil 6	0	25	40	35	75	46	29	2.54	17	15.7
Soil 7	10	46	32	12	44	42	26	2.55	17	16.5
Soil 8	26	30	26	18	44	43	28	2.58	14.5	17.2
Soil 9	20	46	23	11	34	41	26	2.60	13.5	17.4

Table 6.38: Validation using data by Shien *et al.* (2015)

Soil Designation	Optimum water content OMC (%)			Maximum dry unit weight γ_{dmax} (kN.m ⁻³)		
	Measured	Predicted	Absolute Error (AE)	Measured	Predicted	Absolute Error (AE)
Soil 1	18	17.86	0.14	16.60	17.30	0.7
Soil 2	24	23.47	0.53	14.70	14.54	0.16
Soil 3	24	21.31	2.69	14.80	15.78	0.98
Soil 4	19.5	19.33	0.17	16.00	15.60	0.4
Soil 5	14	14.98	0.98	17.20	17.65	0.45
Soil 6	17	20.06	3.06	15.70	16.04	0.34
Soil 7	17	17.14	0.14	16.50	16.77	0.27
Soil 8	14.5	13.85	0.65	17.20	17.40	0.20
Soil 9	13.5	14.16	0.66	17.40	18.74	1.34

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Table 6.39: Correlation matrix A

Soils	LL (%)	Fine (%)	Specific gravity, G_s	Void ratio (e)
BFS-1	58.98	59.51	2.64	0.501
BFS-2	61.27	61.83	2.68	0.561
BFS-3	64.60	65.18	2.71	0.598
WIS-1	63.78	67.52	2.73	0.620
WIS-2	66.22	70.1	2.76	0.657
WIS-3	70.64	74.78	2.78	0.689
WES-1	69.45	73	2.73	0.675
WES-2	74.31	78.11	2.78	0.732
WES-3	78.94	82.98	2.83	0.768
BES-1	40.29	45.14	2.56	0.306
BES-2	43.59	48.32	2.60	0.350
BES-3	48.37	51.55	2.63	0.400

Table 6.40: P-values

Total suction ψ_t (kPa)		Matric suction ψ_m (kPa)	
Terms	P-value	Terms	P-value
Clay, %	0.038	Clay, %	0.032
Sand, %	0.066	Sand, %	0.061
γ_d , kN.m^{-3}	0.009	γ_d , kN.m^{-3}	0.007
L_s , %	0.085	L_s , %	0.079
LL, %	0.017	LL, %	0.015
PI, %	0.056	PI, %	0.052
Void ratio (e)	0.023	Void ratio (e)	0.030
Fine, %	0.019	Fine, %	0.037
W, %	0.011	W, %	0.009
G_s	0.041	G_s	0.037
A_c	0.065	A_c	0.055
FSR	0.067	FSR	0.059
FSI, %	0.032	FSI, %	0.028

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Table 6.41: Models summary and coefficients

Models	R ²	R ² (adj)	R ² (pred)	Intercepts	Coefficients
ψ_m	98.56	98.33	98.01	$\xi_0 = 134.1$	$\xi_1 = 1.1714, \xi_2 = 1.518$ $\xi_3 = 6.64, \xi_4 = 64.70$ $\xi_5 = 2.20, \xi_6 = 0.711$ Box-Cox transformation $\lambda = +0.381777$
ψ_t	98.87	98.65	98.40	$\eta_0 = 190.4$	$\eta_1 = 1.543, \eta_2 = 2.142$ $\eta_3 = 8.13, \eta_4 = 76.70$ $\eta_5 = 9.50, \eta_6 = 0.999$ Box-Cox transformation $\lambda = +0.409238$

Table 6.42: Models summary and coefficients

Models	R ²	R ² (adj)	R ² (pred)	Intercepts	Coefficients
Saturated friction angle, ϕ' (°)	92.78	91.97	90.64	$\xi_0 = 12.13$	$\xi_1 = 0.1397; \xi_2 = 11.24$ $\xi_3 = 0.2208; \xi_4 = 0.0888$ $\xi_5 = 0.0296; \xi_6 = 0.1810$ Box-Cox transformation $\lambda = 0.482967$
Matric suction angle, ϕ^b (°)	96.64	96.33	96.05	$\eta_0 = 5.68$	$\eta_1 = 0.00062; \eta_2 = 0.03$ $\eta_3 = 0.0939; \eta_4 = 0.0424$ $\eta_5 = 0.0081$ Box-Cox transformation $\lambda = 0.5$
Swelling stress angle, ϕ^s (°)	98.04	97.86	97.73	$\alpha_0 = 125$	$\alpha_1 = 0.264; \alpha_2 = 146.80$ $\alpha_3 = 5.468; \alpha_4 = 0.7716$ $\alpha_5 = 5.748$
Cohesion, c' (kPa)	94.14	93.18	92.21	$\zeta_0 = 9.22$	$\zeta_1 = 0.1239; \zeta_2 = 0.2616$ $\zeta_3 = 11.39; \zeta_4 = 0.0848$ $\zeta_5 = 0.2641$ Box-Cox transformation $\lambda = 0.5$

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Table 6.43: P-values

Saturated friction angle, ϕ' (°)		Matric suction angle, ϕ^b (°)		Swelling stress angle, ϕ^s (°)		Cohesion, c' (kPa)	
Term	P-value	Term	P-value	Term	P-value	Term	P-value
W (%)	0.000	Gs	0.002	PI (%)	0.031	LL (%)	0.031
γ_d , kN.m ⁻³	0.045	Void ratio (e)	0.054	W (%)	0.018	Clay (%)	0.018
FSI (%)	0.063	W (%)	0.000	FSR	0.056	Fine (%)	0.046
FSR	0.071	γ_d , kN.m ⁻³	0.038	Fine (%)	0.021	Void ratio (e)	0.051
Clay (%)	0.012	FSR	0.055	γ_d , kN.m ⁻³	0.060	C _u	0.060
Fine (%)	0.022	FSI (%)	0.063	C _u	0.063	FSI (%)	0.060
Gs	0.003	C _u	0.058	Gs	0.021	FSR	0.081
Void ratio (e)	0.058	PI (%)	0.025	Clay (%)	0.056	Gs	0.006
LL (%)	0.011	LL (%)	0.018	FSI (%)	0.004	γ_d , kN.m ⁻³	0.004
PI (%)	0.015	Clay (%)	0.038	Void ratio (e)	0.055	W (%)	0.031
C _u	0.061	Fine (%)	0.027	LL (%)	0.052	PI (%)	0.000

Table 6.44: Predictive models

Reference	Equation
Chen <i>et al.</i> (2012)	$\log \phi' = -(0.0186W + 0.28)\gamma_d + 0.0141W - 1.15$ $c' = -4.13W + 149.4$
Roy and Dass (2014)	$\phi' = -29.604 + 34.220\gamma_d$ $c' = -0.525 + 24.1G_s$



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