EVALUATION OF DIFFERENT SOIL SHEAR STRENGTH TESTING PROCEDURES TAKING INHERENT SOIL VARIABILITY INTO ACCOUNT

by

VIVIAN CHARLES DE VILLIERS

A dissertation submitted in fulfilment of the requirements for the degree

Master of Engineering in Civil Engineering

in the

Department of Civil Engineering

of the

Faculty of Engineering, Built Environment and Information Technology

of the

Central University of Technology, Free State, South Africa

Supervisor: Prof, E Theron

Co-supervisor: Dr, PR Stott

October 2025

DECLARATION

I, the undersigned, declare that the dissertation hereby submitted by me for the degree *Master of Engineering in Civil Engineering* at the Central University of Technology, Free State, is my own independent work and has not been submitted by me to another University and/or Faculty in order to obtain a degree. I further cede copyright of this dissertation in favour of the Central University of Technology, Free State.

Vivian Charles de Villiers

Signature: ……

Date : October 2025

Bloemfontein, South Africa

ABSTRACT

For several decades, geotechnical engineers relied on the Working Load Design approach, where the uncertainty and variability of soil were addressed using subjective global factors of safety. The Limit States Design concept was first introduced in the late 1990s and later adopted in South Africa for geotechnical engineering. This approach involves selecting characteristic values based on experience, cautious estimation, standard tables, and/or statistical methods. However, relying solely on past experience and standard tables may not be sufficient to ensure safety against failure. The most logical approach to estimating the value of a soil parameter seems to be the use of statistical methods. Ensuring a high degree of reliability and confidence in the selected characteristic value requires extensive testing. Most testing standards only require a small number of tests, typically three, to obtain an average or “representative” value. Due to cost and time constraints, it is often unfeasible to conduct a large number of tests on a particular soil. This reduces our understanding and knowledge of the inherent soil variability at hand. To address these challenges three shear strength testing procedures, the vane shear, (undisturbed) fall-cone, and pocket penetrometer, were evaluated to enable more accurate testing and decision-making for measuring inherently variable soils. This leads to more reliable data analysis and establishing a less conservative characteristic value for design inputs. Testing was conducted on cohesive soils across multiple sites in the Mangaung Metro Municipality. The vane shear test proved to be inadequate due to a low shear strength capability. The fall-cone yielded very low penetration readings and underestimated the shear strength compared to the pocket penetrometer. Using Hansbo’s equation to convert penetration data to shear strength values scatters the data and introduces higher variability with an increase coefficient of variation. In contrast, the pocket penetrometer produced more consistent results with lower variation in testing data. None of the test methods appeared to be an ideal fit for a lognormal distribution. To improve shear strength measurements on undisturbed soil samples, the study recommends using a larger cone factor and a modified fall-cone apparatus.

Keywords: soil variability, fall-cone, cone factor, vane shear, pocket penetrometer, characteristic value, COV, probability density functions, Limit States Design, Eurocode 7.

ACKNOWLEDGEMENTS

A number of special acknowledgements deserve specific mention:

(a) Above all, acknowledgement to my Heavenly Father, Jesus Christ, for giving me guidance, plans to prosper, hope and a future (Jer. 29:11).
(b) The Rectorate and relevant functionaries from the Central University of Technology, Free State, for the opportunity of completing this research;
(c) The various agencies for funding and in particular the Central University of Technology, Free State and the Royal Academy of Engineering, England;
(d) Professor Elizabeth Theron, my supervisor, for guidance and support given throughout the research project, my personal life and career;
(e) Doctor Phillip Stott, my co-supervisor, for guidance, patience and support given;
(f) Members of the Central University of Technology's Soil Mechanics Research Group (Dandre Buitendag, Dewald van Wyk, Johno Steenkamp, Sam Waters, Yannis Glynos and all the WIL students) for assistance in soil sampling, collection and testing;
(g) Kevin Coertzen and Cole Herbig from GeoCalibre, for assisting with the site investigations and organising of the machinery, and equipment;
(h) PDTS from the Central University of Technology, for manufacturing and supplying the researcher with additional soil sample rings;
(i) Dirk Bester, from Tswane University of Technology, for guidance, support and motivation given; and
(j) My family, friends, and colleagues, for their patience and understanding throughout this research project.

TABLE OF CONTENTS

LIST OF TABLES

Page

LIST OF FIGURES

Page

LIST OF ABBREVIATIONS

ASTM	American Society for Testing and Materials
BS	British Standard
CD	Consolidated Drained
CEN	European Committee for Standardisation
CPT	Cone Penetration Test
CU	Consolidated Undrained
CUT	Central University of Technology
FC	(undisturbed) Fall-cone
FoS	Factor of Safety
ISO	International Organisation for Standardisation
IQR	Interquartile Range
LSD	Limit State Design
MMM	Mangaung Metro Municipality
PDF	Probability Distribution Function
PDTS	Product Development Technology Station
PP	Pocket Penetrometer
SANS	South African National Standards
SMRG	Soil Mechanics Research Group
SPT	Standard Penetration Test
TLB	Tractor-loaded-backhoe
UU	Unconsolidated Undrained
VS	Vane Shear (field)
WIL	Work Integrated Learning
WLD	Working Load Design
WSD	Working Stress Design

CHAPTER 1: INTRODUCTION

1.1 Problem Statement

The current design approach (Working stress design) for geotechnical engineering in South Africa does not take variability into account. This approach only requires average/representative values for soil properties obtained from testing methods. Soil preparation before testing entails mixing and quartering of soil samples to obtain a representative value for soil properties. The shear strength of soil is an important property used in the design of geotechnical structures. It is therefore important to determine the soil shear strength accurately.

The Limit State Design concept is a simplified approach to reliability-based design. It involves applying partial factors to the characteristic values of actions and/or resistances to mitigate the risk of failure. Eurocode 7 accounts for variability by selecting characteristic values for actions and resistances that accept a 95% confidence in reliability. Despite this, some uncertainty in the real level of reliability with a 5% probability of failure remains.

Statistical methods can be used to address variability. Many statistical procedures, however, apply only to a normal distribution. Furthermore, to establish an appropriate degree of reliability theoretically requires more than 600 tests (Eaton et al., 2019; Stott, 2020a; Stott, 2020b). It is therefore unfeasible for many soil shear strength tests to establish the 5^{th} percentile of a soil property due to time and expenditure. The usual way to address this problem is to assume the form of the probability density function. The Eurocodes specify a lognormal distribution. If the actual distribution of soil is not lognormal, the statistical analysis may be worthless.

Three testing methods are proposed to be used in such a way that soil variability is accounted for. The testing procedures potentially make possible a wide dataset of soil strength measurements that will allow for reliable variability analysis. From these tests, a probability density function for soil shear strength can be assessed and ultimately lead to procedures that actually account for variability.

1.2 Research Aim and Objectives

The aim of this research project is to evaluate some shear strength testing procedures, which can provide a wide dataset of measurements by providing many test results in an acceptable time, in order to measure the inherent shear strength variability of soil samples accurately.

Objectives of the research project include:

(a) Identify the shortcomings in current testing procedures and design approaches, which provide shear strength values for design calculations relating to geotechnical structures.

(b) Construct and assess reliable probability density functions from data measurements obtained from site and laboratory investigation using testing methods that can obtain large datasets in a quick and economical way.

(c) Evaluate the distribution type of cohesive soils in and around the Mangaung Metropolitan Municipality area to determine if they confirm a lognormal curve as stipulated by Eurocode 7.

(d) Compare the shear strength probability density functions obtained from proposed testing devices to establish if there are similarities in the patterns or distribution type(s) between these tests.

1.3 Research Methodology

The methodology followed to complete the research study and compile the dissertation is depicted in Figure 1-1 below:

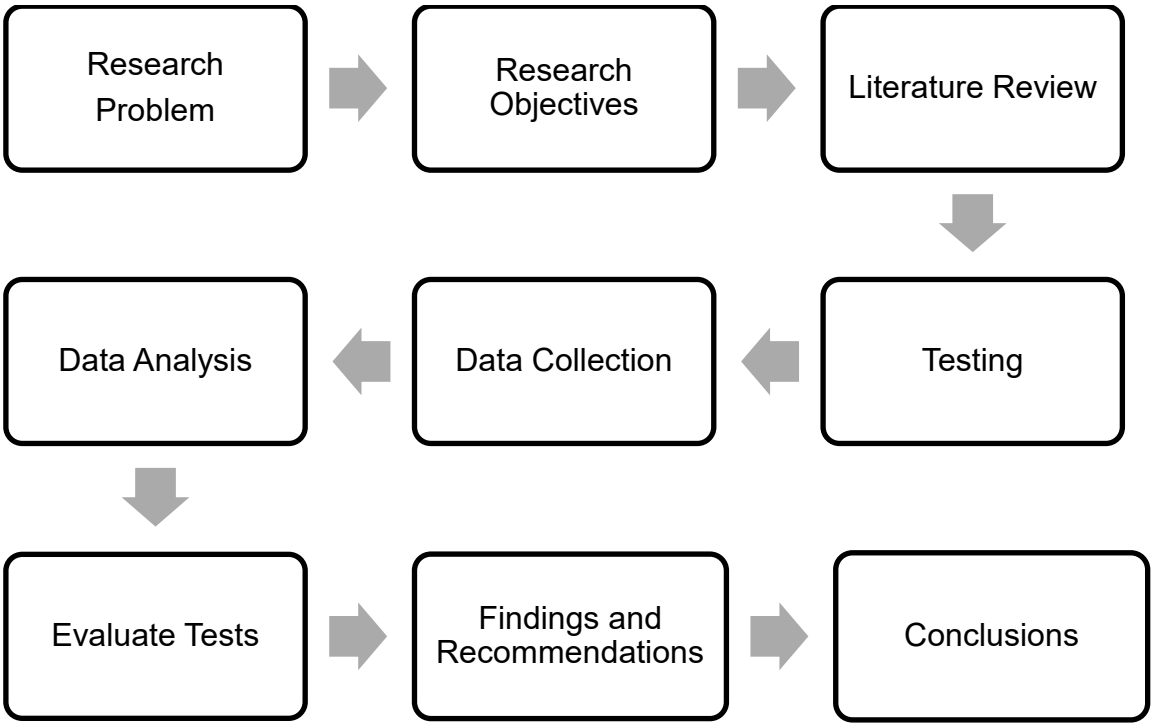


Figure 1-1: Research methodology

1.4 Contribution of the research study

Findings and results obtained from this study will indicate the importance of obtaining large datasets, which take into account the variability of soil, for more reliable shear strength values and analysis for design approaches. This will allow for more accurate design inputs and decision-making. Additionally, these tests can be used in a more economical and less time-dependent way than most shear strength tests currently in use.

1.5 Scope and limitations of the research study

This study investigates the variability and reliability of three undrained shear strength testing methods, vane shear, fall-cone, and pocket penetrometer, on undisturbed cohesive soils collected from selected sites within the Mangaung Metropolitan

Municipality. The aim is to assess each method's suitability for use in Limit States Design, with a focus on deriving less conservative and more statistically representative characteristic values. Emphasis is placed on evaluating the impact of Hansbo's equation in converting fall-cone penetration data and assessing the goodness-of-fit to a lognormal distribution. The study is limited by the number of test samples, as more than 600 tests are typically required to develop a statistically reliable probability density function (PDF), while this research was constrained by time and cost. Equipment limitations also affected results, with the vane shear proving inadequate for stiffer soils and the fall-cone underestimating shear strength. Moreover, findings are specific to local soil conditions and may not be directly applicable to other regions without further validation.

1.6 Dissertation Outline

The dissertation is categorised into six chapters as follows:

Chapter 1: This chapter provides the background and introduction of the study. It begins with the problem statement, followed by the research aims and objectives. Additionally, is resents an overview of the research methodology. The chapter concludes by highlighting the contributions of the research study.

Chapter 2: This chapter reviews the existing literature to this field of study. Previous studies are analysed, serving as a foundation and guideline for the research. Additionally, the various sections of the research are also individually examined and discussed in detail.

Chapter 3: This chapter emphasises the methodology of the research. The chapter consists of the study area, site geology, and research methodology used. The in-field sampling methods and testing procedures are discussed along with the sample preparation. Lastly, the testing apparatus is listed along with the Fall Cone (FC) testing procedure used to obtain shear strength measurements for data analysis.

Chapter 4: This chapter presents the data analysis and research findings Each section is individually discussed and described in detail. The results are visually

presented through tables, graphs, probability density functions, and screen captures.

Chapter 5: The chapter provides conclusions based on the results presented in the previous chapter. It discusses the findings in detail, highlighting their implications on the research outcomes. Additionally, recommendations for future research are proposed.

Chapter 6: This chapter contains a detailed reference list for each source that has been cited in this thesis to support and formulate this research topic.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

In this chapter, cohesive soils and soil variability are first introduced. A literature review on the two primary design approaches is followed by a discussion of the shortcomings of the most popular shear strength tests. Additionally, the proposed shear strength tests for this research project will also be reviewed for the evaluation and recommendation thereof.

2.2 Background

Soil can be broadly classified into two main groups, namely, cohesive and non-cohesive soils. Cohesive soils (clays and silts) can be defined as fine-grained, low-strength, and easy deformable soil with a tendency for particles to cling together when saturated. Non-cohesive or cohesionless soils (sand, gravel, and stone) exhibit granular characteristics in which the particles remain separate from one another and do not form clods or clumps of particles (Bobrowsky and Marker, 2018). Figure 2-1 below broadly illustrates the international classification of soils according to particle size in mm.

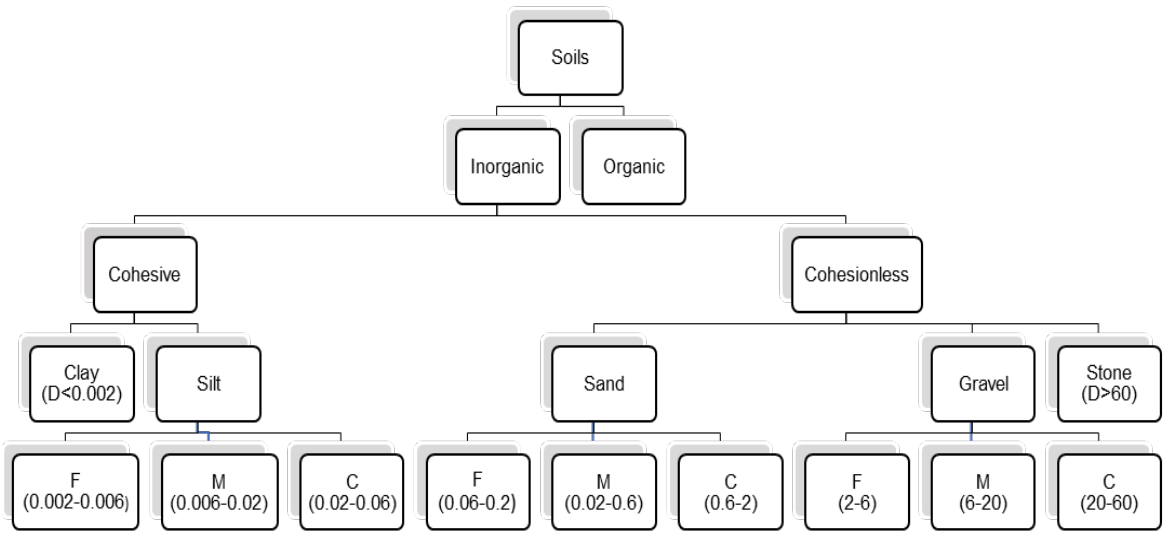


Figure 2-1: The classification of soil according to particle size in mm (Adapted from Jackson, 2020)

Mitchell and Soga (2005) classify soil as cohesive if the weight of fines (silt and clay-size material) exceeds 50% of the total soil weight. According to Das and Sobhan (2012) a soil is typically referred to as “clay” if it contains 50% or more particles with sizes of 0.002 mm or less. The South African Bureau of Standards (1995) classifies soils with a grain size of between 0.002 and 0.075 mm as silt and less than 0.002 mm as clay. The structure of clay in cohesive soils greatly influences the engineering behaviours and characteristics (Bobrowsky and Marker, 2018). Clay minerals are tiny crystalline substances that are primarily produced from chemical weathering and certain rock-forming minerals. The crystals can only be seen with an electron microscope and can be described as flaky in shape. These flakes comprise several crystal sheets that have a recurring atomic structure (Holtz and Kovacs, 1981). The sheets can either be tetrahedral (silica) or octahedral (alumina) sheets. Figure 2-2 illustrates an isometric view of a tetrahedral and octahedral sheet.

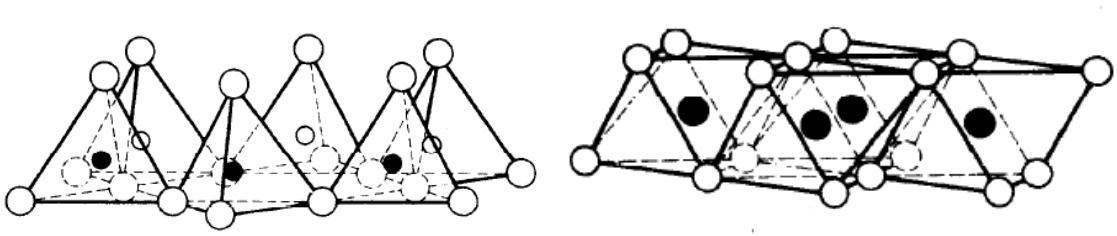


Figure 2-2: Isometric view of a tetrahedral sheet, to the left, and octahedral sheet to the right (Holtz and Kovacs, 1981)

The problem with cohesive soils is due to the fact that civil structures are frequently being built on clay-rich formations. Clay soils are widely distributed throughout the Free State province, as seen from the soil map in Figure 2-3. With low-cost and social housing rapidly on the rise, the need for more affordable and reliable soil testing procedures that will lead to safe and economical structures increases. The quality of soil samples is also critical for obtaining accurate data readings.

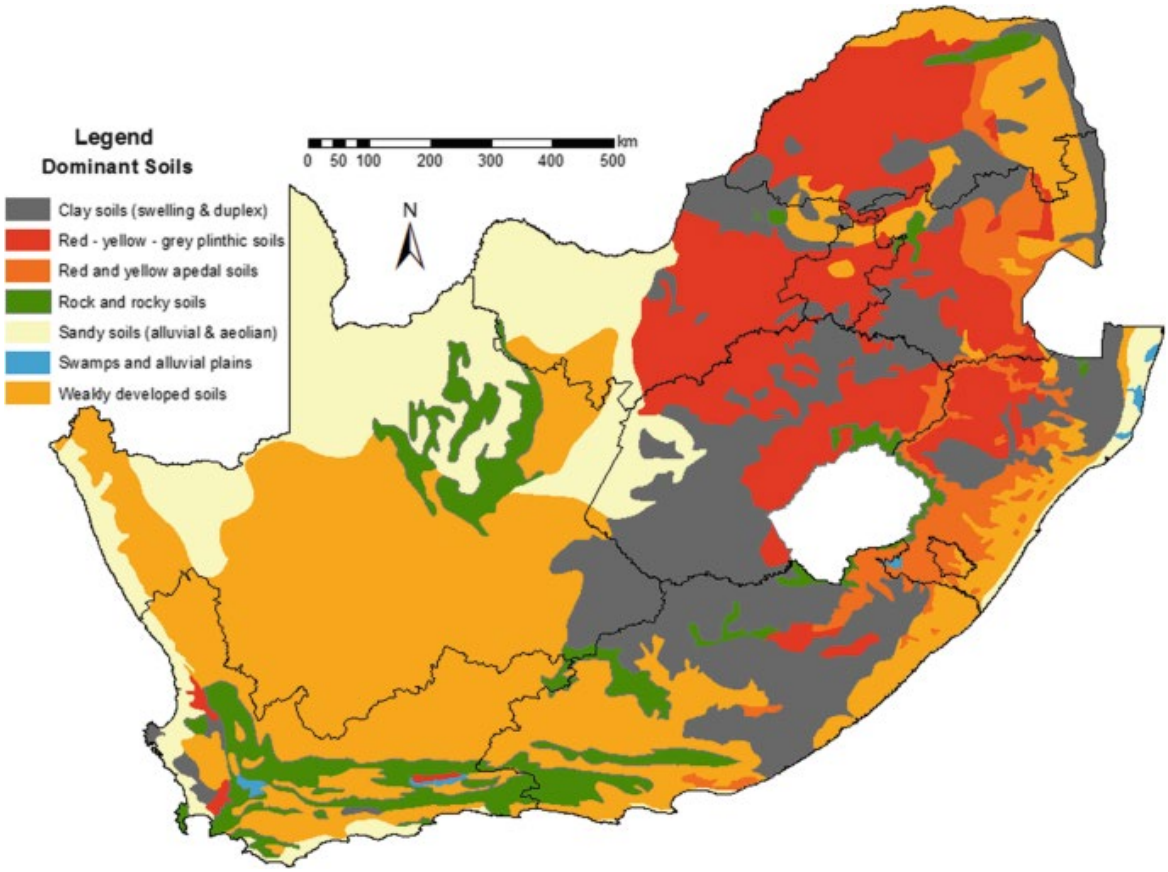


Figure 2-3: Regional distribution of dominant soils in South Africa (du Preez et al. 2019)

2.3 Soil variability

Geomaterial (soils and rocks) variability in geotechnical design is a complex attribute that results from a variety of different sources of uncertainty. According to Kulhawy et al. (1992) uncertainty is noted as an important factor in the vast majority of textbooks and courses but is then demoted to a lesser part while the remainder of the text/course is evaluated in a traditional deterministic way. The main sources of uncertainty in geotechnical engineering are inherent soil variability, measurement error, and transformation uncertainty (Kulhawy et al., 1992). Statistical uncertainty is a fourth uncertainty that may also be present when estimating material properties (Ehnbom and Kumlin, 2011; Schneider and Schneider, 2013). Inherent soil variability arises from natural geological processes, whereas measurement error results from the equipment and procedures used during soil testing. Transformation uncertainty occurs when measurements or readings are converted into design properties using empirical formulas and correlation models. Statistical uncertainty is

commonly included within measurement error (Ehnbom and Kumlin, 2011) and can be reduced by conducting a sufficient number of objective tests. For the purposes of this research project, inherent soil variability is mainly discussed and observed. Figure 2-4 below illustrates the various sources of uncertainties for soil properties.

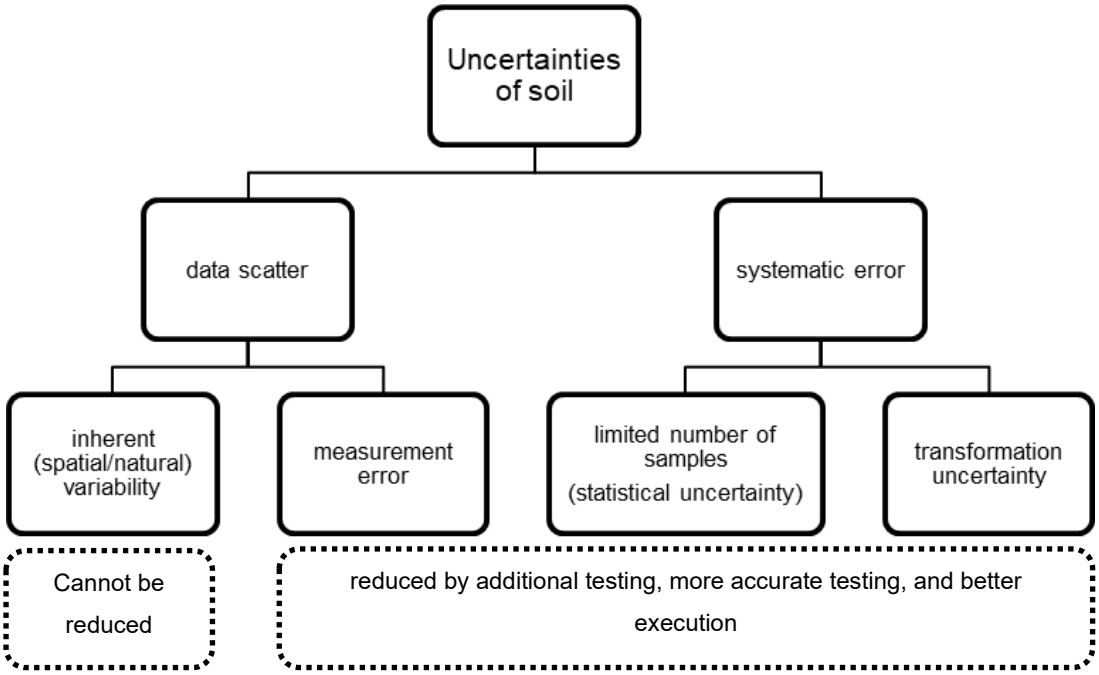


Figure 2-4: Various sources of uncertainties of soil properties
(Adapted from Schneider and Schneider, 2013)

Inherent soil variability refers to the variation of soil properties from one spatial location to another within the same soil mass (Uzielli et al., 2006). Natural deposits of soil generally display a range of variability when it comes to their geotechnical properties. Soil properties exhibit considerable variability even when soils are considered reasonably homogeneous (Vanmarcke, 1977). This is primarily a result of geological and environmental processes. Deposition and diagenesis contribute to the heterogeneity, anisotropy, and variability of soil properties (Galeandro, Doglioni and Simeone, 2017). According to Phoon and Kulhawy (1996) inherent soil variability results primarily from the natural geologic process that formed and continuously modifies soil. The variability of the materials engineers deal with in geotechnical engineering is significantly higher than that of other materials in civil engineering branches. Little attention has been given to the concept of variability by numerous working engineers and in geotechnical engineering courses at a tertiary

level (Stott and Theron, 2016). Because many geomaterials are highly variable in nature, determining their mechanical and chemical properties is also fundamentally difficult (Bond and Harris, 2008). Table 2-1 below, from Bond and Harris (2008) shows a significantly higher degree of variability for various geotechnical materials compared to manufactured materials like concrete, steel, and aluminium. Ground properties are not fixed values. Soil is intrinsically variable, and determining the properties of soil can be very difficult.

Table 2-1: Coefficient of variation of geotechnical and manufactured materials (Bond and Harris, 2008)

Material	Parameter	COV
Soil	Coefficient of shearing resistance	5-15%
	Effective cohesion	30-50%
	Undrained strength	20-40%
	Coefficient of compression	20-70%
	Weight density	1-10%
Concrete	Resistance of beams and columns	8-21%
Steel		11-15%
Aluminium		8-14%

There are various scales of soil variability that exist, from the geological scale of hundreds of meters or kilometres to the micro level at grain size scale. The multiple spatial scales contribute to the soil variability of samples, and it is therefore important to evaluate the effects of soil variability. Figure 2-5 from Maximilian Huber (2013) illustrates the multi-scale nature of soil. Inherent soil variability is an uncertainty that will always exist, meaning that under idealised testing conditions, a parameter's variation arising from testing various specimens is solely attributable to inherent variability (Lo and Li, 2007). In order to gain a good insight into the variability of a soil property, testing must be accurate and extensive.

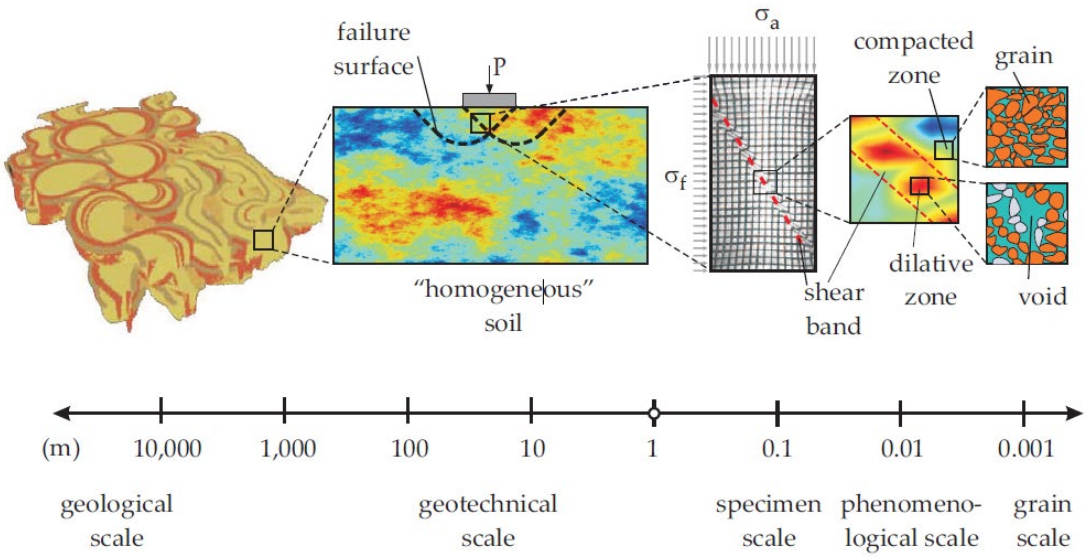


Figure 2-5: Multi-scale nature of soil (Huber, 2013)

Harr (1987) emphasised that statistical measures, such as the mean, variation, and coefficient of variation (COV), are effective tools for quantifying the substantial variability in soil properties. Variability in any soil parameter reflects the dispersion of data points relative to each other and to the central value of a distribution. This variability is often measured using the COV, defined as the ratio of the standard deviation of the sample(s) to the mean (μ), as expressed by the following equation:

$$COV = \frac{\text{standard deviation } (s)}{\text{mean } (\mu)} \times 100 \quad (1)$$

In general, clay soils have a higher COV than silty and sandy soils, indicating greater variability in their properties (Chew et al., 2015). The degree of variability decreases as the COV approaches zero. Harr (1987) further categorised the degree of variability into three groups based on the COV, as shown in Table 2-2 below.

Table 2-2: Degree of variability (Harr, 1987)

Degree of variability	COV%
Low	<15%
Moderate	15%<COV<30%
High	COV>30%

2.3.1 Factors influencing variability in soil property estimation

Several important factors contribute to the variability observed in the estimation of soil properties, particularly undrained shear strength. These factors include the inherent characteristics of the soil type, the specifics of the measurement process, and the choice of transformation model used to interpret test results, as mentioned in the previous section. This section examines each of these factors to understand and quantify their influence on the reliability and accuracy of soil property estimates.

The inherent characteristics of the soil type play a crucial role in the variability of estimated soil properties. Different soil types, such as clays, silts, sands, and gravels, exhibit distinct physical and mechanical behaviours influenced by particle size, mineralogy, moisture content, and compaction state. For example, expansive clays are known for their high variability in shear strength and compressibility due to moisture fluctuations, whereas granular soils generally demonstrate more consistent behaviour (Das, 2010). The heterogeneity within the same soil type, caused by layering or localised changes in composition, further contributes to variability, making accurate characterisation a challenge. Understanding the soil type is essential for interpreting test results and predicting soil behaviour reliably (Craig, 2004).

The process used to measure soil properties significantly affects the variability in results. Testing methods, whether in-situ or laboratory-based, come with inherent limitations such as instrument precision, operator skill, sample disturbance, and environmental conditions during testing (Terzaghi et al., 1996). For instance, cone penetration tests (CPT) provide continuous profiles but may be influenced by soil layering and penetration rate (Robertson, 2016), while laboratory shear tests can be affected by sample extraction and preparation. Variations in calibration, data recording, and interpretation procedures also introduce discrepancies. Consequently, the measurement process introduces a layer of uncertainty that must be quantified and minimised to improve the reliability of soil property estimation.

Transformation models, which convert raw test data into meaningful soil property estimates, add another dimension of variability. These models rely on assumptions

and empirical relationships that may not fully capture the complex behaviour of soils under different conditions (Lunne, et al., 1997). For example, converting cone resistance readings into shear strength values often uses factors derived from limited datasets, which may not be universally applicable (Robertson and Campanella, 1983). The choice of model parameters, such as correction factors or soil behaviour indices, impacts the accuracy and generalisability of results. Critically evaluating these models for their appropriateness to specific soil conditions and validating them against independent datasets is vital to reduce estimation uncertainty and ensure robust geotechnical design (Phoon and Kulhawy, 1999).

The variability in estimated soil properties is significantly influenced by the inherent characteristics of the soil type, the precision and limitations of the measurement process, and the assumptions embedded within transformation models, all of which must be carefully considered to ensure accurate and reliable geotechnical assessments.

2.4 Shear strength

The shear strength of soil is a very important property of soil. Engineers use shear strength values to evaluate foundation bearing capacity, determine the stability of retaining walls, slopes, dams, and embankments, as well as to design and build highway and airfield pavements (Adeyeri, 2015; Das, 2008). Soil strength is defined as the resistance to mass deformation and is the maximum magnitude of shear stress that a material can withstand (Bowles, 1996). A soil's shear strength is as a result of inter-particle friction, with a component of cohesive strength (Bobrowsky and Marker, 2018). Gravitational forces influence frictional shear strength, which is common in coarse-grained soils, whereas cohesion shear strength is a function of surficial forces common in fine-grained soils (Robertson and Chock, 2017). According to Dolinar (2004), the primary factors that determine soil strength are the soil moisture, clay size, and quantity of clay particles. Figure 2-6 from Bond and Harris (2008) illustrates the wide range of strengths for soil that can be encountered in nature compared to manufactured materials.

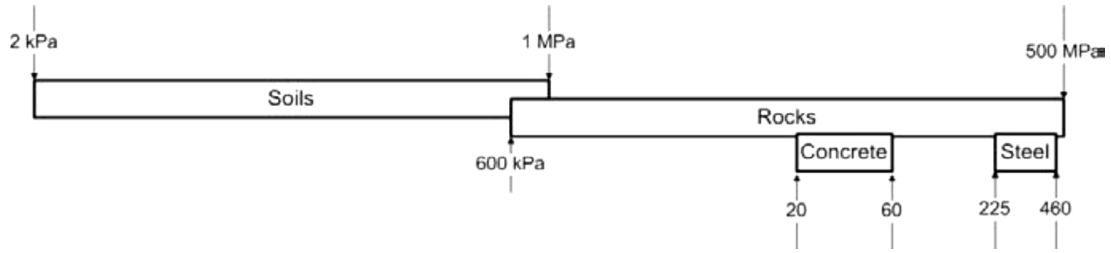


Figure 2-6: Range of soil and rock strengths encountered in nature, compared with strengths of manufactured materials (Bond and Harris, 2008)

As stated by Day (2012), the shear strength of cohesive soils is divided into three categories: undrained, drained, and drained residual shear strength. The undrained shear strength refers to a condition where the cohesive soil does not allow water to enter or exit during shearing. For a drained shear strength test, pore pressure is allowed to completely dissipate during the shearing condition. Drained residual shear strength is the shear strength of soil in a drained condition after it has undergone extensive shear deformation. The shear strength is an important parameter of soil that needs to be measured accurately in order to obtain reliable input values for design approaches.

The undrained shear strength of a soil is frequently determined and used for design analysis and may be obtained by laboratory and field tests. According to Wahls (1983), a common requirement of these tests is that failure stresses must occur without any drainage or change in volume. Common laboratory tests to determine the undrained shear strength of soils include the unconfined compression test, the triaxial compression test [unconsolidated undrained (UU) and consolidated-undrained (CU)], and the direct shear test. The most common in situ tests are the vane shear tests, penetration tests, and pressuremeter tests. Undrained shear strength testing must be conducted on relatively undisturbed samples (Wahls, 1983).

2.5 Shear strength tests

Various tests and testing methods have been developed and used over the years to determine the shear strength parameters of soil samples. The shear strength can be determined by laboratory testing on undisturbed or remoulded samples obtained from the field or by conducting in-situ strength tests. Considerable attention and

careful judgement are necessary in the sampling, handling, transporting, and storage of soil samples before laboratory testing, especially for undisturbed samples, where the aim is to maintain the in-situ structure and moisture content of the soil (Craig, 2004). Design approaches rely on these testing methods and testing standards to obtain characteristic and design values. Figure 2-7 illustrates the various shear strength tests available. For the purpose of this research project, only the most commonly used shear strength testing procedures will be discussed.

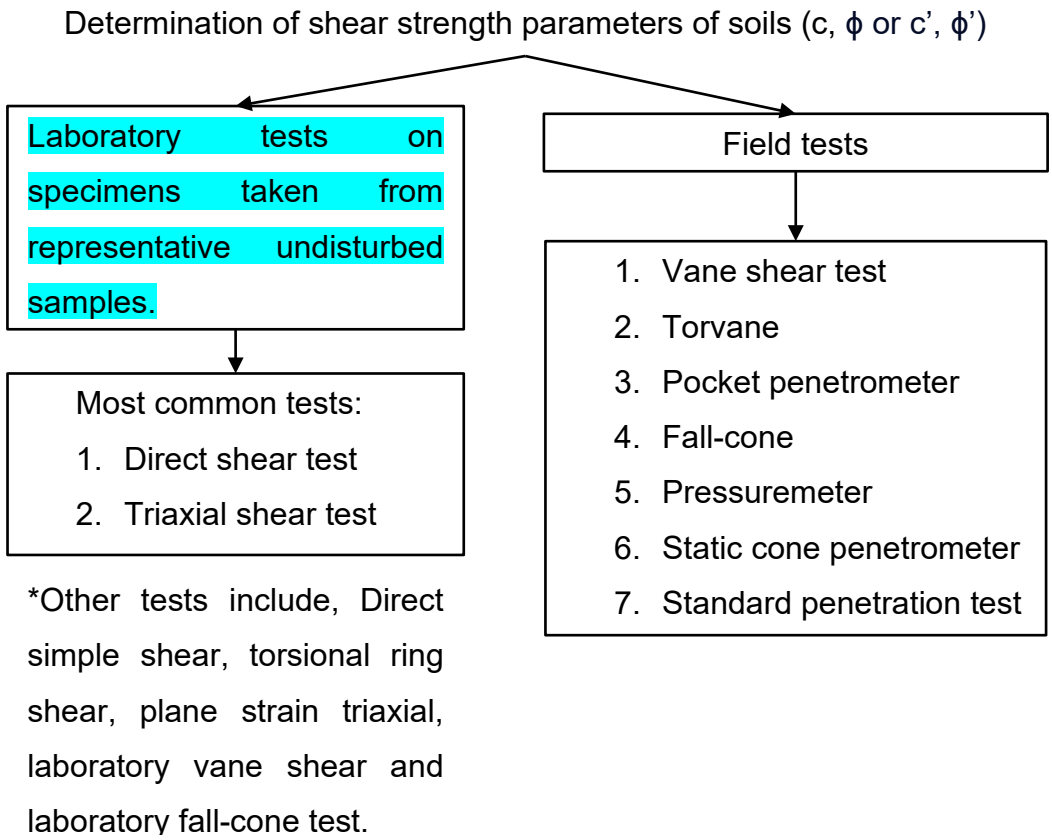


Figure 2-7: Determination of shear strength parameters of soil by most common testing methods (Adeyeri, 2015)

2.5.1 Direct shear test

The direct shear test is arguable one of the most common methods used for determining the strength of a soil due to its relative operational simplicity (Bro, Stewart and Pradel, 2013) which can be performed on both undisturbed and remoulded samples. Practicing geotechnical engineers mainly recommended the

direct shear test to determine the cohesion and angle of internal friction of granular, cohesionless soils. The test can also be used on cohesive soils. The drained direct shear test is a commonly used method for determining the shear strength of saturated sand and clay. Figure 2-8 illustrates a schematic representation of the direct shear test equipment.

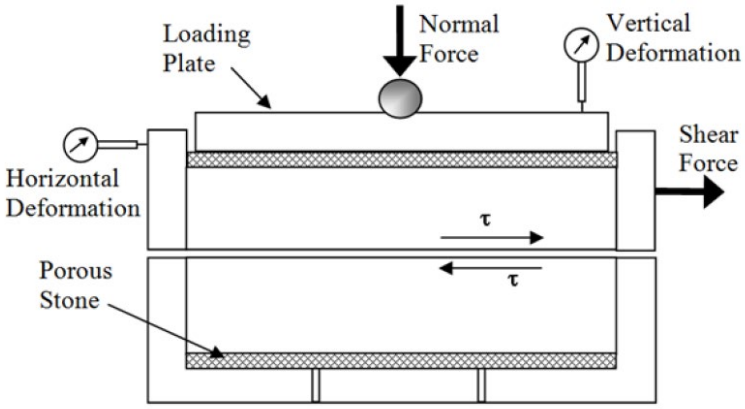


Figure 2-8: Direct shear test arrangement (IIT Gandhinagar, 2008)

The accuracy of the direct shear test is directly impacted by proper sample preparation (Backus, 2020). It is best to prepare the specimens in a humid environment to prevent moisture loss using an initial sample that is big enough to yield at least three identical test specimens. Most frequently, errors occur when trimming or moving the test sample from the cutter to the shear box (Backus, 2020). Once a sample has been prepared, the sample can go into the shear box. Soil samples are contained in a metal box (shear box). The box consists of a square or circular cross-section split horizontally with a small clearance between the two halves. Porous plates are placed underneath and on top of fully or partially saturated samples to allow for free drainage, while solid plates may be used for dry samples. A normal stress load is selected for the initial consolidation of the specimen to represent field conditions. Intervals for load applications may be up to 24 hours. Deformations are measured periodically to determine the consolidation. The shear rate during horizontal loading is determined by deformation and time data from this phase. The shear rate must be slow enough to permit any significant pore pressure to dissipate. According to Backus (2020), *ASTM and AASHTO* test standards do not specify the exact values used to perform the test. These parameters can be determined by carefully calculated equations noted in the test method or typically

determined by stakeholders (geotechnical/field engineer) requesting the tests. Once the shear rates have been calculated and set, the lateral movement device is turned on to start the test. In order to cause failure in the soil sample, shear force is applied by moving one half of the box relative to the other. Readings are recorded of the time, loads, and vertical and lateral displacement. Backus (2020), further notes that a space between the upper and lower halves of the shear box needs to be maintained with caution throughout the process. It might be necessary to stop the test in order to reset the gap opening to prevent interference. The soil specimen is sheared to a point specific by specifying authority or at least to 10% relative displacement. Finally, the shear box is removed after the shear test concludes. Several soil samples are tested under different vertical forces in order to obtain shear stress values at failure, which are plotted against the normal stress for each test. The best line fitting the plotted points is then used to calculate the shear strength parameters (Craig, 2004).

The drained direct shear test is a widely used method for determining the shear strength of saturated sand and clay. In this test, a container capable of holding water to saturate the sample is placed within the shear box that contains the soil sample. The test is conducted at a slow loading rate to ensure that any excess pore water pressure generated in the soil is fully dissipated through drainage via porous plates. Since clay has a much lower hydraulic conductivity than sand, it requires a longer period for complete consolidation after a normal load is applied to the sample. As a result, the shearing load must be reduced (Das, 2008) to allow for porewater pressure to dissipate. A single test on a clay/cohesive sample can last anywhere from three days onwards, with at least three tests required at different normal stresses to determine the shear strength parameters of a soil sample. Even though the direct shear test is commonly recommended, there are several limitations associated with the test method that should be considered when interpreting the results:

i. Sample size - The test requires a relatively large soil sample, which can be difficult to obtain for certain types of soils or in certain field conditions. This limitation is particularly relevant for cohesive soils, where small samples are more likely to exhibit anisotropy.

ii. Anisotropy - The test provides information on the shear strength of a soil in one plane, whereas in the field soil is subjected to stresses in multiple planes (Leung, 2003). Since the soil sample is not allowed to fail along the weakest plane, the reliability of the test results may also be questioned (Das, 2008).

iii. Stress state - A soil's shear strength can only be determined by a single point in a direct shear test; the soil's stress-strain properties cannot be determined over a wide range of shear strains. The two-dimensional stress state that the direct shear test assumes might not adequately reflect the actual stress state of the soil in the field (Leung, 2003).

iv. Drainage – Drainage conditions cannot be controlled (Wahls, 1983). The test assumes the pore pressure to be zero, which may not be true in the field (Craig, 2004). Bro et al. (2013) confirm that the shear displacement rate has a significant impact on drainage conditions within clay specimens. As a result, practicing engineers may unintentionally measure partially drained responses if they conduct direct shear tests at rates that are too rapid. The drained shear strength would be underestimated for consolidated or light-consolidated materials, whereas the drained soil strength would be over-predicted for severely over-consolidated materials exhibiting dilatant behaviours during shear.

Standards on the direct shear include ASTM D3080, AASHTO T236, BS 1377:7, and EN 17892-10. A South African testing standard (SANS standard) on the test does not exist. It is also important to mention that the ASTM D3080 standard was withdrawn in 2020, pending a revision. A direct shear apparatus with all the necessary equipment and accessories can cost anything from R300 000.00 upwards, with varying models available on the market. A single test done at a commercial laboratory roughly cost R5000. The direct shear test is an expensive and relatively complicated test to perform on cohesive soils and is mainly used on dry or saturated sandy soils.

2.5.2 Triaxial test

The triaxial test is the most widely used shear strength test utilised for all kinds of soils. The ability to regulate sample drainage and quantify pore pressure makes the

test preferable to other test methods used to estimate shear strength (direct shear). Parameters such as cohesion (c'), internal angle of friction (φ'), and shear strength can be determined. Other parameters, like stiffness and permeability, can also be determined with additional equipment and appropriate skills.

One of the first triaxial apparatuses was developed in 1912 by a Hungarian engineer. The test was later overhauled by Arthur Casagrande in the 1930s. He recognised the need for an improved shear testing technique that took the natural confining forces surrounding soil materials in place, into account. At present, specialised sample preparation equipment is used to shape and trim soil samples. Samples can either be undisturbed and extruded from tubes (generally Shelby tubes) or remoulded or compacted. To control fluid movement, a latex membrane, porous stones, a cap, and a pedestal for mounting in the test cell are all attached using a test cell kit along with additional equipment or accessories. The assembled test cell is mounted in the load frame along with the prepared soil specimen and water. Water and air pipes are connected to the triaxial control panel. The load and deformation measurement instruments are mounted and zeroed. Triaxial tests have three phases, and each phase differs depending on the chosen testing method. Thorough reading and comprehension of the test method being used is necessary beforehand for successful testing (Backus, 2021).

The following three conventional triaxial test types are generally used on soil samples: Unconsolidated Undrained (UU), Consolidated Undrained (CU), and Consolidated Drained (CD). The UU test is the simplest form of triaxial testing. The UU test is used on cohesive samples where samples may be intact, compacted, or remoulded. No drainage occurs during the consolidation or shear phases; additionally, samples may not be completely saturated. Pore pressures are generally not measured. The sample theoretically does not consolidate, and loads are applied relatively fast. The test is mainly intended for soil samples with very low permeability rates. The test is relatively fast and generally overpredicts shear strength (Ladd and DeGroot, 2003). In the CU test, drainage is allowed during the consolidation phase, leading to volume change, but no drainage is permitted during axial loading. Total stress and axial compression are determined by measuring axial load, deformation, and pore-water pressure. Soil samples are fully saturated during

the test. A strength envelope can be defined using data from testing three samples under different consolidation stress phases. The CD test is applicable to intact or recompacted soil samples. Since drainage is allowed during both the consolidation and shear phases, there are no excessive pore pressures at the point of shear failure. Measurements of the axial load and deformation, as well as volumetric changes, are used in the calculations for the primary stresses and axial compression. The test method is suitable mainly for free-draining soils, where testing on cohesive samples may take up to two weeks until completion.

The first phase of testing is saturation, followed by consolidation and shearing of the soil sample. Saturation is the process of water filling the voids in a soil sample without causing the sample undue disruption in the process. Depending on the type of test, soil samples are saturated with de-aired water produced in a deaeration device. Most tests use de-aired water in conjunction with backpressure saturation, additional pressure applied to the pore-water of the sample (Backus, 2021). Saturation of the sample is increased by the backpressure which compresses air in the sample and forces it into solution. De-aired water is a vital requisite where pore pressure needs to be measured. Dissolved air in the water could potentially cause pore pressure readings to be inaccurate. Consolidation takes place when the fluid in the test cell is subjected to confining pressure. For the UU test, where drainage is prohibited, restricting forces (i.e., excess pore water pressure) prevent a completely saturated material from consolidating. Samples that are partially saturated can consolidate and may test differently depending on the confining pressure (Backus, 2021). CU and CD specimens that were drained during consolidation will see a change in sample volume. Shearing is the final phase of testing. The rate of loading is generally set as 0.3% to 1% per minute for the UU test, with failure occurring in roughly 15 minutes. Strain rates for the CU and CD tests are calculated based on values from each consolidation phase. The strain rate or rate of loading for the two tests is much slower than the UU test. For these two approaches, the overall testing periods may be several days or even weeks. The triaxial test is widely used to determine the mechanical properties of soils; however, there are some limitations to the test:

i. Sample size – testing is typically conducted on small sample sizes, about 36 mm in diameter, which may not accurately represent the behaviour of larger soil masses (McCarthy, 2006).
ii. Disturbance – In order to perform the triaxial test, a cylindrical sample must be taken out of the soil, which may disturb the soil's natural state and affect the test results (Powrie, 2014). According to Ladd and DeGroot (2004), UU test results are unreliable and can exhibit considerable scattering due to the effects of sample disturbance.
iii. Stress state – a single loading path and relatively high loading rate are generally used, which might not accurately reflect the complex loading paths and loading conditions that occur in the field respectively (Murthy, 2002).

Each test's calculations and data reporting need to adhere to guidelines established by the specific test methodologies. For the UU test, the *ASTM D2850* or *AASHTO T296* is used as a testing standard and guideline, while the *ASTM D4767* and *D7181* are used for CU and CD testing, respectively. Based on the chosen technique and different sample types, each standard has a number of variables. Triaxial testing is not a one-size-fits-all approach and can be a very complex and time-consuming procedure. For successful testing and reliable findings, a thorough understanding of the chosen test methodology is required. Figure 2-9 below illustrates the different types of triaxial testing under different drainage conditions.

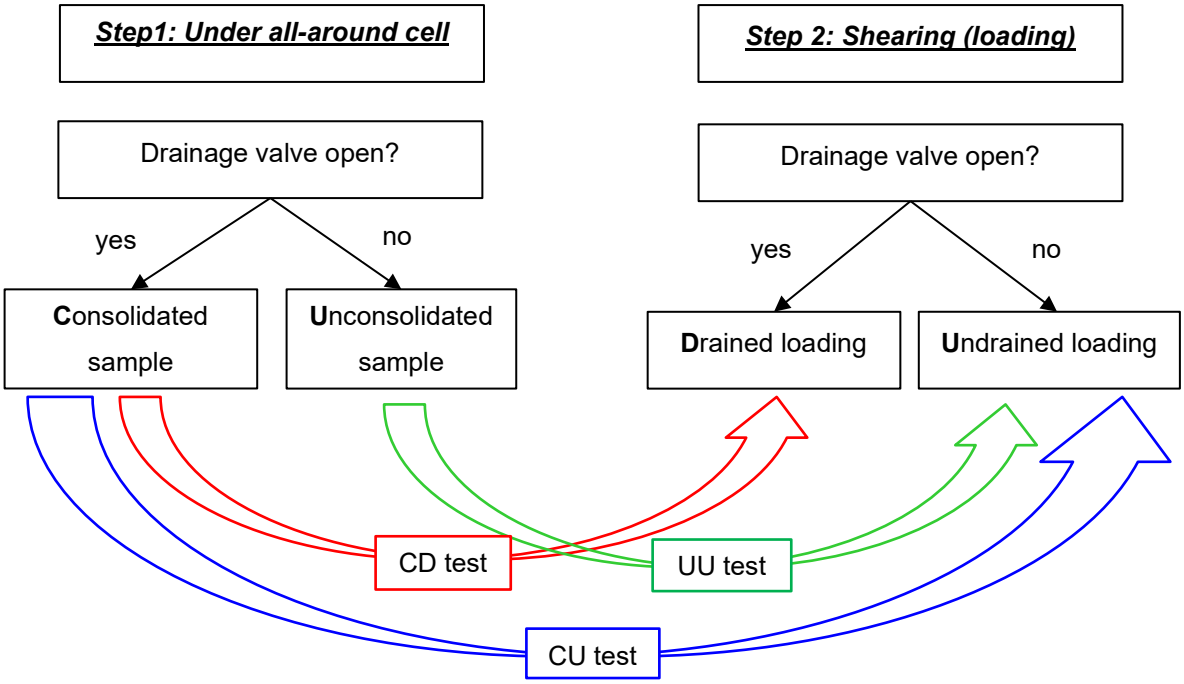


Figure 2-9: Types of triaxial tests (Adapted from Bappi, 2018)

Triaxial testing is currently not broadly used in South Africa for geotechnical engineering (Waters, 2022) with no associated testing standard for the country. A triaxial test protocol is being standardised as part of the revision of the South African Mechanistic Design Method (SAMDM) and is expected to become a testing standard for granular and stabilized materials (Mgangira, et al, 2011). The cost of a complete triaxial device with all the necessary accessories (cutter, porous discs, filter paper discs, dolly, software, etc.) costs approximately R350 000,00. A single-cost triaxial test ranges from R5000 to R10000 upwards (excluding transportation) depending on the type of triaxial test required and soil type.

2.5.3 Direct simple shear test

The first direct simple shear test capable of uniformly deforming a soil sample in pure shear was developed in 1936 by the Swedish Geotechnical Institute. Since then, many additional direct simple shear devices have been created Kjellman (1951) developed an apparatus for testing on circular soil samples that was enclosed in a rubber membrane surrounded by aluminium rings. The apparatus was later improved by the Norwegian Geotechnical Institute in the mid-1960s. Later, a

pressurised cell was developed to ensure complete saturation of the sample and provide accurate pore pressure measurements (Franke, Kiekbusch and Schuppener, 1979).

The testing apparatus was mainly developed to overcome the shortcomings of the direct shear test. Different shearing conditions apply to soil samples in the two tests. In the direct shear test, shearing occurs along a predetermined axis, which may not correspond to the weakest plane of the soil. In contrast, the direct simple shear test causes the entire sample to deform, without the formation of a single shearing surface (Olson, 1989). Shearing stresses are not uniformly distributed across a soil sample, however; it is still an improvement over the direct shear test (Das, 2008). Figure 2-10 shows the difference between the direct shear and direct simple shear tests on the following page.

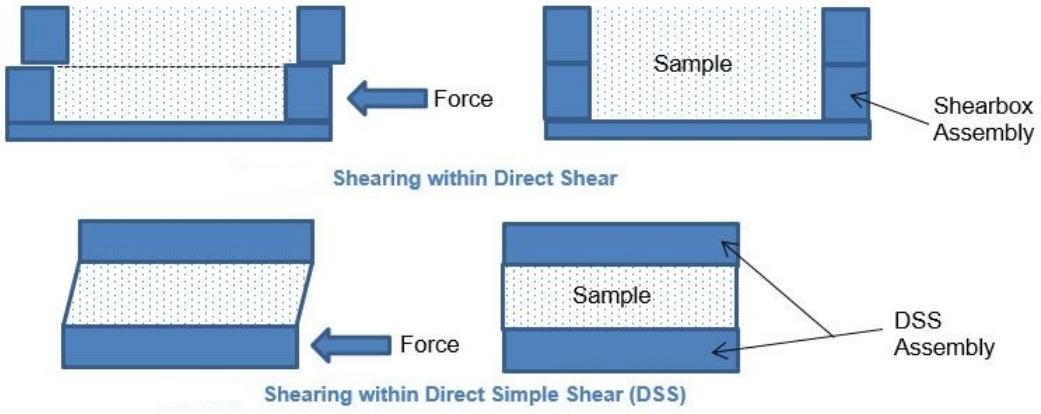


Figure 2-10: Shearing within the direct shear and simple shear tests (VJ Tech, 2021)

The basic concept of the direct simple shear test is to subject a cylindrical soil sample to direct shear. Vertical stress is applied to the sample from the top plate after it is placed between the base pedestal and the top plate. The sample is then sheared by moving the rings from side to side. The cross-sectional area remains constant during the test. While the test apparatus maintains the same initial stress state as a direct shear test, it avoids the stress concentration issues typically associated with the direct shear test (Bareither, Benson, and Edil, 2008). The test

can be conducted on either undisturbed or remoulded soil samples. Some notable shortcomings include:

I. Vertical stress variation: the test confines soil samples laterally throughout testing. The vertical stress changes that occur under normal field conditions are not replicated when test results are applied to actual soil layers with varying overburden pressures. This oversimplification can lead to inconsistencies (Ladd, et al., 1977).
II. Plane strain: Natural soil deposits often experience a triaxial stress state due to vertical loads. The test imposes a plane strain condition on the soil sample, which might not fully represent the complex stress and stain relationships in natural soil deposits (Das, 2008). As peak conditions approach, the stress and strain distribution in the direct simple shear test becomes extremely non-uniform (Babalola, 2016).
III. Stress distribution: Stress concentration can occur near the edges of soil samples, which can lead to nonuniform stress distributions that influence the measured shear strength properties (Mitchell and Soga, 2005).
IV. Shearing rate: It is possible that the rate of shear deformation in the laboratory test and the rate of deformation in the field is different. The shearing rate can impact how the soil reacts, particularly in situations involving time-dependent behaviour or rapid loading conditions.

The ASTM D6528 standard is used to determine the undrained shear strength of fine grain soils. The direct simple shear test is not popular in geotechnical practice. Past studies have given limited attention to the direct simple shear test as many geotechnical engineers prefer the direct shear test due to simplicity (Babalola, 2016).

2.5.4 Laboratory fall-cone test

The fall-cone is a laboratory test apparatus that was first introduced in 1915 by John Olsson at the Geotechnical Commission of the Swedish State Railways to determine the consistency and strength of clays. Consistency was expressed in terms of a strength number, which represents the weight of a specific cone required to achieve

a 10 mm penetration into a clay sample. The ratio of the strength number of an undisturbed clay sample to that of the remoulded clay was used to define the sensitivity of the clay. These strength values were later calibrated to correspond to the undrained shear strength of the clay, as measured using the shear box apparatus (Garneau and LeBihan, 1977). The strength number was defined as a function of the penetration depth of a standardized cone, weighing 80 g with a 30° apex. Figure 2-11 below indicates a typical fall-cone assembly (cone and shaft).

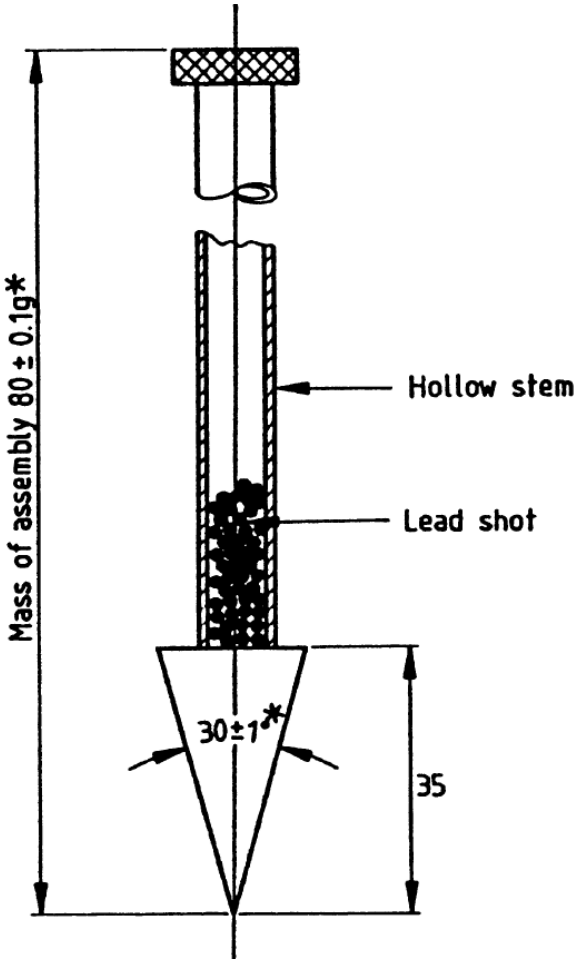


Figure 2-11: Cone assembly (BS 1377-2, 1990)

At present, the fall-cone is widely used as an alternative to the Casagrande cup in measuring the Atterberg limits (liquid and plastic limits) of soils. This is mainly due to less operator dependency and more repeatable testing as opposed to the Casagrande cup (Feng, 2005; Hrubesova, Lunackova and Brodzki, 2016). Alternatively, the undrained shear strength of remoulded as well as intact (undisturbed) clay or fine-grained soil samples can also be determined based on

Hansbro's cone factor (k). The test is performed by allowing a specific cone and mass to free fall, typically 5 seconds, with its tip into a soil sample. The resulting penetration (mm) of the cone into the sample is measured. Penetration values can then be used to estimate the undrained shear strength of the soil sample using an empirical formula. According to Canelas, Fernandes, and Lopes (2018), the fall-cone test is specifically designed for cohesive soils and offers several advantages over other methods. It is quick, requires a smaller test sample, and has a broad range of applicability, particularly when the water content is near the liquid limit, something that is difficult to achieve with other tests such as the direct shear test, triaxial test, and unconfined compression test. The two most common testing standards for the fall-cone are ISO 17892-6 and BS 1377:2. This testing method has not yet been adopted by the vast majority of commercial laboratories in South Africa. This is due to dissatisfaction with the amount of sample required compared to the Casagrande cup and no widely accepted method of correlating test results between the two (Theron, et al., 2019). Vosloo (2022) recommends the use of the fall-cone device and made recommendations for the adoption of a South African standard. The study was based on remoulded soil samples rather than undisturbed samples. A single fall-cone setup, with all the relevant accessories and additional cone tips and weights, approximately costs R30 000.00.

2.5.5 Vane shear tests

The vane shear device is a simple and accurate test used since the early 1960s to determine the undrained shear strength of intact or remoulded cohesive soils (Gylland, Thakur and Emdal, 2016). Most site investigation manuals and codes include a description of the equipment and recommendations for its use. The device is mainly used for taking in-field measurements of excavations, trenches, and test pits and can also be used in the laboratory for further evaluations. The shearing resistance of soil is measured by rotating the four-bladed vane at a constant rate after the vane is driven into the soil. The torque required to cause failure is measured by a gauge, which provides the conversion to a shear strength value. Figure 2-12 illustrates a schematic sketch of the rotation and torque applied to the blades during testing.

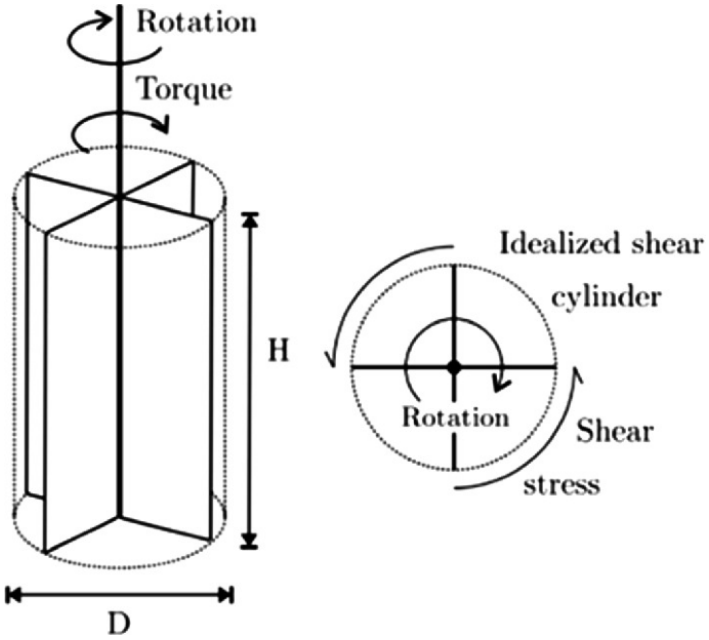


Figure 2-12: Vane shear blade (Gylland, et al., 2016)

Gylland et al. (2016) describe the vane shear test as a quick and reliable "clay detector" when the device is used and setup properly. The main advantage of the field vane test is that it provides an almost direct measurement of the in-situ undrained shear strength. Field vane shear testing is typically performed using vanes equipped with a slip coupling. According to Selänpää et al. (2017) slip coupling has a major role in the test measurement accuracy of field vanes, which can lead to inaccurate evaluation of the soil-rod friction. Torvanes and geovanes are both types of field vane devices that are also readily used to determine the undrained shear strength of soil samples. ASTM D2573 is the most prevalent standard relating to shear vanes. A shear vane field tester kit, which includes three interchangeable vanes and an extension rod, approximately costs R18 000.

The laboratory vane shear test can also be used to determine the shear strength of undisturbed or remoulded cohesive soils with very low shear strengths. The apparatus is considered a cheaper and quicker method to be used as opposed to the sophisticated triaxial or direct shear test. The vane consists of four blades fixed 90° to adjacent blades. From edge to edge, the vane is typically 12 mm in width with a height and thickness of 24 mm and 0.5 mm, respectively. The vane rod is 60 mm above the vanes. Two or three samples are prepared before a sample is mounted to the base of the vane shear apparatus. Samples of at least 30 mm in diameter and

75 mm long are used. The shear vanes are lowered into the soil sample to the full length of the blades without disrupting the sample. The blade tops ought to be 10 mm below the top of the soil sample. The vanes are then rotated at a constant velocity until the sample fails. The final reading is obtained from the torque indicator. A calibration curve is used to convert the readings since the laboratory vane shear does not read the torque directly. An empirical formula is used to obtain shear strength readings. The formula is based on the assumption that the shear strength in the horizontal and vertical directions is the same. Secondly, that the shear strength is equally mobilised at the end and centre surfaces. Lastly, the shear surface is cylindrical with a diameter equal to the diameter of the shear vane. Vanes should be treated occasionally to avoid corrosion and rusting, which could have an influence on measurements.

Overall, there are some drawbacks to vane shear devices. Accurate results may not be obtained if the failure envelope is not horizontal. Testing is also not suitable for clay soils containing sand lenses and seams or silt laminations (SUDAS, 2013). Furthermore, application for stiff clays is limited, slow, and time-consuming.

2.5.6 Pocket penetrometer

A pocket or handheld penetrometer is a simple, lightweight device used to classify soils in the field. The device also indicates consistency, shear strength, and approximates unconfined strengths of cohesive soils in a fast and reliable way (Yasun, 2017). A typical penetrometer device consists of a flat-tipped cylindrical rod loaded by a calibrated spring that is enclosed in a moveable encasing. The spring measures the force needed to drive the tip through a fixed surface and transfers that measurement to a scale. Values are obtained from the reading scale, which corresponds to the equivalent unconfined compressive strength of a soil specimen.

There are various models and types of handheld penetrometers available on the market. According to Thomas (1965) the penetrometer shape (sectional area, apex angle, shape of the rod), rate of penetration, and physical soil characteristics can all influence the penetration resistance. ASTM D2488 deals with the testing standard

of pocket penetrometers. A heavy-duty penetrometer device, which includes three interchange heads, approximately costs R8000.00.

2.5.7 Pressuremeter test

In the mid-1950s, Ménard (1956) first developed the pressuremeter test, which consists of a horizontal in-situ loading test carried in a borehole by means of an expandable cylindrical probe. The test can be used to obtain the deformation modulus, undrained shear strength (clays and weak rocks), the friction angle of sands, and the in-situ horizontal stresses of samples. The type of tests and interpretation of the data will determine the degree of success for any of the parameters. According to Braatvedt (1995) the major differences between the types of pressuremeters lie in the installation method of the device in the ground. Mair and Wood (1987) first categorised the pressuremeter into two broad tests, which are distinguished according to the installation method. The first is the Menard type (pre-bored), in which the device is installed in a pre-drilled borehole. The second is the self-boring type, in which the device bores its own way into the ground in order to minimise soil disturbance. The soil disturbance is the most significant factor affecting the pressuremeter results (Ferreira and Robertson, 1992).

Numerous scholars (Baguelin, et al., 1972; Windle and Wroth, 1977; Wroth and Hughes, 1974; Eden and Law, 1980) observed that pressuremeter tests overestimate strength values compared to other tests like the vane shear test, triaxial test, plate loading test, and the Dutch cone test. Over the years, pressuremeter testing devices and their associated design methodologies have been continuously improved; however, the undrained shear strength obtained from pressuremeter tests is generally higher than that from other field and laboratory tests. According to Isik, Ulusay, and Doyuran (2014), this overestimation is likely due to the different modes of failure and the presence of a disturbed zone around the probe. Rui and Yin (2017) conclude that consolidation affects the undrained analysis of soil, which leads to the overestimation of soil stiffness and strength for clay soils.

2.5.8 Static cone penetrometer

The static cone penetration test was originally developed in the Netherlands in the early 1930s and was first used to determine the ultimate bearing capacity of driven piles found in sand. The test method is also referred to as the Dutch sounding test, the Dutch probe, and cone penetration testing (CPT). Currently, the test is used across the world to assess soil behaviour and stratigraphy for hydrological, geotechnical, and geoenvironmental investigations. During testing, a 60º cone is pushed into the ground surface at a rate of 20 mm/sec. The device is typically fitted with a friction sleeve that matches the cone's diameter. Throughout the testing, the penetration resistance, total penetration resistance, and side friction resistance of the friction sleeves are measured continuously. The main advantages of this method are its simplicity and repeatability. Additionally, the test results lend themselves to rational analysis, as opposed to relying solely on empirical correlations (Braatvedt, 1995). A continuous record of soil resistance values is provided throughout the entire depth of penetration.

The main shortcoming is penetration depth constraints due to machinery capacity. The test is rarely effective in horizons with gravels and boulders (Braatvedt, 1995). Weathered rock profiles are also not suitable for this testing method. Furthermore, no soil samples are obtained from the site. Mechanical cone penetrometers have a telescopic action that requires an outer probe sleeve and inner rod, which leads to a slow incremental test procedure. They have limited accuracy when it comes to very soft soils and are labour-intensive. The electric cone penetrometer offers more accurate test results due to rapid testing, higher repeatability, and automatic data logging as opposed to the mechanical method. The friction sleeves and cone point move together as a single unit. Initial equipment costs and maintenance can be significant. A piezo-cone penetration test was later developed using a pressure sensor in the cone tip to measure in-situ pore pressure. The tip resistance is theoretically related to the undrained shear strength of saturated cohesive materials. Testing rigs remains very costly and is not very common for everyday investigations in South Africa. It is also important to note that CPT testing refuses in stiff soils.

2.5.9 Standard penetration test

The test was first standardised in the 1920s and 1930s. A standard split spoon sampler is driven into the soil at the bottom of a borehole. The test has been mainly accepted worldwide for foundation designs. A hammer freefalls from a set height, using a trip mechanism, providing the driving force. The number of blows required to drive the sampler in 150 mm increments is recorded. A total penetration of 450 mm is recorded. The first increment is discarded, and the sum of blow counts for the second and third increments is known as the “N” value. Results provide a sample for soil classification and provide an empirical qualitative guide to the in-situ engineering properties of cohesive and cohesionless soils (Braatvedt, 1995). Results of the SPT tests may be affected by incorrect drilling and sampling procedures. This is mainly due to inadequate cleaning of borehole beds and damage to the spoon tip. The freefall of the hammer may also not be obtained. Blow counts and penetration should be recorded accurately by the drilling crew when carrying out the tests. CPT tests are done close to the borehole positions to check the corelation between the two. This validates whether the SPT values are reliable (Braatvedt, 1995).

2.6 Factor of safety and soil variability

Safety factors, design margins, conservatism, and prudence are all preventative measures taken to lessen the possibility of dangers brought on by failure (Matthews, 1998). The proportion between a design value and failure is known as a factor of safety. For instance, if a rope carrying a person breaks under the load, the permitted load of the rope needs to be decreased by using a safety factor. In soil mechanics, the FoS can generally be described as the ratio of the shear strength at the plane of potential failure and the shear stress acting in the same plane. FoS values are used to reduce the risk of dangers caused by failure. In geotechnical engineering, FoS values are often highly generous and normally range from 3 to 5 (Handy and Spangler, 2007). A safety factor can be reduced if the soil is less variable and if the investigation is more thorough. A higher FoS is used for more variable soil and/or if the investigation is less comprehensive. Even a little increase or decrease of 0.5 on the FoS value can result in significant design cost increases

or decreases. It is therefore preferable to strive and reduce the FoS by increasing the number of soil tests and improving the statistical reliability of the results (Handy and Spangler, 2007). Therefore, a more economical test (vane shear and pocket penetrometer) that is performed repeatedly might be more accurate and give a higher confidence than an expensive and more sophisticated test (triaxial and direct shear test) that can only be performed two or three times for the same cost. In highway design where it is acceptable to have periodic failures, a lower FoS is generally used. Periodic repairs of weaknesses are more economical than overdesigning the entire project. In contrast, a higher FoS value is used on dams where failure could endanger countless lives.

2.7 Design Approaches

The main objectives of engineering design approaches are safety, reliability, and economy. The overall economy of designs involves balancing the cost of improved safety against the cost of probable failure. The simplest way to reduce the risk of probable failure is the use of safety factors, which are applied to loads and resistances according to various methods of design (de Koker and Elvin, 2018). Codes and standards have been developed and introduced over the years to assist engineers in appropriate decision-making while developing safe and economical designs in accordance with accepted methods (Lysay, 1999). The two design approaches currently used in South Africa for geotechnical engineering are the predominant Working Stress/Load Design (WSD) method and the less prevalent Limit States Design method (LSD) which is a simplified form for reliability-based design (RBD). The WSD method relies on factor of safety values to account for failure, while the LSD relies on multiple partial factors of safety to account for probable failure. Figure 2-13 on the next page illustrates the evolution of design methods in geotechnical engineering.

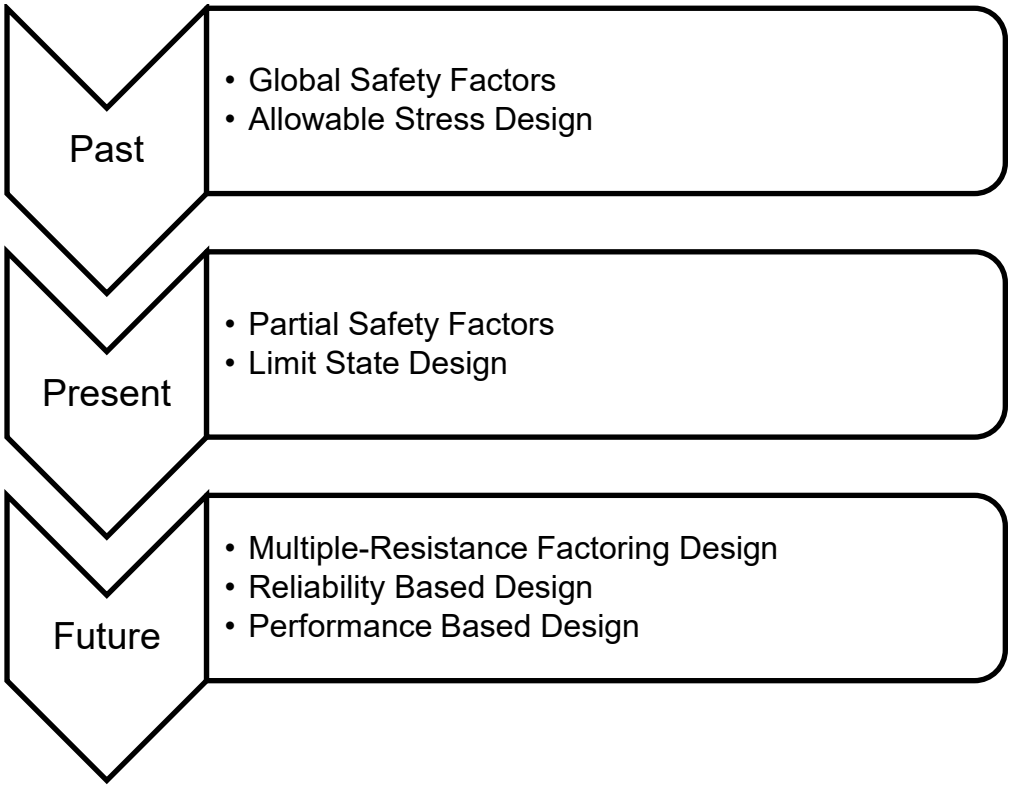


Figure 2-13: The evolution of design methods in geotechnical engineering (Bogusz and Godlewski, 2019)

2.7.1 Working Stress (Load) Design

Traditionally, geotechnical designs have relied on the Working Stress Design (WSD) approach. Subjective global factors of safety have been applied to account for various uncertainties, ensuring adequate reliability across a range of scenarios. Material resistance and uncertainty in loads are combined into a single factor of safety, which does not adequately account for variability of loads and resistances (Schweckendiek and Calle, 2010). Furthermore, factor of safety values for different applications are mostly derived from past observations and experience and are generally large enough to ensure that deformations are within acceptable limits (Bogusz and Godlewski, 2019; Chang, 2011; Reddy, 2014). This approach, however, does not provide sufficient margin of safety against every scenario, and the uncertainty involved relating to geotechnical design has led to overly conservative or unconservative designs (Allen, 2007). For this design approach, selected soil parameters are based on the mean values of the available data with the exclusion of outliers (Day, 2024). Even though the WSD design approach is

fundamentally flawed and outdated, with the factor of safety values being a poor measure of reliability, this design approach is still extensively used by industry and remains the preferred design approach for geotechnical practice in South Africa.

2.7.2 Limit States Design

The LSD concept is based on the use of partial safety factors, with each associated uncertainty, in order to decrease the probability of failure. Almost all partial factors used in this design approach were calibrated based on an overall factor of safety resulting from previous experience and standards and successful design (Phoon, et al., 2017). LSD uses characteristic values based on judgement or statistical distributions, as well as partial factors assigned to loads, materials, and resistances, to achieve a target level of reliability. The LSD concept has been widely adopted across the globe in the form of reliability design (Reddy, 2014). The most notable set of standards concerning this design approach are the Eurocodes, and more specifically Eurocode 7, for geotechnical design practice. Eurocodes is a set of European standards for geotechnical engineering that was developed by the European Committee for Standardisation (CEN). The standards provide guidelines for the design and construction of geotechnical engineering projects, such as foundations, slopes, and embankments. It has been adapted or adopted by many nations around the world. A similar course of action is currently being taken in South Africa. With the publication of SANS 10160 Part 5, the LSD approach was first introduced to South Africa for geotechnical design in 2010. A newer, updated version was later published in 2021.

2.7.3 Reliability Based Design

According to Fenton and Griffiths (2010), geotechnical design codes worldwide are increasingly adopting some form of reliability-based design (RBD). This shift can be seen in several standards and codes, such as Eurocode 7 (2003), AS 5100 (2004), AASHTO (2007), and the National Building Code of Canada (2005). RBD provisions are typically presented through limit state design, incorporating load and resistance factors. The load and resistance factors are calibrated to meet the desired target reliability levels for various limit states, which define the failure states. Resistance

factors for man-made materials are well established, but developing resistance factors for geotechnical applications is much more challenging. Thorough site investigations are required to estimate the mean properties of the soil and understand its variability, which is essential for producing reliable, reliability-based designs in geotechnical engineering. Little guidance is provided on determining the characteristic design values for soil on collected data or how to use estimated variation for design adjustments. Several studies have been conducted for the reliability-based optimisation of geotechnical systems, none of which seems fully bulletproof. The application is still very limited, with the main limitation being the difficulty of direct coupling between reliability assessments and cost minimisation (Mahmood, 2020). For the time being, LSD (Eurocode 7) remains the newly selected design approach in the country until an alternative design approach is standardised.

2.8 Determining the characteristic value(s)

Selecting characteristic values for “actions” and “resistances” is a fundamental aspect of this design approach (Eurocode 7 and SANS 10160-5). The process of determining suitable values for geotechnical parameters based on field and laboratory test results is referred to as ground characterisation (Bond and Harris, 2008). After applying appropriate partial factors to account for uncertainties in the available data, these values are used in design calculations. This process consists of three steps: first, converting test results into derived values; second, selecting a suitable characteristic value from the derived values; and third, applying partial factors to these characteristic values to ensure greater reliability for design purposes. Bond and Harris (2008) illustrate the overview of ground characterisation in the Figure 2-14 on the next page.

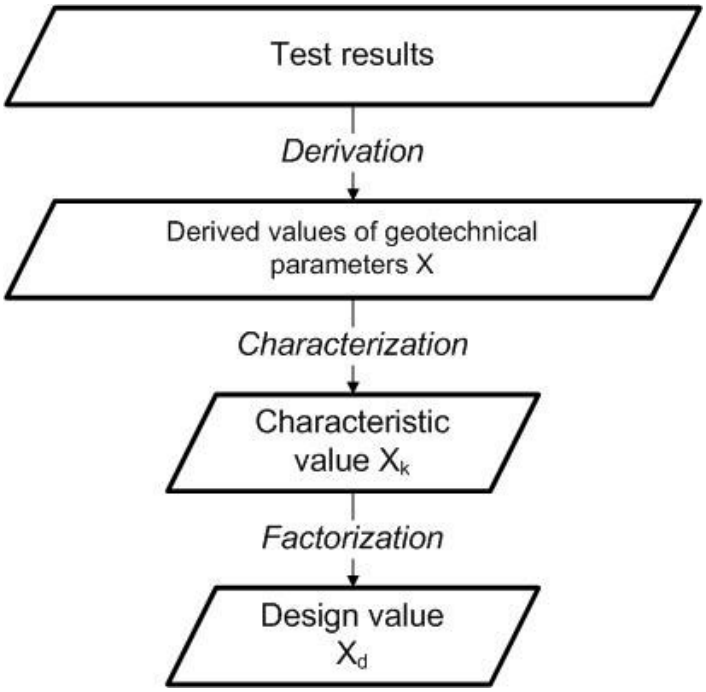


Figure 2-14: Overview of ground characterisation (Bond and Harris, 2008)

According to the Eurocodes (CEN: 1990), material properties (including those of soil and rock) should be represented by characteristic values. If the limit state verification is sensitive to the variability of a material property, both upper and lower characteristic values must be considered. According to CEN (1990), a characteristic material property is defined as follows:

"*where a low value of material or product property is unfavourable, the characteristic value should be defined as the 5% fractile value; where a high value of material or product property is unfavourable, the characteristic value should be defined as the 95% fractile value*." (CEN 1990 S4.2(3))

Figure 2-15 summarises the various methods for selecting a characteristic value according to Eurocode 7. Test results from laboratory or field tests can be converted into derived values through correlations, theoretical considerations, or empirical guidelines. Relevant data or research studies on materials encountered may be used to supplement the test results. Derived values can also be evaluated directly if the engineer is aware of the limit state for which the derived value is necessary.

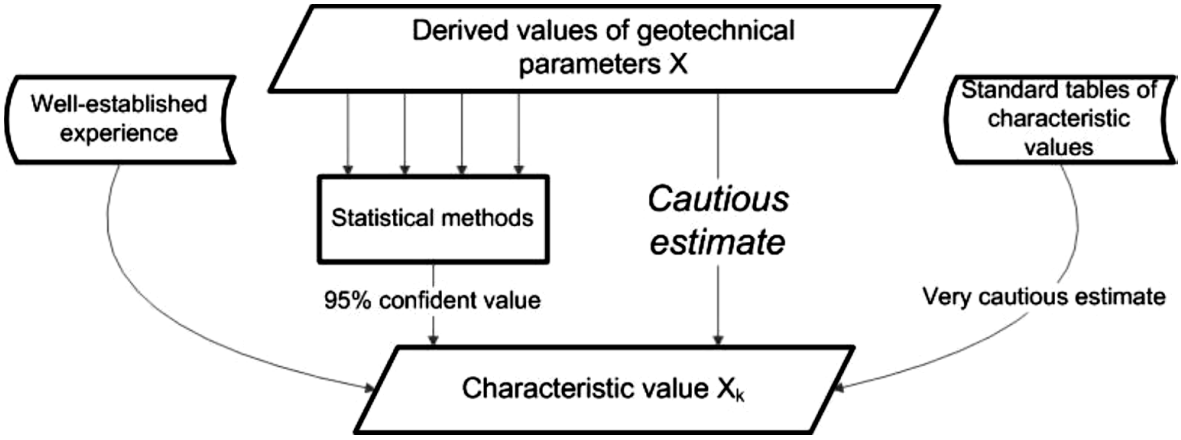


Figure 2-15: Characterising geotechnical parameters (Bond and Harris, 2008)

According to EN 1997 (2004), the selection of characteristic values for geotechnical parameters must be based on results and derived values obtained from laboratory or field tests, supplemented by "well-established" experience. What can be defined as “well-established” experience? Three years, five years or twenty years of experience? Furthermore, the characteristic value of a geotechnical parameter must be selected as a cautious estimate of the value affecting the occurrence of the limit state. For the majority of the circumstances, a cautious estimate is made from the derived values in selecting the characteristic value of any given geotechnical parameter. Increased caution is required when using standard tables of characteristic values in the absence of available data. The term “cautious estimate” has led to some debate and confusion among engineers and scholars alike. Bond and Harris (2008) define a cautious estimate as an approximate calculation or judgement that is careful to avoid problems or dangers. Eurocode has made a concession by allowing a “‘cautious estimate” of the value affecting the occurrence of the limit state rather than the fifth percentile required for other branches of engineering by Eurocode due to a lack of measured probability density functions for soils. Spear and Selenates (2011) reported that 94% of geotechnical experts were hesitant to make a “cautious estimate” and requested guidance on how to arrive at the required cautious estimate. Eurocodes provides little guidance. It appears apparent that there is no way to estimate the value of a soil property allowing only a 5% chance of failure except from establishing meaningful probability density functions.

If statistical methods are used, characteristic values must be selected so that there is no more than a 5% chance of failure due to a less favourable parameter value. To minimise the risk of failure, partial factors are applied to the characteristic values of actions and/or resistances (Stott, 2020b).

2.9 Representative values

Stott (2020b) notes that the testing methods and standards (mentioned in Section 2.5) used for obtaining the values of soil parameters are aimed specifically at the working load design approach, which only requires a representative or average value. The testing standards do not account for variability. Several testing standards, like the shear box, triaxial, etc., specify only three test values to be obtained; these values are then averaged and factored to obtain a design value. Testing standards and methods will need to be amended if industry in the country is to adopt and move over to the new Limit States Design Approach, which relies on probability density theory and statistical analysis for test values.

2.10 Using Statistical Methods

Statistical methods can be used to address variability and uncertainties relating to designs (Hernandez, 2017; Phoon, Kulhawy and Grogoriu, 1995). Several statistical methods are available. Probability density functions and the coefficient of variation (COV) are commonly used to quantify inherent soil variability of geotechnical properties (Phoon and Kulhaway, 1997; Cortellazzo, 2000). A large number of reliable measurements are required in order to perform detailed and reliable statistical anlysis. To establish a reliable PDF theoretically requires more than 600 tests (Eaton et al., 2019; Stott, 2020a; Stott, 2020b). It is therefore unfeasible for many soil shear strength tests to establish the 5th percentile of a soil property due to time and expenditure. It is rarely feasible to conduct more than a minimum number of tests due to financial cost and time constraints, which reduces our understanding of soil variability and knowledge of soil variability suffers (Blight, 2013). In order to find the value of a soil property that leads to at most a 5% chance of failure, one needs to construct a reliable PDF for measured values of the soil parameter required. The upper or lower 5% value can then be determined from the PDF.

Many statistical procedures are available for selecting and fitting a probability distribution to approximate datasets. Common techniques include plotting a histogram of the data and selecting a distribution that best fits it, as well as using Pearson's moment-based system for fitting the distribution (Baecher and Christian, 2003). The probability distribution is typically selected as lognormal when dealing with geotechnical strength properties (e.g., angle of shearing resistance and undrained shear strength). It is chosen to be lognormal because negative values are inadmissible (unlike most distributions), and most soil properties can be modelled adequately as lognormal random variables (Akbas and Kulhaway, 2009). Eurocode 7 assumes soil properties to have a lognormal distribution; however, testing undertaken by Stott and Theron (2019) illustrates measured probability density functions, which are not good approximations to a lognormal curve. Galeandro et al. (2017) note that none of the considered probability density functions used (normal, lognormal, gamma) for the specific study may be fully reliable. Eurocodes specifies a lognormal distribution. If the actual distribution of a soil property is not lognormal, the statistical analysis may be worthless.

2.11 Proposed tests to determine probability density functions

Based on the literature review, previous research, and testing done by the Soil Mechanics Research Group (SMRG) at the Central University of Technology (CUT), three testing methods are proposed for this research project in order to compare and evaluate the most sufficient method for more reliable soil variability analysis. These testing methods are conventional tests used to determine the soil resistance and shear strength of soils. Proposed tests are to be conducted on fine-grained soils with a moderate clay content (cohesive soils). Obtained probability density functions, from the proposed testing devices' shear strength results, will be compared to see if there are similarities in the patterns or distribution type. Shear strength readings obtained from the various test methods will also be compared and analysed. Furthermore, these testing devices can be utilised to obtain a large number of soil shear strength measurements in a shorter and more economical way than most shear strength tests as discussed in the previous section.

As discussed in the previous section, all soil parameters are assumed to have a lognormal distribution based on codes of practice (SANS 10160-5, Eurocode 7). These tests may provide an indication of how valid this assumption may be. Test results may lead to new procedures that actually account for soil variability.

2.11.1 Vane Shear

The handheld vane shear is an in-situ testing device that is simple and accurate in determining the undrained shear strength of cohesive soils. A large amount of data can be obtained with the use of this testing device (Arnold, Askarinejad and Zhang, 2018; Peila, Viggiani and Celestino, 2019). Most site investigation manuals and codes include the description of equipment and recommendations for its use. Eurocode 7 also recommends the use thereof. Shear vanes are mainly used for taking in-field measurements of excavations, trenches, and test pits and can also be utilised in the laboratory for evaluations. The device can be used on the side walls of test pits or trenches to obtain shear strength values from various soil layers present. The shearing resistance of soil is measured by applying a torque to the vane. Figure 2-16 illustrates a typical vane shear device used in the field.

Figure 2-16: Field vane shear apparatus with interchangeable blades (Controls Group, 2022)

Testing done by Stott (2020b) indicates that there is a good shear strength correlation between the vane shear test, fall-cone test, and suction shear testing.

2.11.2 Pocket penetrometer

The pocket/handheld penetrometer is a simple and cost-effective tool used for in-situ soil testing to evaluate unconfined shear strengths for soft soils. It approximates unconfined shear strength and indicates consistency in a fast and reliable way (Yasun, 2017). The testing procedure potentially makes possible a wide dataset, which allows for detailed variability analysis and research into the best fitting probability density functions (Galeandro, et al., 2017; Abdi, et al., 2020). Figure 2-17 illustrates a typical pocket penetrometer.

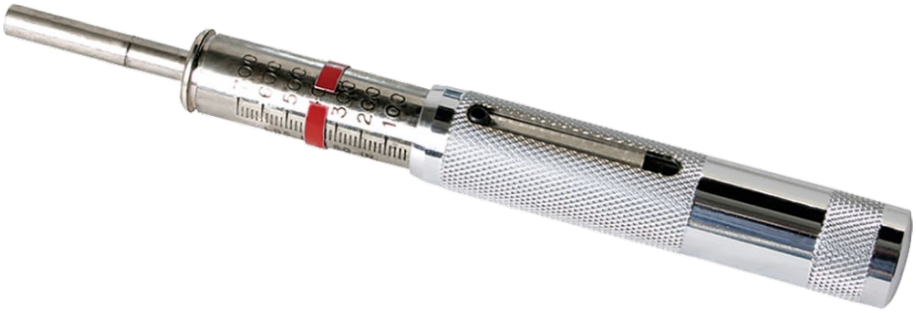


Figure 2-17: Typical pocket penetrometer device (Humboldt, 2022)

Galeandro et al. (2017) used this device in a case study to enhance our understanding of the inherent variability in geotechnical properties of clays. The study focused on marine stiff clay deposits, which were found to be relatively homogeneous. More than 800 unconfined compressive strength readings could be obtained, for various depths, from extracted soil columns from a borehole. To assess the inherent variability of strength, the study estimated the average value, standard deviation, and coefficient of variation (COV) of the measured compressive strengths. The results revealed that the variability in soil strength along the studied log was higher than anticipated for such a homogeneous soil deposit. Additionally, the study observed that none of the probability distributions considered (normal, lognormal, and gamma) could be deemed fully reliable after fitting the measured dataset to different statistical distributions. Figure 2-18 illustrates the fitting of data to PDFs and CPFs for various depths of a marine stiff clay deposit for this study.

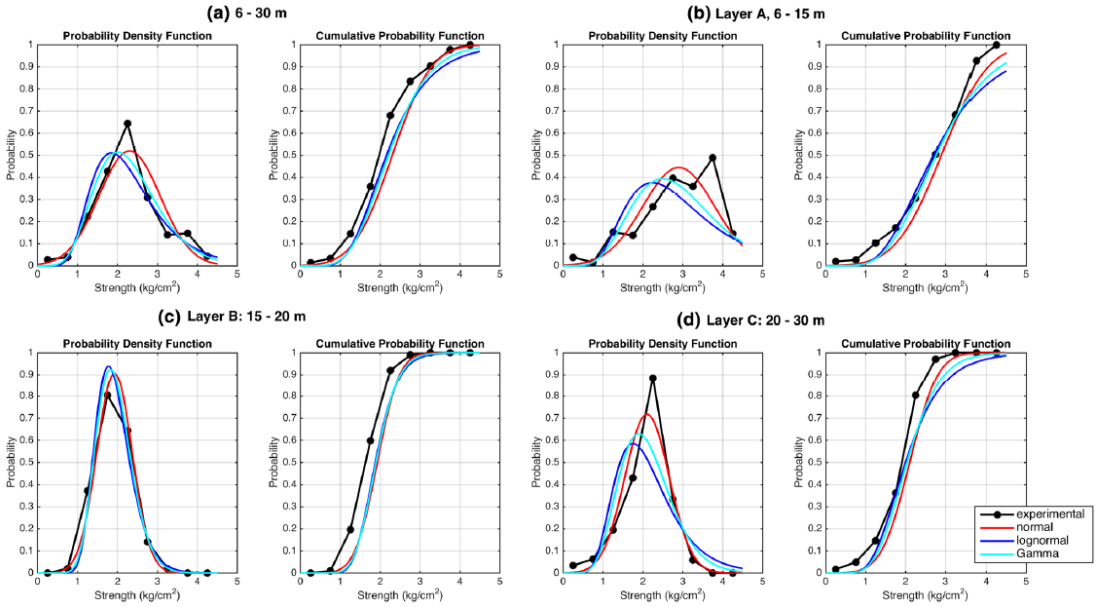


Figure 2-18: PDFs and CPFs for soil samples between various depths (Galeandro, et al., 2017)

For this research project, the researcher will attempt to follow a similar methodology in investigating and analysing the variability of undrained shear strength of soil samples. Instead of using the pocket penetrometer on extracted soil samples, the researcher will use the device to obtain shear strength measurements directly from various soil layers from excavated test pits.

2.11.3 Laboratory fall-cone

As mentioned in the previous section, the FC test is a laboratory test used to estimate the undrained shear strength of cohesive soils. This method is most commonly used on remoulded specimens of fine-grained soils. Hansbo (1957) proposed the following relationship between the cone penetration depth (d) and the undrained strength (S_u) of a soil:

$$S_u(kPa) = k.\frac{W}{d^2} \qquad (2)$$

Where $\boldsymbol{W}$ is the cone weight in Newtons and $\boldsymbol{d}$ the penetration depth of the cone in millimetres. The $\boldsymbol{k}$ represents the cone factor value which varies depending on the type of cone angles used. The analysis assumes a constant cone factor value as

proposed by Hansbo (1957). A similar expression was derived by Wroth and Wood (1978) from dimensional analysis. There are a number of international standards on the fall-cone test, with the main variations relating to the cone tip angle and cone mass. Table 2-3 below summarises the various fall-cone standards used around the world.

Table 2-3: Fall-cone standards across the world (adapted from Feng, 2005)

Country	Standard	Cone angle (◦)	Cone mass (g)
Australia	AS 1289	30	80
Canada	CAN/BNQ 2501-092-M86	60	60
China	SD128-007-84	30	76
France	NF P94-052-1	30	80
India	IS 2720-5	30.5	148
Japan	JGS 0142	60	60
New Zealand	NZS 4402	30	80
Norway	NS 8002	60	60
Russia	GOST 5184-49	30	76
Sweden	SS 027120	60	60
United Kingdom	BS 1377-2	30	80

Figure 2-19 from Dastider, Chatterjee and Basu (2021) illustrate the different cone types typically used for laboratory fall-cone testing.

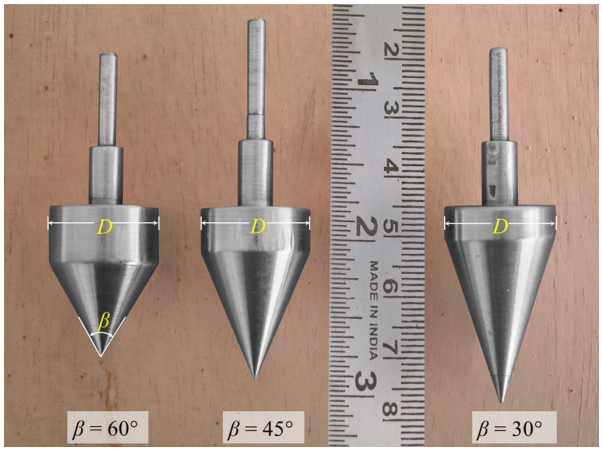


Figure 2-19: Typical cones used in laboratory (Dastider, et al., 2021)

The recently published ISO 17892-6 (2017) standard describes the fall-cone method as an estimation of shear strength, making the test as valid as the laboratory vane test, direct shear box test, or triaxial test, according to Canelas et al. (2018). Eurocode 7 also recommends and mentions the use of the fall-cone testing for the strength index testing of a soil. For this research project, the researcher will perform fall-cone testing on extruded undisturbed samples as opposed to testing on remoulded soil samples.

2.11.3.1 Variation of the fall-cone factor (k)

The selection of a fall-cone factor has a significant impact on how accurately undrained shear strength estimates are made (Dastider, et al., 2021). Numerous studies attempted to quantify the influence of different parameters (cone surface roughness, shearing rate, and cone angle) on the cone factor. Hansbo (1957) suggested that the cone factor depended mainly on the cone angle and is also influenced by the rate of shear and sensitivity of the clay at hand. Wroth and Wood (1978), Karlsson (1961) and Houlsby (1982) found that the specific type of soil, which represents the friction coefficient between the cone and soil was also an influencing factor, on the cone factor value, in addition to the cone angle. However, no research has been done to determine how the friction coefficient and cone angle affect the cone factor (Zeng, et al., 2020).

Houlsby (1982) and Brown and Huxley (1996) recognised the cone factor value as a function of the depth penetration. However, according to Feng (2005) no experimental data have been reported. Feng (2005) confirmed with experimental data that the cone factor value greatly decreases when the penetration depth decreases and as the undrained shear strength increases. Feng (2005) further concluded that the reduction in cone factor and in strength ratio had no effect on the criteria for determining the liquid and plastic limits of depth penetration. It is important to note that the experimental research was done on a remoulded kaolinite sample. A cone reduction factor can be obtained using Feng's (2005) results, as indicated in Figure 2-20. However, the proposed reduction factors may not be adequate for intact undisturbed samples, as the shear strength may be significantly higher than 100 kPa.

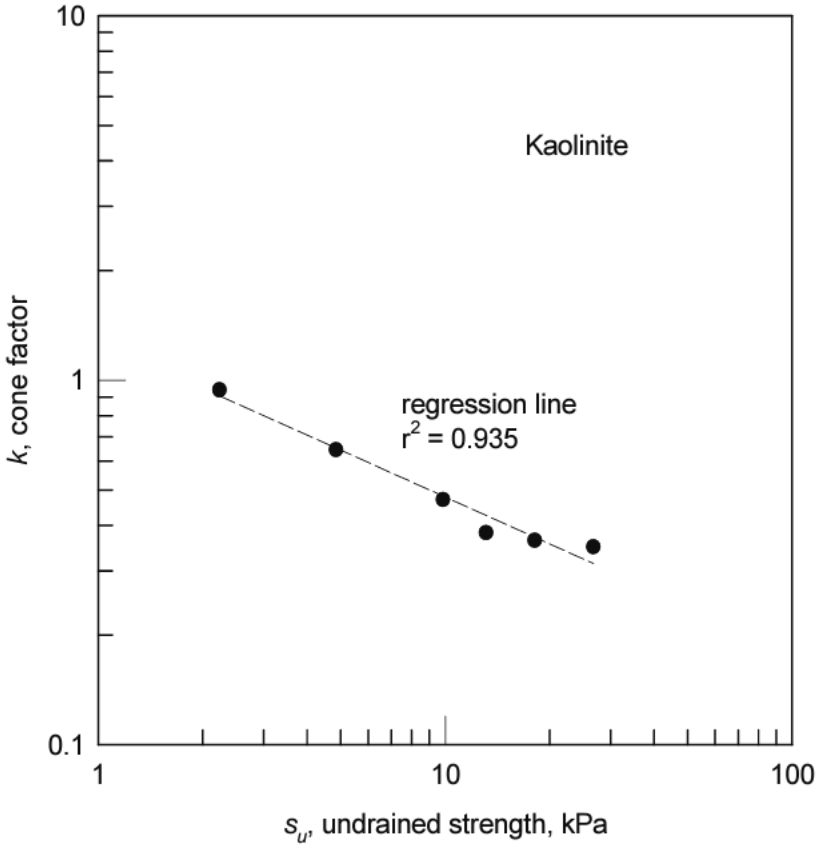


Figure 2-20: Relationship between the k value and undrained shear strength of a kaolinite sample (Feng, 2005)

Numerous scholars (Hansbo, 1957; Houlsby, 1982; Wood, 1985; Muntohar and Hashim, 2005) have claimed that the surface roughness of fall-cone devices also influences the cone factor value and penetration depth. Cone factor values determined by Wood (1985) were determined using cones wiped with an oily cloth in an attempt to keep the cone surface friction as uniform as possible. Zeng et al. (2020) also note that a thin layer of Vaseline should be applied to the surface of cones before testing in an attempt to reduce the effect of the cone surface roughness effectively. At the same time, Llano-Serna and Contreras (2020) have disputed these claims with experimental data, finding that surface roughness has little to no effect on the penetration depth. They concluded that this could be due to the small contact area of friction between the cone and the soil during penetration, which is not significant enough to alter the depth of penetration.

Hansbro (1957) recommended a cone value of 0.3 for a 60° cone used on remoulded clay samples. According to Llano-Serna and Contreras (2020) the cone

factor values for a 30° cone can range from 0.5 to 1.33. Table 2-4 indicates the various cone factors used and reported by various researchers.

Table 2-4: Reported cone factor values

Source	Cone angle (◦)	k value
Hansbro (1957)	60	0.3
Karlsson (1961)	30	0.7-0.86
	60	0.25-0.35
Wood (Hansbro) (1982)	30	1.2
Houlsby (1982)	30	0.96-smooth
		0.51-rough
Wood (1985)	30	0.85
	45	0.49
	60	0.29
	75	0.19
Zreik et al. (1995)	30	0.83

2.11.3.2 Determining the K factor for "undisturbed" clay

Hansbo (1957) highlighted the importance of relating the fall-cone test to the type of sampler used for undisturbed samples. In his research, he recommends a k value of 1.0 and 0.8 when using the ordinary piston sampler and pneumatic piston sampler, respectively, using a 100 g 30° cone. Hansbo (1957) further found that a cone factor value of 0.25 and 0.2 could be used for the two samplers observed when using a 100 g 60° cone. The sampler disturbance may have an influence on the cone factor value used. To date, very little research has been done on the cone factor values specifically for undisturbed samples using various samplers and/or sampling methods. Most of the cone factor values recommended by scholars have been on remoulded clay samples. Stróżyk and Tankiewicz (2013) conducted experimental research on undisturbed borehole samples using constant cone factor values of 0.8 and 1.0, as recommended by Hansbo (1957). The obtained results were corrected using a correction factor (μ) which is dependent on the liquid limit

(W_L) as proposed by PKN-CEN ISO/TS 17892:2009. The undrained shear strength should be corrected as follow:

$$C_{u(corr)} = \mu . C_u \quad (3)$$

Where:

$$\mu = \left(\frac{0.43}{W_L}\right)^{0.45} \quad (4)$$

The study found that most of the samples collected showed the undrained strength being greater than 300 kPa which can be classified as weak rock. The study concluded that the relationship derived by Hansbo (1957), as recommended by PKN CEN, does not apply to stiff, heavily consolidated clays. According to Stróżyk and Tankiewicz (2013), further research is needed to establish the correct relationship between cone penetration, cone mass, and strength when using the fall-cone test to evaluate the undrained shear strength of consolidated clays. Significant uncertainties remain in estimating cone factor values when fall-cone test results are used to predict the undrained shear strength of fine-grained soils (Llano-Serna and Contreras, 2020).

2.12 Summary

The most common shear strength tests each have their own advantages and shortcomings. Test equipment can be expensive and requires sufficient knowledge and understanding of how the apparatus operates. Most testing procedures can only obtain a hand full of readings, which are then averaged to obtain a “representative” value of the sample at hand, disregarding the range of variability. Both design approaches rely on the same conventional testing methods to determine characteristic values for design calculations. The Working Stress or Load Design approach is based on the ratio of capacity and demand (FoS). This approach does not consider statistical distribution of parameters directly and is now an outdated design approach internationally. LSD, a highly simplified form of reliability-based design, achieves a target level of reliability using characteristic values based on judgement or statistical distributions. Statistical methods can be used to account for soil variability. Reliable probability density functions require a vast amount of test

values to obtain a reliable 5% or 95% fractile value for design input. This can be done by using simpler, more economical, and faster test methods.

Research shows that the undrained shear strength distribution type might not be the same as other engineering materials like manufactured products. Eurocode 7 or SANS 10160-5 (LSD) can be followed to obtain design values that could be used with a reasonably high degree of confidence. However, doing so would necessitate abandoning present testing techniques, which are bound by the load factor design approach. This will require an overhaul of several testing standards and methods used. SANS10160-5 does not recommend which testing methods or standards to be used for determining characteristic values. South Africa is still far from establishing its own relevant shear strength testing standards (SANS standards) as opposed to relying on testing standards from overseas countries (AASHTO, ASMT, BS, ISO, etc.).

CHAPTER 3: STUDY AREA AND RESEARCH METHODOLOGY

The previous chapter presented the literature review of the study. The literature review primarily covered the two main design approaches used in geotechnical engineering along with the various shortcomings of different shear strength testing methods available. The chapter identified three testing methods to be investigated in the research study to obtain reliable probability density functions. This chapter consists of the study area and methodology followed for this research project.

3.1 Introduction

The research presented in this chapter is based on tests conducted in the field and laboratory by the researcher together with members of the SMRG. Undisturbed samples were collected from soil layers/strata in-field in various test pits for the laboratory fall-cone test and protected against drying before testing. The pocket penetrometer and the handheld shear device were both used to obtain measurements from the same soil layer/strata infield as the collected undisturbed samples. The number of tests conducted with these devices is dependent on the test pit size, soil layer size, and the soil conditions encountered in the field. Data obtained from each test were captured on data sheets for later analyses using Microsoft Excel and MATLAB.

3.2 Site locations

Soil testing and sampling for this study were conducted at various locations in and around the Manguang Metro Municipality (MMM), as follows: Tempe, CUT Agricultural Farm, Bloemdal, Somerton Estate, Bloemspruit, and Estoire. The location of each site is indicated in yellow and is shown in Figure 3-1.

The researcher also attended site investigations at the following locations: Heidedal (SASSA building), Roodewal (Haasbroek St), Ehrlich Park (Bloemwater), Groenvlei (Route 66), and Estoire (liquor distribution centre). These locations are indicated in red in Figure 3-1. However, due to unfavourable soil conditions (non-cohesive soils) and/or site characteristics, sufficient data and reliable measurements for analysis could not be obtained from the latter sites.

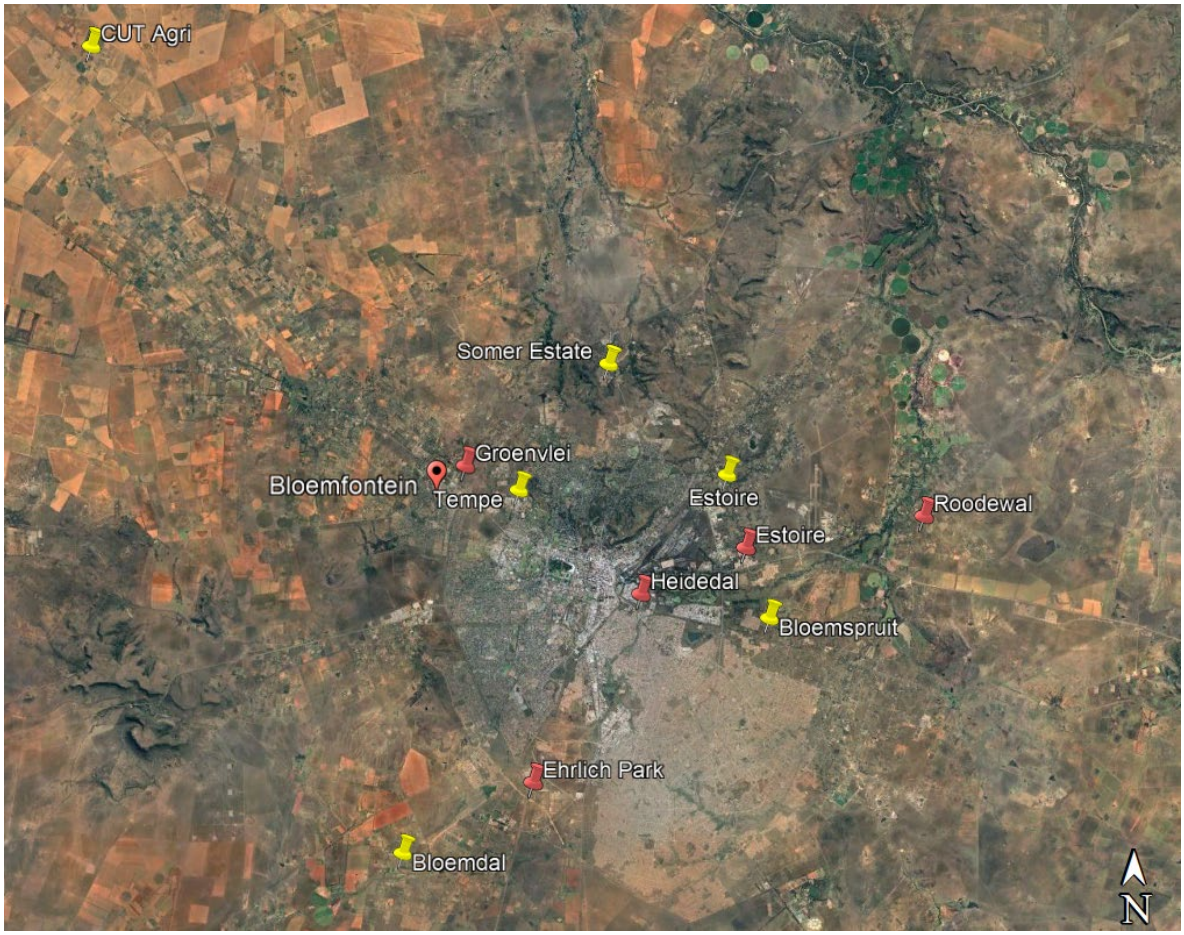


Figure 3-1: Locations of various sites investigated

3.3 Site Geology

According to the geological map of South Africa, the sites scattered across the MMM fall under the Adelaide Subgroup of the Beaufort Group of the Karoo Supergroup, which mainly consists of mudrock and sandstone (Pa) and dolerite (J- d), as shown in Figure 3-2.

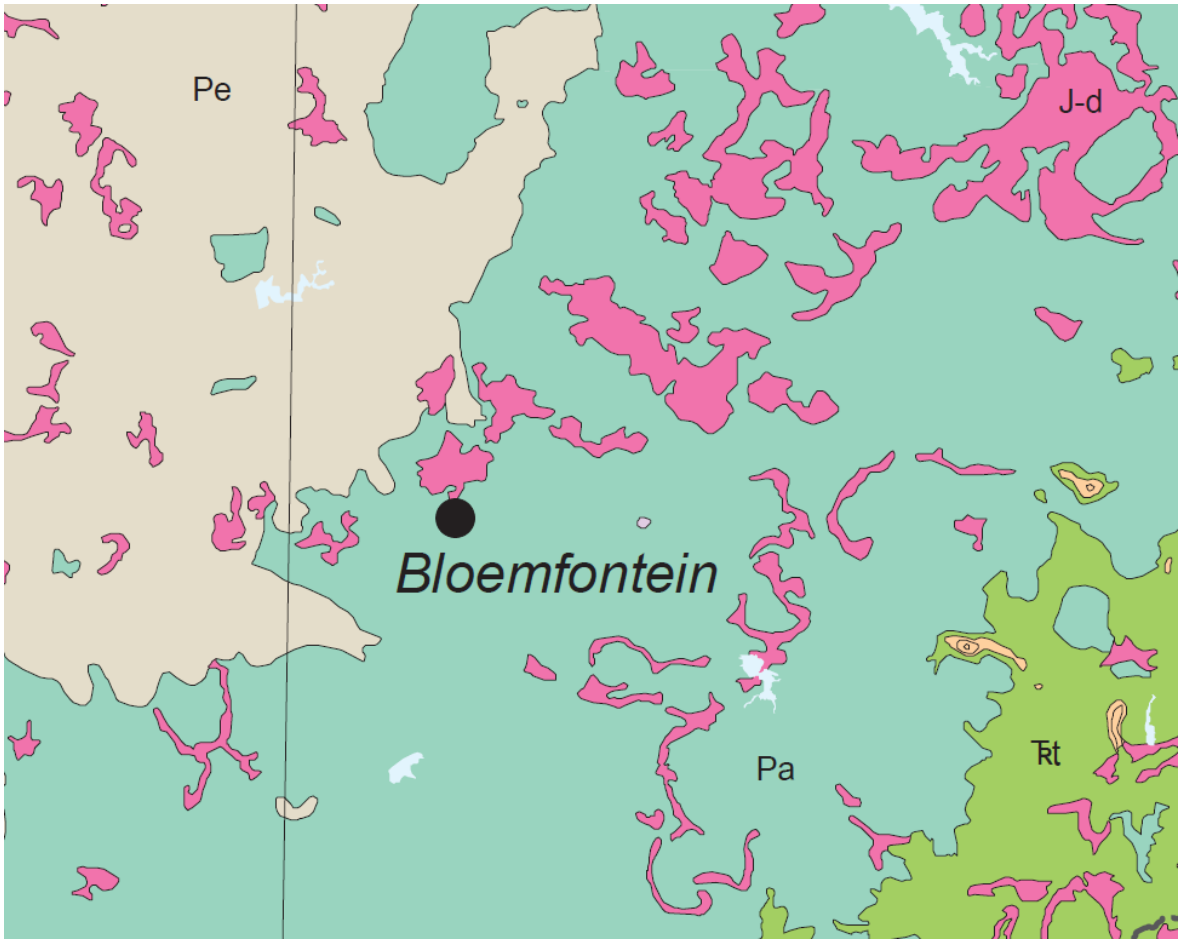


Figure 3-2: Dominant lithologies of the Bloemfontein area (Geoscience, 2008)

3.4 Physical properties and classification of soil samples

Soil samples collected during the site investigation were described following SANS 633 (2012). A geotechnical engineer from GeoCalibre assisted the researcher with the basic physical descriptions to ensure accuracy and consistency. Table 3-1 summarises the physical properties of samples from the various sites visited.

Table 3-1: Basic physical properties for each sample

Site name	Sample name	Soil description
Tempe	TP1L2	Slightly moist, light greyish brown, stiff, structureless, clayey-silt, residual mudrock, traces of fine roots.
CUT Agri	TP2 L4	Moist, light creamy brown, mottled orange and black, firm, structureless, sandy-clay, residual mudrock, traces of ferricrete nodules, traces of fine roots.
	TP4L4	Moist, light grey, mottled orange and white, structureless, firm, sandy-clay, residual mudrock, traces of calcrete nodules.
Bloemdal 1	TP4L1	Slightly moist, light grey, blotched cream, firm, structureless, clayey-silt, residual mudrock, traces of fine roots.
	TP6L3	Slightly moist, light grey, blotched yellow, mottled black, firm, slickensided, clayey-silt, residual mudrock, traces of ferricrete nodules.
	TP7L2	Moist, light orangery brown, blotched olive, firm, structureless, sandy-silt, reworked, residual sandstone, leached fabric, traces of fine roots, reworking decreases with depth.
Somerton	TP1L2	Slightly moist, dark grey, speckled white, firm, shattered, silty-clay, alluvium, minor fine roots, traces of ferricrete nodules, rapid transition to weathered sandstone bedrock at the base.
Bloemspruit	TP1L2	Moist, dark greyish brown blotched, blotched purple and olive, stiff, slickensided, residual mudrock, traces of calcrete nodules.
Estoire	TP1L2	Moist, dark gey, stiff, slickensided, silty-clay, alluvium, traces of fine roots.

Bloemdal 2	TP4L2	Very moist, dark brown, blotched grey, firm, structureless, clayey-silt, alluvium, moderate seepage at the base.

3.5 Field testing and sampling

Field testing and sampling for this study was carried out by the researcher and members of the SMRG, along with an industry partner, GeoCalibre. A field kit containing a pocket penetrometer and vane shear device along with interchangeable heads and vanes was used for in-situ field testing. Sampling rings required for extruding undisturbed samples from the site were manufactured by the Product Development Technology Station (PDTS), which falls under the department of Mechanical Engineering at CUT. The rings were designed and manufactured according to a standard oedometer ring used in the SMRG laboratories for consolidation testing. Stainless steel was selected as material for the sample rings to increase longevity by limiting corrosion as opposed to steel. Figure 3-3 illustrates the manufactured sample ring.

Figure 3-3: Manufactured sample ring from PDTS

3.5.1 Infield Soil Testing

Soil shear strength readings were obtained by the researcher using both a pocket penetrometer device and a vane shear device on the same soil layer/strata for each individual test pit. Measurements with the pocket penetrometer were made by pressing the retracting head of the penetrometer device into a soil layer of the trench sidewall. The red plastic indicator ring should be set at the lowest reading on the

scale before testing. The device is held at a right angle to the surface of the soil being tested before the device is slowly pushed into the soil layer at hand. On a calibrated scale, the amount of force necessary to press the head into the soil is measured. The plastic indicator holds its position after the head is released. Figure 3-4 shows pocket penetrometer testing in-field. A great number of readings were obtained from the various soil layers at hand.

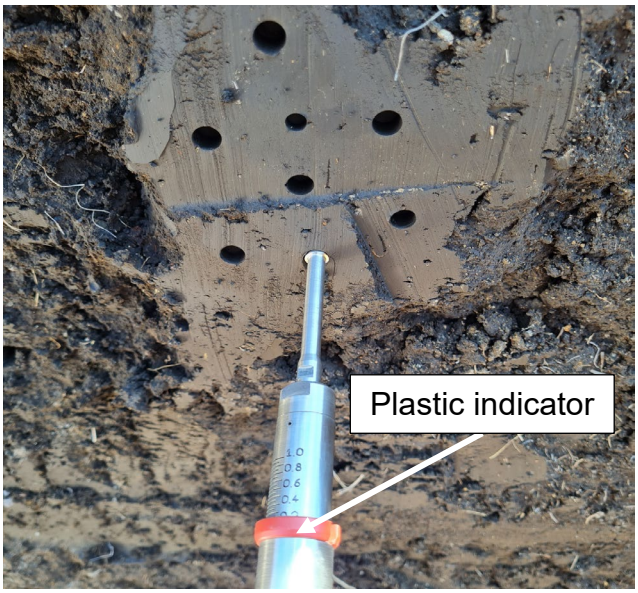


Figure 3-4: Pocket penetrometer testing in-field

The vane shear device was also used to obtain measurements from soil layers from the sidewall of each trench for the various sites investigated. The spring-loaded device with a vane blade was pushed into the soil sample at hand and turned in order to obtain a reading. After each measurement, the vanes were cleaned with a cloth and brush in order to remove any clay or soil attached to the vane blades. Figure 3-5 shows the vane shear after testing the soil layer at hand.

Figure 3-5: Vane shear after testing

Depending on the soil stiffness/softness, different penetrometer heads and vane shear blade sizes were used. For softer soils, a bigger head is selected, and for stiffer/harder soils, a smaller head is used. Test measurements for both testing devices were written down in a field book by an assistant for later capturing and assessment using Microsoft Excel and MATLAB, respectively.

3.5.2 Soil sampling

After excavation was done by a TLB on site, undisturbed samples were obtained for fall-cone testing from the side walls of the excavated trenches. Manufactured stainless steel sampling rings, 5.5 cm in diameter and 2 cm in height, with a cutting edge on their top side, were inserted into the various soil layers observed. The rings were inserted in the same direction as the other two methods to account for anisotropy. A hammer (mallet) was gently used with a wooden block in order for the sample rings to penetrate stiffer soil layers. After the sample ring had penetrated the soil layer to the full depth of the sample ring, a geological pick was used to mark the area around the sample ring. An area of approximately 20 by 20 cm was marked out to be cut and removed. A large lump of soil was then removed from the soil layer of the trench sidewall. Thereafter, a plastic disc was placed on the bottom side of the sample ring. Perforated wrap was also used to wrap the soil sample. This was mainly done for prevention of moisture loss and disintegration of the sample during transportation. Samples were named and labelled for later testing (FC) in the

laboratory. The typical collection of the undisturbed soil samples is shown in Figure 3-6.

Figure 3-6: Sample collection for FC testing

3.5.3 Sample preparation before testing

This section outlines the procedure followed once soil samples from various geotechnical investigations were delivered to the CUT laboratory. The sample preparation consisted of carefully cutting off any disturbed material at the top of the sample ring and placing the sample on a flat surface below the fall-cone apparatus. Once the cone is locked in an elevated position, the supporting assembly can be lowered so that the cone tip barely touches the surface of the soil sample.

3.5.3.1 Testing apparatus used to perform fall-cone testing

The following equipment was used to perform the fall-cone test on the various samples collected from site:

- Sample ring;
- Wire cutter;
- Fall-cone device, which includes the cone holder and an automatic release mechanism set to a release period of 5 seconds;
- Assembled 30° cone with an 80 g weight (shaft with cone) depending on the particular soil sample at hand;
- Additional weights (to increase the weight);
- Digital depth gauge measuring the penetration depth;
- Laboratory scale for weighing;
- Fall-cone test sheet.

3.5.4 Testing procedure followed for the fall-cone test

The undisturbed samples collected from the various sites were tested in the laboratory, according to ISO 17892-6 (2017), with means of a standard fall-cone device. Any excess material at the sample ring's top was cautiously cut away with a wire cutter, leaving a clean, fresh, and flat surface for fall-cone testing to commence. The amount of excess material to be cut off largely depends on the sample size. A small chunk of the trimmings was also taken to determine the moisture content of the soil sample at hand. The test sample was then placed on the flat surface of the testing device, with its smooth and level surface below the cone. The cone was lowered so that it just touches the surface of the soil sample before it is locked into place. The initial zero penetration reading of the device was recorded with a digital depth gauge. Figure 3-7 on the next page illustrated the fall-cone setup before testing.

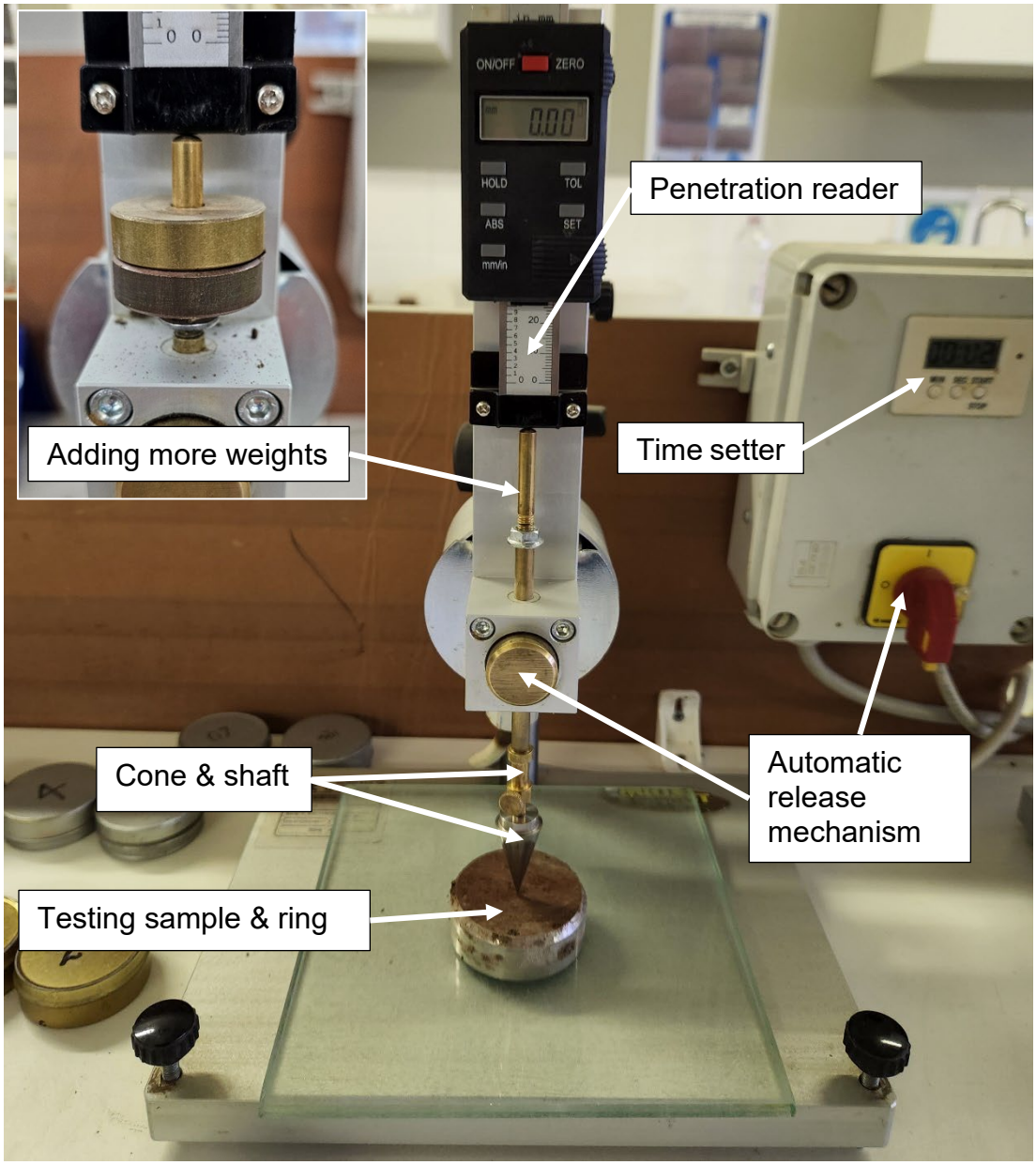
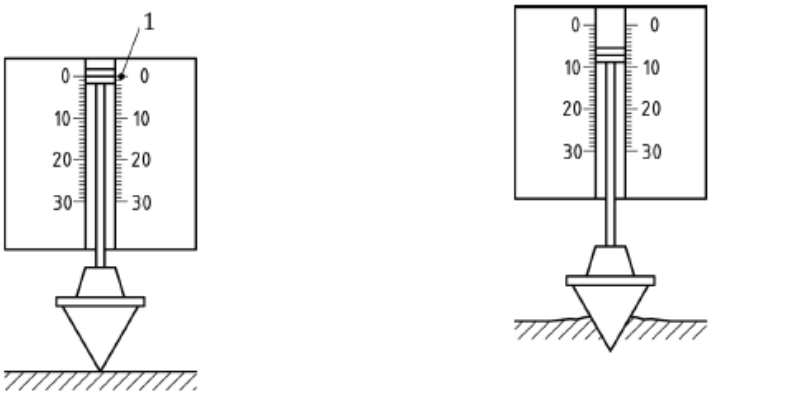


Figure 3-7: Fall-cone setup before testing

The cone is released by a push of a button on the time setter, activating the automatic release system. The mechanism allows the cone and shaft to penetrate the soil sample for a specified duration of five seconds. After this period, the system locks to prevent further penetration. The digital depth gauge is then used to measure the penetration depth of the standardised cone as it freely falls from a predetermined height and embeds itself into the soil sample.

Figure 3-8 illustrates the typical position of the cone prior to and after the release and fall of the cone.



a) Position of cone prior to fall

b) Typical position of cone after fall

Key

1 measurement scale, in mm

Figure 3-8: Fall-cone test before and after testing (ISO 17892-6, 2017)

After each and every reading, the cone is raised and lifted out of the soil sample to be cleaned carefully before the next reading can be obtained. Figure 3-9 illustrates penetrations on a sample after testing.

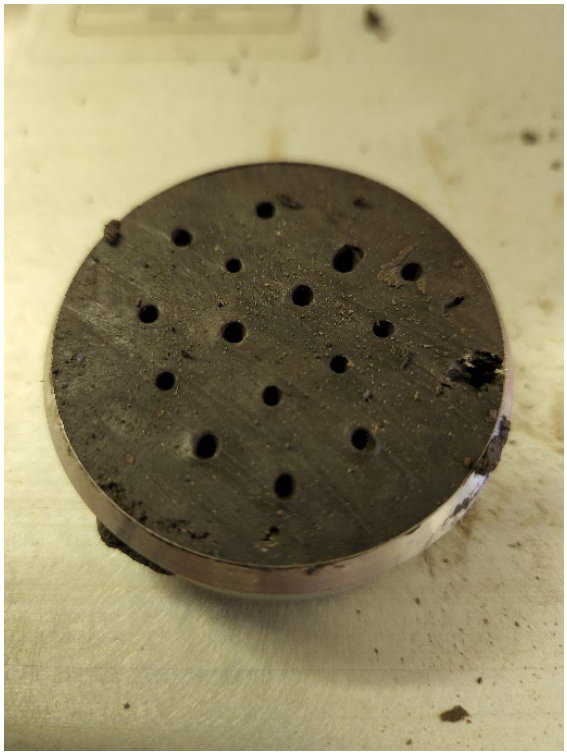


Figure 3-9: Penetrated soil sample (Estoire)

Lastly, the type of cone and weight used for testing are recorded. The cones were not wiped with an oily cloth or substance as recommended by Wood (1985) and Zeng et al. (2020), since these claims were disproved by Llano-Serna and Contreras (2020). The ISO standard also makes no mention of using an oil layer on the cones. ISO 17892-6 (2017) specifies a minimum of three testing points or measurements with the same cone in order to determine the average penetration value of an undisturbed sample. For the purpose of this study, the researcher attempted to obtain a maximum number of measurements (more than the standard of 3 test measurements) possible from each sample in order to have sufficient measurements for the data analyses in order to account for variably and obtain reliable probability density functions as proposed by Stott (2020a; 2020b).

An empirical formula is used to determine the undrained shear strength of a soil sample in accordance with ISO 17892-6 (2017):

$$c_{ufc} = c \,.\, g \,.\, \frac{m}{i^2} \tag{5}$$

Where;

c_{ufc} is the undrained shear strength of the undisturbed soil specimen in its tested state (kPa);

$c\ (k)$ is the cone constant used, dependent on the tip angle of the cone, where:

$c = 0.80$ for cones with a 30° tip;

$c = 0.27$ for cones with a 60° tip;

g is the acceleration due to gravity at free fall (9.81 m/s^2);

m is the mass of the cone (g);

i is the average cone penetration (mm);

The cone factor value recommended by ISO 17892-6 (2017) is slightly less (0.8) than the factor value recommended by numerous scholars as tabulated in Section 2.11.3. The recommended cone weight of 80 g was used for the first few site investigations. Heavier weights of 100 g and 400 g were later used to obtain higher penetration readings on the samples obtained from site.

3.6 Summary

Testing was done in the field with two handheld devices for quick in-situ shear strength testing. Undisturbed samples were taken for fall-cone testing in the laboratory using the recommended cone factor and weight from literature and the ISO standard, respectively. An empirical formula estimated the shear strength values of the undisturbed samples, which can then be compared with the in-situ field results. Readings are then analysed and assessed using statistical methods as described in the following chapter.

CHAPTER 4: RESULTS AND FINDINGS

4.1 Introduction

In the previous chapter, the site location(s) and site geology were presented. The in-field and laboratory testing methods for this research project were discussed, along with the sampling technique used for the undisturbed samples. The testing apparatus for the fall-cone test was also listed.

This chapter reflects the results of this study and presents empirical data. The results show the average shear strength readings and COVs for each site between the various testing devices used. Lognormal probability plots, probability density functions and violin plots, for each test sample are also presented. This chapter discusses and evaluates the testing methods used for the various sites, the problems, and challenges.

The results and findings of this chapter may lead to new ways of analysing soil variability for perspective researchers and practicing geotechnical engineers.

4.2. Problematic vane shear

Even though the vane shear test is a widely adopted and used method for estimating the undrained shear strength of cohesive soils, it was not suitable for testing for this research project. This is mainly due to the testing duration, procedure, and its low shear strength capability.

For almost all the soils encountered on the various site investigations, the vane shear had difficulty obtaining measurements. Most of the soils encountered were very stiff and/or very hard in nature, and the operator had great difficulty penetrating the vane blades into the soil layers at hand. The vane shear could only measure to a maximum of 280 kPa when using the smallest blade available. Furthermore, after testing, the blades would also need cleaning since the cohesive soil attaches to the blades, as illustrated in Figure 4-1. Cleaning can be difficult and time-consuming due to the stickiness of samples. Cleaning the blades with a damp cloth or even a steel brush may also influence subsequent test results.

Figure 4-1: Vane blade after testing

Due to the small number of readings obtained, no comparison or correlation could be made between the vane shear and the other two testing devices used for the research project. Most of the soils gave much higher shear strength readings (>280 kPa) using the pocket penetrometer and fall-cone devices. This method is thus more effective for soft clays with low shear strengths, which was not the case for this research project. Using or maybe even designing smaller vane blades might overcome this challenge; however, this was not part of the project scope. Vane shear testing was conducted in the same direction as the other two test methods to ensure consistency and account for soil anisotropy. This alignment helps provide more accurate and comparable results across the different testing procedures.

4.3 Fall-cone challenges

The fall-cone method is a widely used method to determine the liquid limits and estimate the shear strength of fine-grained soils. However, this test method may not always be appropriate for determining the shear strength of soil in all cases. This is due to low shear strength readings obtained on undisturbed soil samples and the empirical formula used to convert penetration data to shear strength readings. The k factor value influences the correlation between the fall-cone results and undrained shear strength of soils. Due to different standards and calibration methods, there

are varying cone factor values available, which influence the consistency and comparability of test results across various regions, countries, and testing protocols. The main challenges of the fall-cone test method on undisturbed soil samples are discussed and highlighted in the following sections.

4.3.1 Low penetration readings

Using the prescribed recommended cone weight of 80 g, from the literature and ISO 17892-6, provided very low shear strength readings for fall-cone testing. The smaller the penetration reading, the greater the shear strength. The difference in shear strength between two penetration readings also increases significantly as the penetration values decrease. For the majority of sites, the recommended 30° 80 g cone was used, and latterly the mass of the cone was increased to 400 g to obtain higher penetration readings. Table 4-1 indicates the average penetration readings obtained from the various sites along with the cone mass for each site's samples.

Table 4-1: Average penetration and shear strength using the FC

Site name	Sample name	Average I (mm)	Cone weight (g)	Shear strength (kPa)
Tempe	TP1L2	1.42	80	468
CUT Agri	TP2 L4	1.78	80	306
	TP4L4	1.94	80	262
Bloemdal 1	TP4L1	0.52	80	6257
	TP6L3	0.35	80	29559
	TP7L2	0.17	80	471477
Somerton	TP1L2	0.71	80	8078
Bloemspruit	TP1L2	2.98	400	571
Estoire	TP1L2	4.21	400	207
Bloemdal 2	TP4L2	5.76	400	113

Bloemdal 1 gave the lowest penetration readings with the highest shear strength for all of the sites investigated, followed by Somerton. Bloemdal 1 and Somerton were not analysed further due to unrealistic shear due to very low penetration readings

(<1 mm). The increase in the cone mass did not significantly increase the penetration readings between site samples.

Figure 4-2 and Figure 4-3 indicate the relationship between penetration readings (1 mm to 20 mm) and the shear strength for each penetration using a 30° 80 g cone and a 30° 400 g cone, respectively.

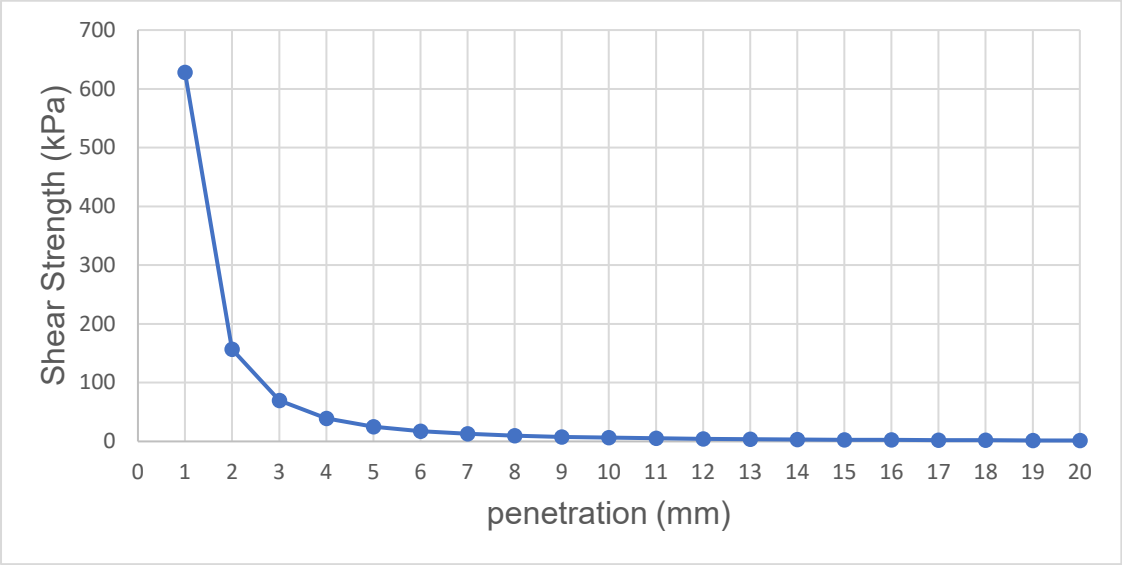


Figure 4-2: Penetration and shear strength relationship for a 30° 80 g cone

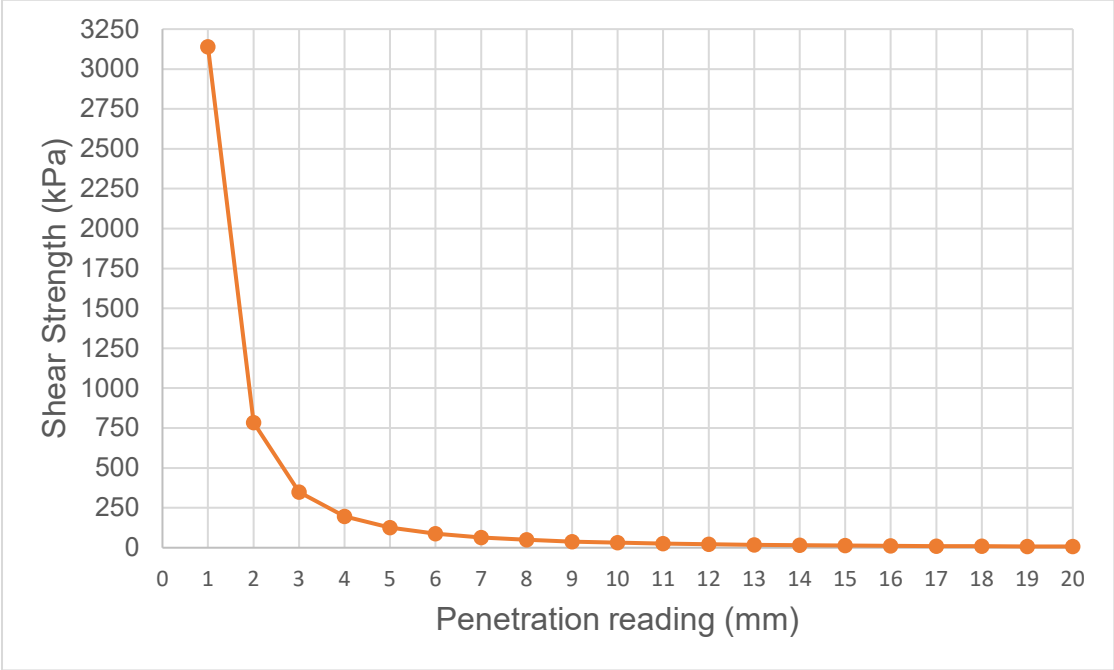


Figure 4-3: Penetration and shear strength relationship for a 30° 400 g cone

From Figures 4-2 and 4-3, it is evident that the shear strength increases slightly below the 11 mm penetration mark and rises significantly below the 4 mm penetration, with the most notable increase occurring below 1 mm. Table 4-2 below presents the differences and increases in shear strength between penetration depths for a 30°, 80 g cone. The higher the penetration reading, the smaller the jump/increase in shear strength; the lower the penetration reading, the higher the jump in shear strength. The difference in strength between penetration readings is even greater when using a 400 g weight, as illustrated in Table 4-2.

Table 4-2: Shear strength increases for corresponding penetration readings

	30° 80 g cone		***30° 400 g cone***	
penetration (mm)	**shear strength (kPa)**	**difference in strength (kPa)**	**shear strength (kPa)**	**difference in strength (kPa)**
20	1.570		7.848	
19	1.739	0.170	8.696	0.848
18	1.938	0.199	9.689	0.993
17	2.172	0.235	10.862	1.173
16	2.453	0.280	12.263	1.400
15	2.790	0.338	13.952	1.690
14	3.203	0.413	16.016	2.064
13	3.715	0.512	18.575	2.559
12	4.360	0.645	21.800	3.225
11	5.189	0.829	25.944	4.144
10	6.278	1.090	31.392	5.448
9	7.751	1.473	38.756	7.364
8	9.810	2.059	49.050	10.294
7	12.813	3.003	64.065	15.015
6	17.440	4.627	87.200	23.135
5	25.114	7.674	125.568	38.368
4	39.240	14.126	196.200	70.632
3	69.760	30.520	348.800	152.600
2	156.960	87.200	784.800	436.000
1	627.840	470.880	3139.200	2354.400

Penetration readings between 1 mm and 2 mm indicate the greatest jump/difference in shear strength. A slight penetration difference of 0.1 mm can have a significant effect on the shear strength outcomes when using Hansbo's (1957) equation. These penetration readings may not provide accurate and realistic shear strength values and may also introduce outliers and variation to the overall sample data.

4.3.2 Transformation uncertainty

The fall-cone relies on an empirical formula to convert penetration readings to shear strength values. The formula introduces transformation uncertainty as highlighted in chapter 1 and is evident in the COV of measured penetration data and the COV of converted shear strength values as shown in Table 4-3 below.

Table 4-3: COV values for FC penetration vs shear strength values

site name	sample name	penetration (mm) COV %	variability degree (Harr, 1987)	shear strength (kPa) COV %	variability degree (Harr, 1987)
Tempe	TP1L2	28	moderate	52	high
CUT Agri	TP2 L4	39	high	65	high
	TP4L4	36	high	66	high
Somerton	TP1L2	44	high	65	high
Bloemspruit	TP1L2	21	moderate	40	high
Estoire	TP1L2	20	moderate	43	high
Bloemdal 2	TP4L2	12	low	26	moderate

The fall-cone method is not a direct shear measurement, and the formula is logarithmic in nature, which scatters the data when converting from penetration readings to shear strength readings. The COV almost doubles after converting the penetration data to shear strength values. The equation scatters the data and increases the COV to unrealistic high values, introducing higher/more variability. The cone factor may also contribute to the scatter of data in the equation.

4.3.3 Sample disturbance

During the sample preparation for FC testing, the quality of the sample and soil makeup may also influence the test results. Figure 4-4 indicates small stone fragments that were present in one of the samples obtained from Somerton Estate. Stone fragments or even organic matter present can significantly reduce the

penetration readings and influence the cone penetration during testing, resulting in much higher shear strength values.

Figure 4-4: FC sample with stone fragments

Figure 4-5: Disintegrating FC sample

For samples that are silty and/or dry in nature, the sample may not stay intact during testing, as indicated in Figure 4-5. Using a wire cutter to create a smooth finish of the sample may also not always be satisfactory, shearing the sample in the process of cutting, as seen in Figure 4-6.

Figure 4-6: FC sample after cutting

Figure 4-7: Sample shear during sample collection

Figure 4-7 indicates the effect of the sample ring penetrating stiffer/harder soils in-field. Samples that were not suitable for testing were discarded and not included in the results presented.

4.4 Data outliers

Due to the low penetration readings for FC testing and the significant jump in shear strength for these low penetration readings, sample outliers were observed, which has a significant impact on the converted shear strength readings for the fall-cone test. The interquartile range method (IQR) was used to identify outliers (extreme values) for each dataset. This method maintains data integrity that may result from errors and anomalies in data. The IQR is a measure of statistical dispensation and is calculated as the difference between the third quantile (Q3) and the first quantile (Q1) and represents the middle 50% of the data as indicated in the equation below.

Equation 1: $IQR = Q3 - Q1$

The lower and upper outlier thresholds (limits) are then determined using the two equations below (Tukey, 1977).

Equation 2: $Lower\ threshold = Q1 - 1.5 \times IQR$

Equation 3: $Upper\ threshold = Q3 + 1.5 \times IQR$

Any values from the dataset that fall below or above these thresholds were considered outliers and removed from the fall-cone datasets. This technique provides a way to detect outliers, especially in non-normally distributed datasets.

Almost no sample outliers were observed using the pocket penetrometer, with some sites not yielding any sample outliers.

4.5 Fall-cone vs pocket penetrometer

4.5.1 Number of test results

The FC and PP tests produced significantly more test results in a shorter timeframe than the VS and arguably considerably more test results than other shear strength tests (i.e., shear box and triaxial testing), as mentioned in the literature review. The number of test values obtained for the FC is dependent on the number of undisturbed samples collected from the site, sometimes producing less or more test values than the PP. A significant amount of test results could be obtained for each soil sample; however, this is far from the more than 600 test results required for a fully reliable PDF analysis. Table 4-4 indicates the number of tests performed for each sample using the FC and PP.

Table 4-4: The number of PP and FC strength tests performed

Site name	**Sample name**	**Number of tests performed**	
		FC	PP
Tempe	TP1L2	374	40
CUT Agri	TP2 L4	101	190
	TP4L4	148	144
Bloemdal 1	TP4L1	50	133
	TP6L3	61	104
	TP7L2	53	132
Somerton	TP1L2	143	203
Bloemspruit	TP1L2	136	182
Estoire	TP1L2	74	112
Bloemdal 2	TP4L2	23	63

4.5.2 Measured shear strength readings

Table 4-5 presents the average shear strength readings obtained from the various site investigations. The FC test underestimated the shear strength of soils investigated compared to the PP test results for most of the sites. This may be due to the recommended cone factor value (0.8) used in the empirical formula for estimating the undrained shear strength of soils using the FC. For Bloemdal 1 and Somerton, the FC significantly overestimated the shear strength readings, even after removing data outliers, compared to the PP, due to the lower penetrations obtained (<1 mm), as discussed in the previous section.

Table 4-5: The average readings obtained for the PP and FC

Site name	Sample name	Average shear strength (kPa)		Average I (mm)	Cone weight
		FC	PP	FC	FC
Tempe	TP1L2	337	440	1.52	80 g
CUT Agri	TP2 L4	266	471	1.84	80 g
	TP4L4	218	304	2.03	80 g
Bloemdal 1	TP4L1	3207	306	0.57	80 g
	TP6L3	12014	229	0.38	80 g
	TP7L2	22028	475	0.26	80 g
Somerton	TP1L2	1457	937	0.81	80 g
Bloemspruit	TP1L2	324	403	3.31	400 g
Estoire	TP1L2	197	440	4.26	400 g
Bloemdal 2	TP4L2	107	191	5.83	400 g

If the cone factor value is increased to 0.9 or 1.0, the fall-cone shear strength results are more comparable with the pocket penetrometer readings for some of the soils, as indicated in Table 4-6. Using the same cone factor as remoulded testing might not be sufficient for undisturbed testing; however, further research is required in this regard. An increased cone factor might not be universally applicable to all soil samples, indicating the potential need for a variable cone factor tailored to the specific soil type being tested.

Table 4-6: Alternating cone factor values

Site name	Sample name	Average shear strength (kPa)			
		FC			PP
		0.8	0.9	1.0	
Tempe	TP1L2	337	379	422	440
CUT Agri	TP2 L4	266	299	332	471
	TP4L4	218	246	273	304
Bloemspruit	TP1L2	324	364	404	403
Estoire	TP1L2	197	221	246	440
Bloemdal 2	TP4L2	107	120	134	191

4.5.3 Coefficient of variation

Figure 4-8 shows the COV % for shear strength between the FC and PP, side by side, to facilitate comparisons for each site. The COV % for FC testing is significantly higher than the COV % of the PP results, except for Bloemdal 2. This site had a low number of tests with higher penetration values compared to the other sites.

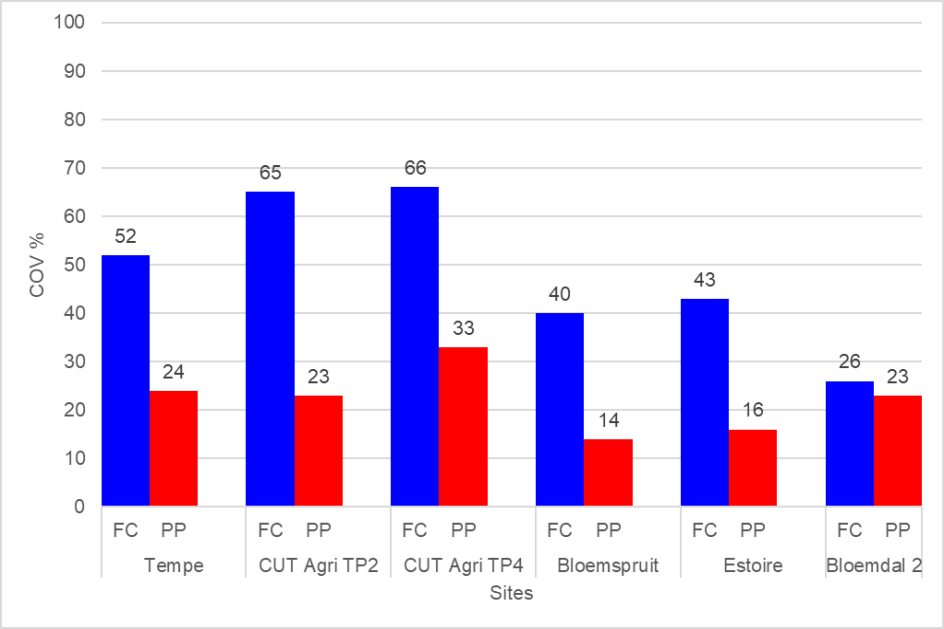


Figure 4-8: COV % for FC and PP shear strength values

The COV values for the FC shear strength range from 26% to 66% and are higher than the COV for undrained shear strength reported in literature by Bond and

Harris (2008). For the PP, the COV values range from 14% to 33%. The COV is quite high for soil layers that appear to be homogeneous.

Table 4-7 lists the COVs along with the degree of variability according to Harr (1987), for FC and PP testing.

Table 4-7: Coefficient of variation for FC and PP shear strength readings

site name	sample name	FC (kPa) COV %	variability degree	PP (kPa) COV %	variability degree
Tempe	TP1L2	52	high	24	moderate
CUT Agri	TP2 L4	65	high	23	moderate
	TP4L4	66	high	33	high
Bloemspruit	TP1L2	40	high	14	low
Estoire	TP1L2	43	high	16	moderate
Bloemdal 2	TP4L2	26	moderate	23	moderate

The PP indicates that all the site samples are relatively variable in nature, except for Bloemspruit that has a low degree of variability. The FC indicates that most of the site samples are highly variably in nature, but this is due to the conversion of penetration data to shear strength values as discussed in the previous sections.

4.5.4 Probability density functions

Figures 4-9 to 4-26 illustrate the histograms and measured probability density functions along with the lognormal distribution fit for each soil sample using MATLAB software. The combined PDFs between the two testing methods used are also illustrated for comparison. For all the FC samples, the PDFs are skewed to the left.

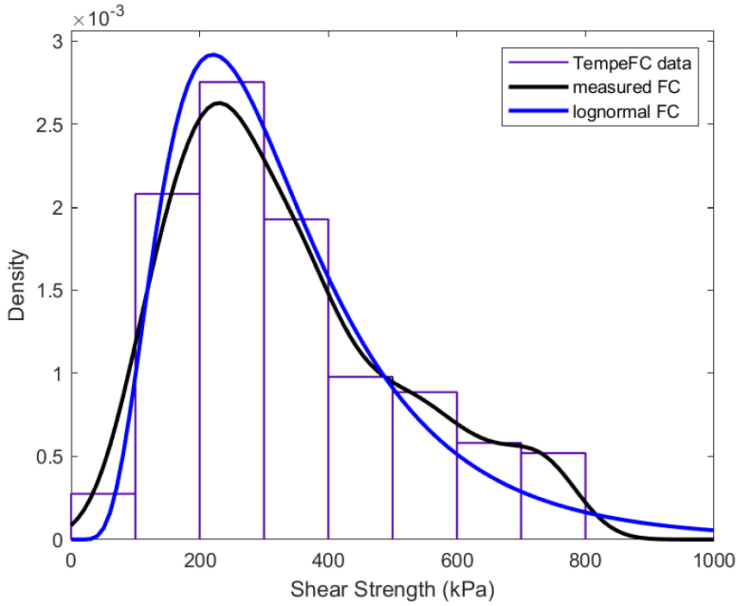


Figure 4-9: Tempe FC PDF

The FC PDF for the Tempe site shows a broader curve with significant skewness, indicating variability in the penetration readings converted to shear strength.

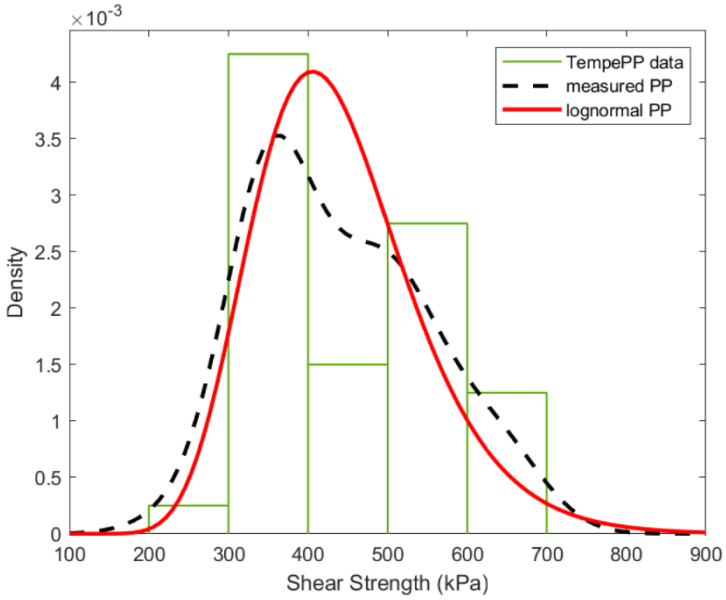


Figure 4-10: Tempe PP PDF

The PP PDF is narrower, suggesting more consistent readings with lower variability.

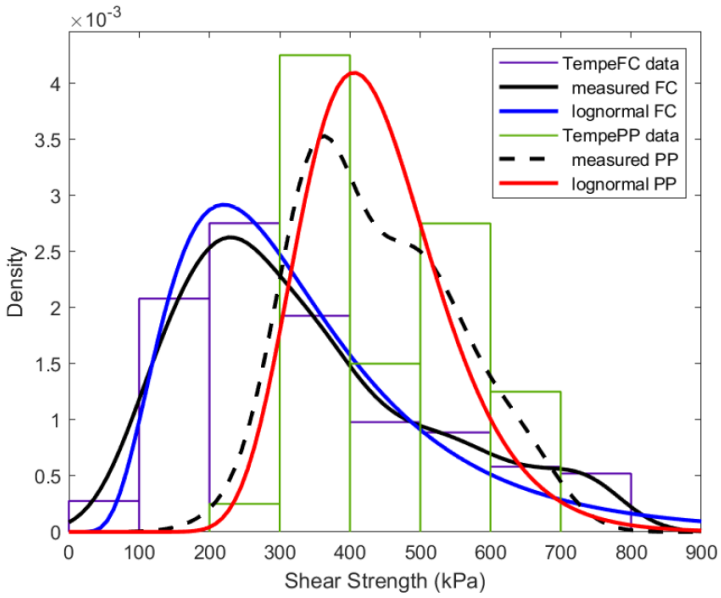


Figure 4-11: Tempe combined PDF

When compared, the FC curve is positively skewed and has a lower peak, while the PP data indicate a more concentrated distribution around a central value.

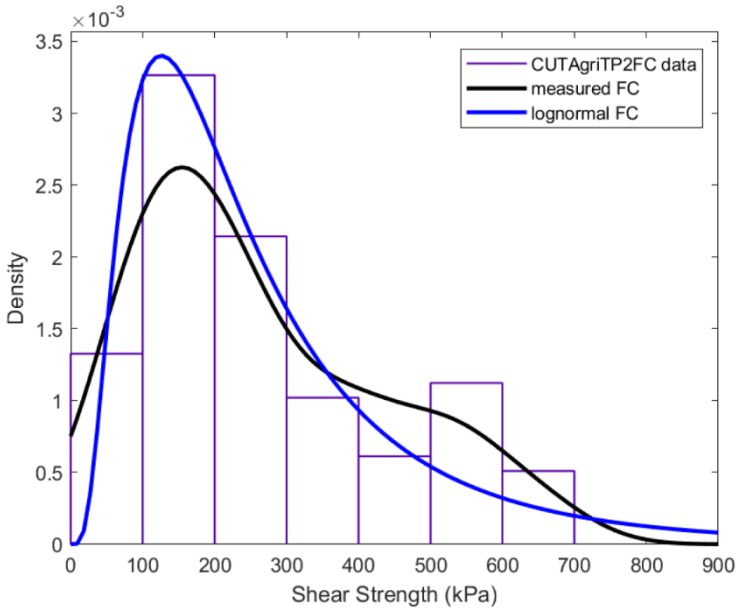


Figure 4-12: CUTAgri TP2 FC PDF

The FC PDF displays a wider spread, representing greater variability in shear strength values.

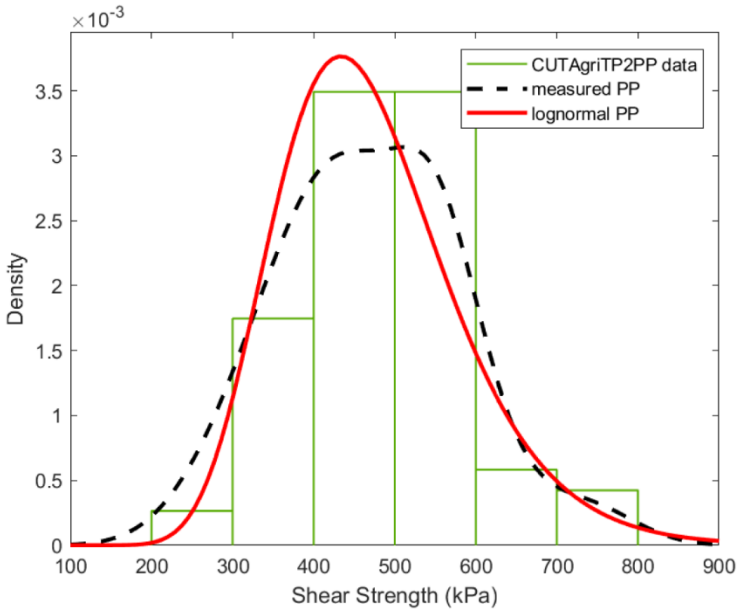


Figure 4-13: CUTAgri TP2 PP PDF

The PP PDF indicates a narrower spread than the FC, indicating more uniform results.

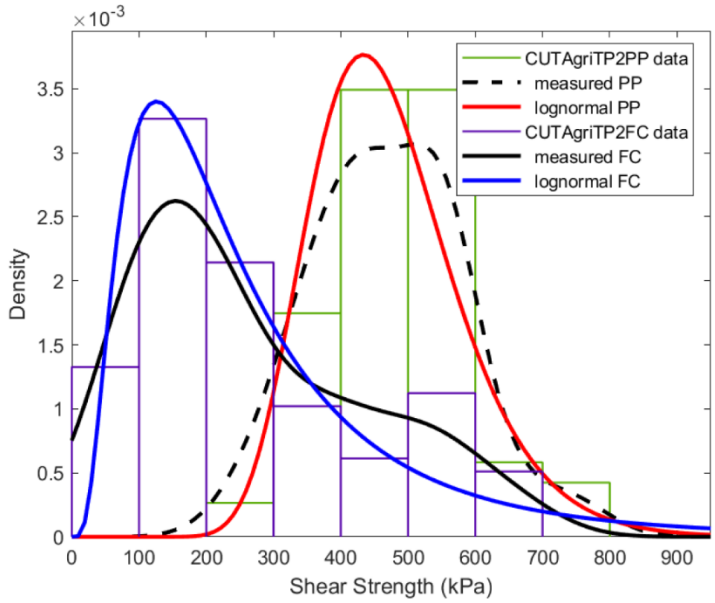


Figure 4-14: CUTAgri TP2 combined PDF

Figure 4-14 indicates a clear divergence between the two methods, with FC skewed to the left, emphasising methodological differences.

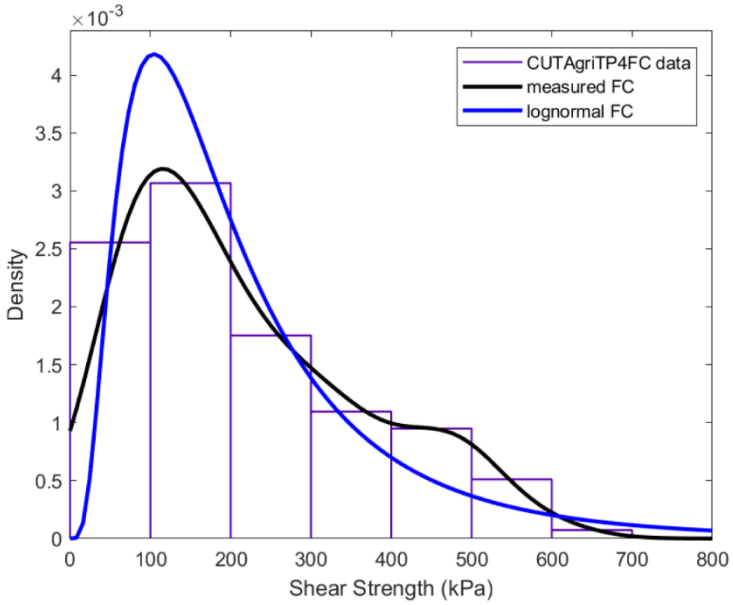


Figure 4-15: CUTAgri TP4 FC PDF

The FC PDF for this site is consistent with other FC graphs, wide and positively skewed.

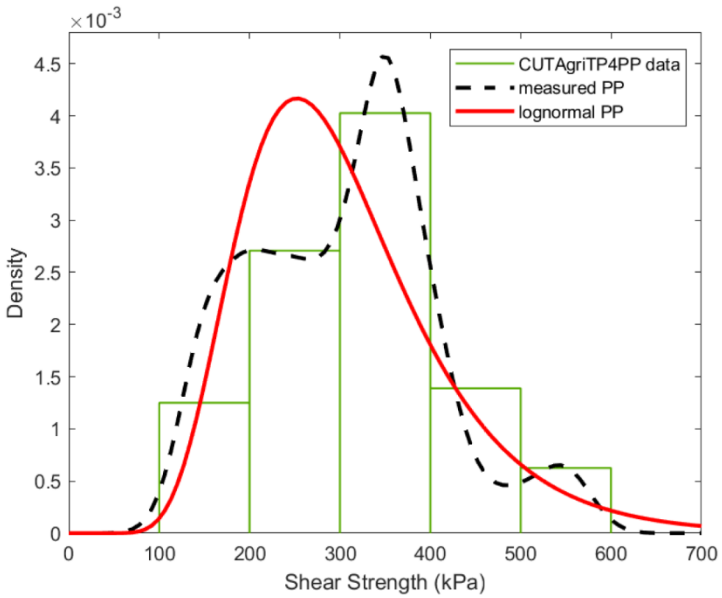


Figure 4-16: CUTAgri TP4 PP PDF

The PP PDF is narrower with a pronounced peak, suggesting lower variability in PP results.

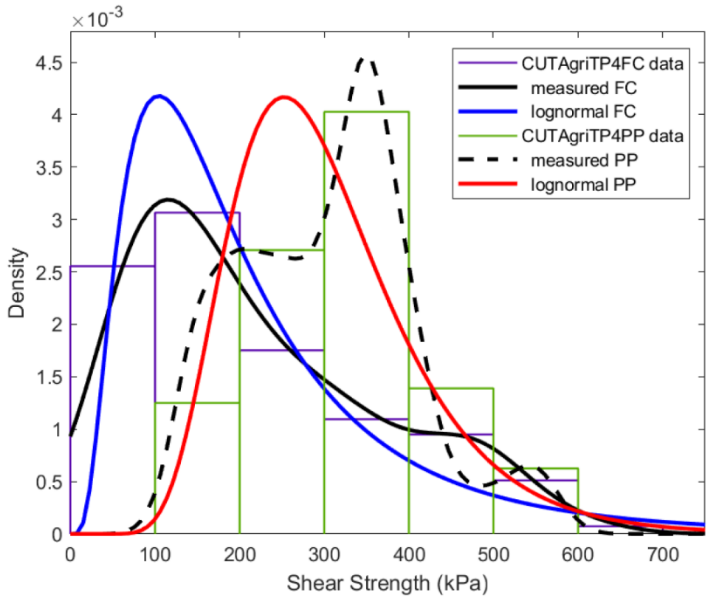


Figure 4-17: CUTAgri TP4 combined PDF

The combined PDFs highlight the limitations of FC in achieving concentrated distributions compared to PP.

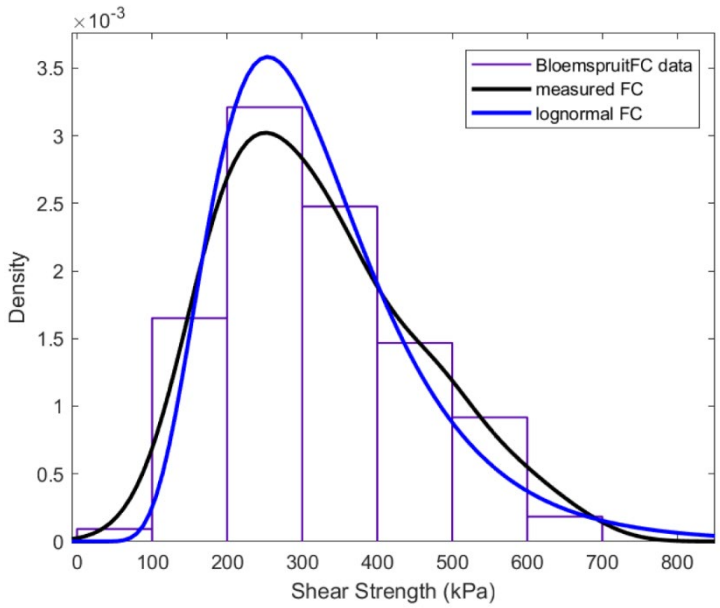


Figure 4-18: Bloemspruit FC PDF

The FC PDF for Bloemspruit is broad, with a smaller spread compared to CUTAgri, but still positively skewed.

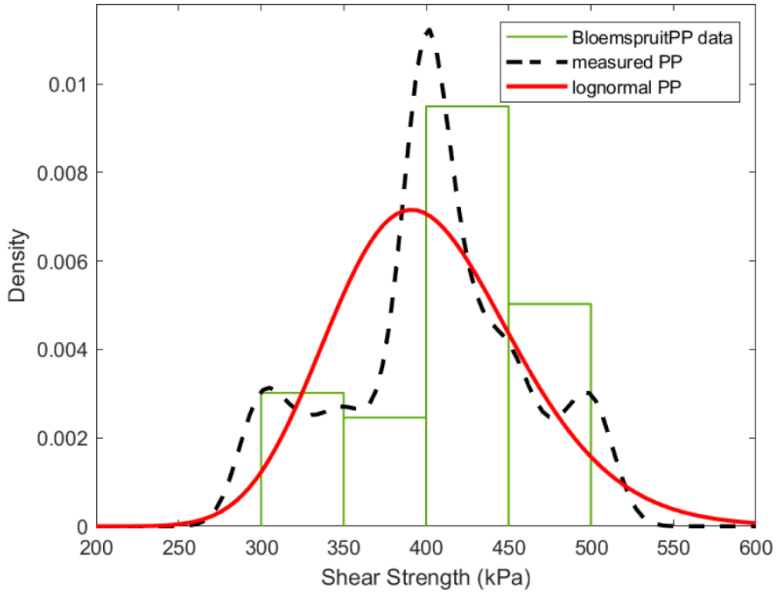


Figure 4-19: Bloemspruit PP PDF

The PP PDF is narrow and high-peaked, implying greater consistency.

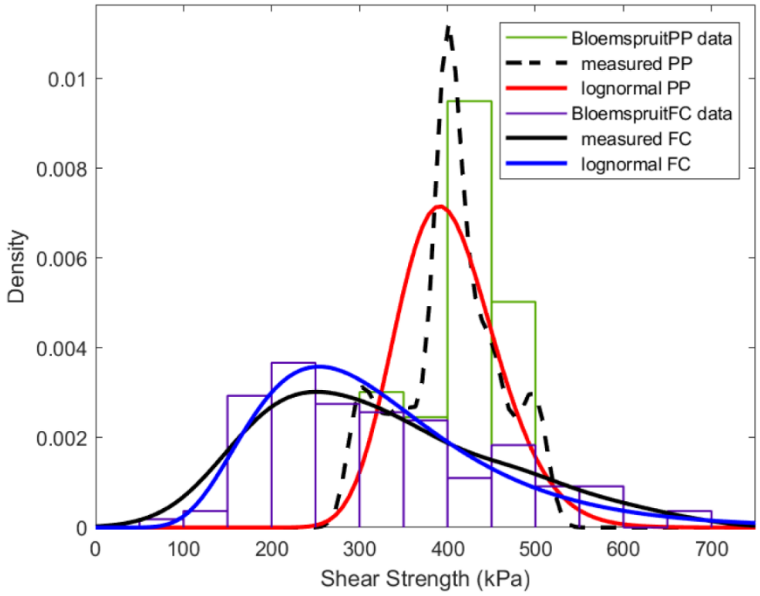


Figure 4-20: Bloemspruit combined PDF

Figure 4-20 illustrates the same trend of variability contrast between FC and PP.

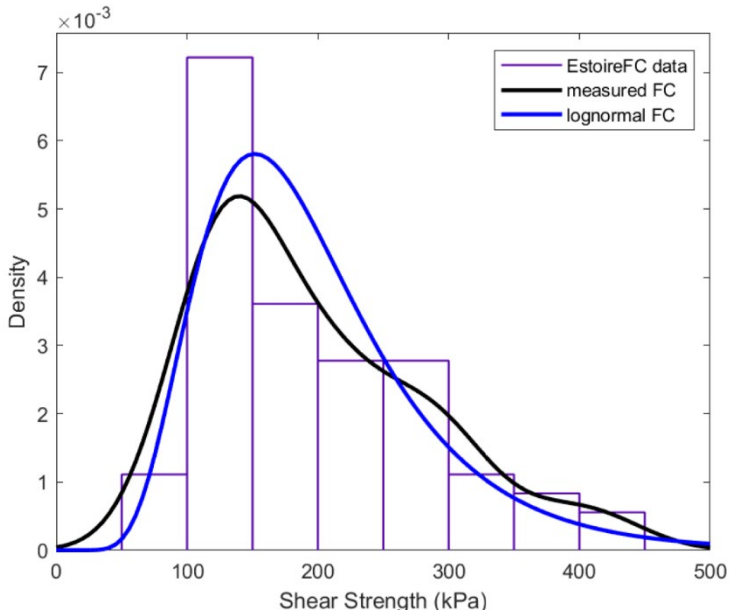


Figure 4-21: Estoire FC PDF

The FC PDF displays high variability with a broad and skewed distribution.

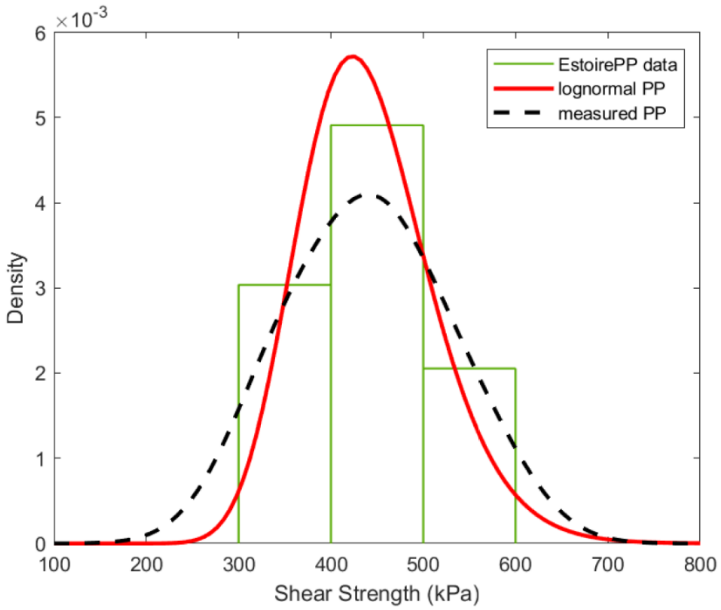


Figure 4-22: Estoire PP PDF

The PP PDF for Estoire demonstrates narrow and tightly grouped data points.

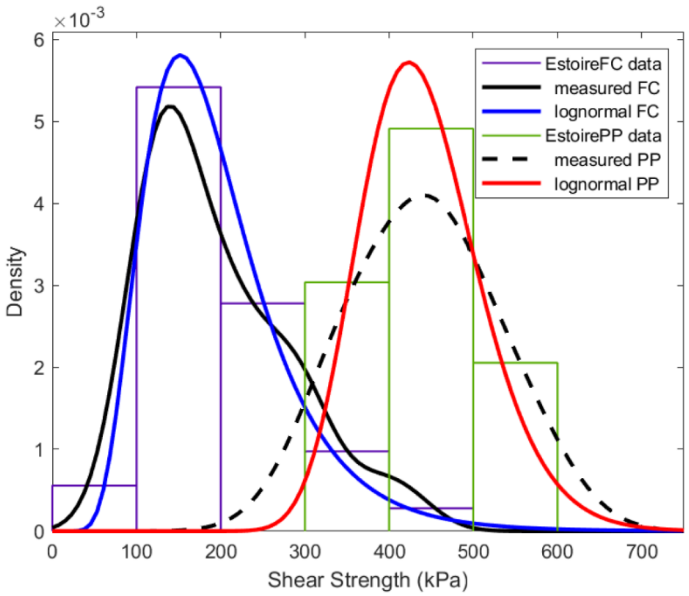


Figure 4-23: Estoire combined PDF

The combined PDFs consistently reinforce the contrast between the methods, with PP showing more reliable distributions.

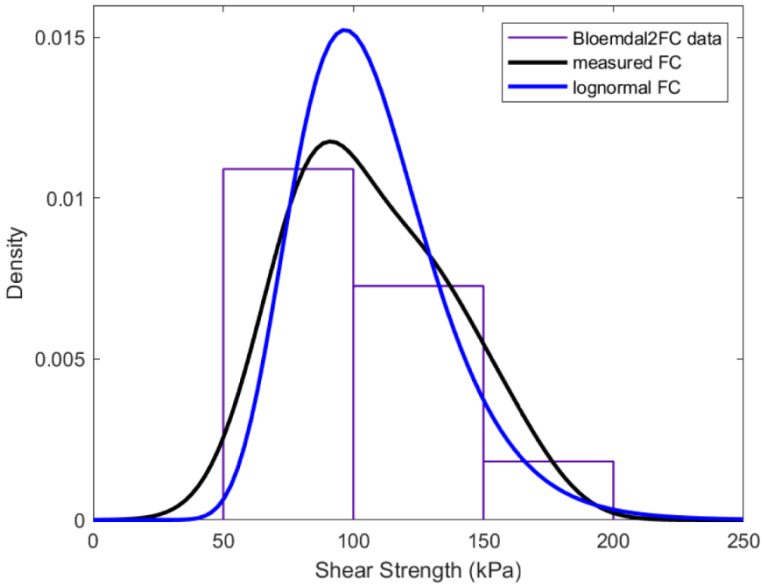


Figure 4-24: Bloemdal 2 FC PDF

The FC PDF for Bloemdal 2 is the narrowest among the FC graphs, possibly due to fewer test samples.

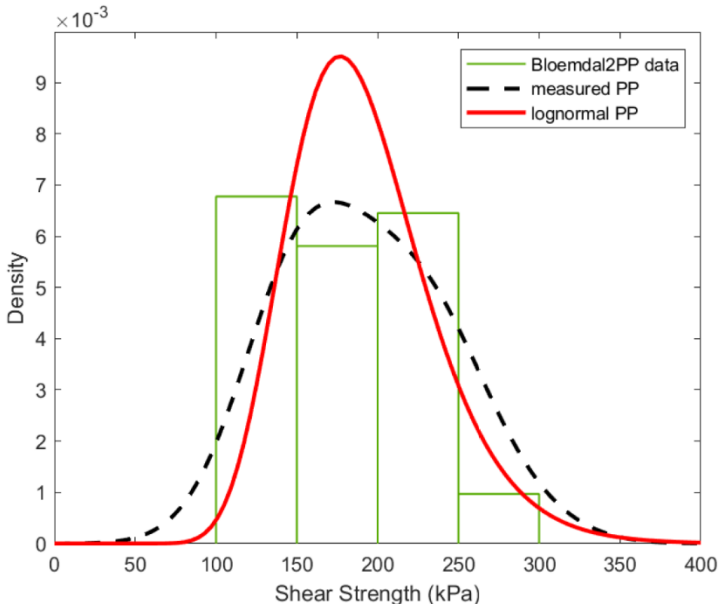


Figure 4-25: Bloemdal 2 PP PDF

The PP PDF follows a similar trend of narrower distributions.

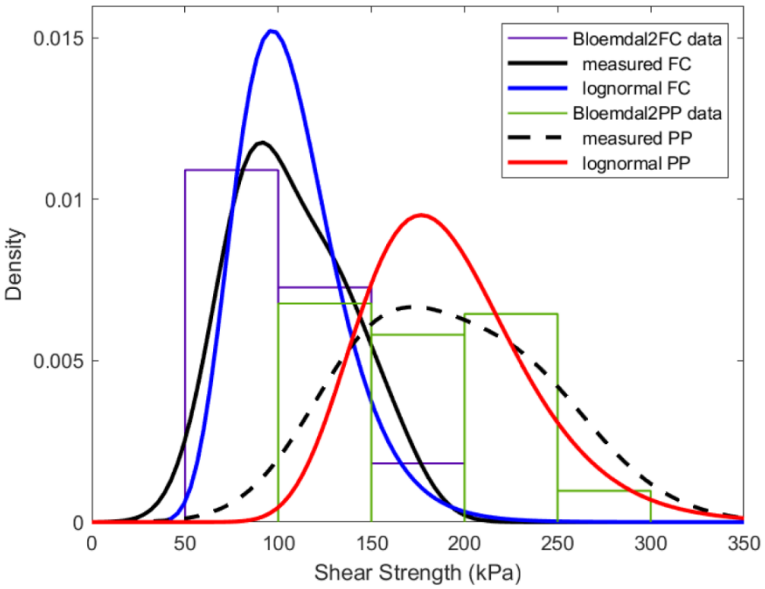


Figure 4-26: Bloemdal 2 combined PDF

For this site, the FC variability appears closer to PP, potentially due to data size constraints.

From Figures 4-19 to 4-26, the following key observations are made:

1. **Variability differences:** The FC PDFs consistently show broader distributions with positive skewness, while the PP PDFs are narrower, indicating more concentrated results.
2. **Skewness:** The FC skewness reflects methodological challenges in converting penetration depth to shear strength, as discussed in the dissertation. An increase in the cone factor could reduce the skewness.
3. **Site-specific Insights:** Bloemdal 2 shows closer alignment between the FC and PP due to fewer data points.
4. **Combined PDFs:** Across sites, combined PDFs highlight methodological divergence, with FC generally underestimating compared to PP.

The 5th percentile is critical in geotechnical design because it represents a conservative estimate of soil strength, ensuring safety under worst-case conditions. Insufficient or poor 5^{th} percentile values implies that the lower bound of expected strength is very low. This can lead to overly conservative designs, increased foundation costs, or the need for ground improvement. Moreover, if the FC method underestimates this percentile compared to the PP, it raises concerns about methodological reliability. Such divergence may influence design decisions and highlights the importance of selecting appropriate and validated test methods.

4.5.5 Box plots

The box and whisker diagrams for both the FC and PP tests across the various sites are illustrated in Figure 4-27. The box itself represents the IQR, its length reflects the spread of the data. The whiskers, or long thin lines, indicate the rest of the data and extend from the first and third quantiles to the minimum and maximum values, respectively. The box shows the data spread of the central portion of the data. A longer box indicates greater variability, while a shorter box indicates less variability. A whisker that is noticeably longer than the other suggests that the data is skewed in that direction. The median values are marked with an ‘X’ on the graph.

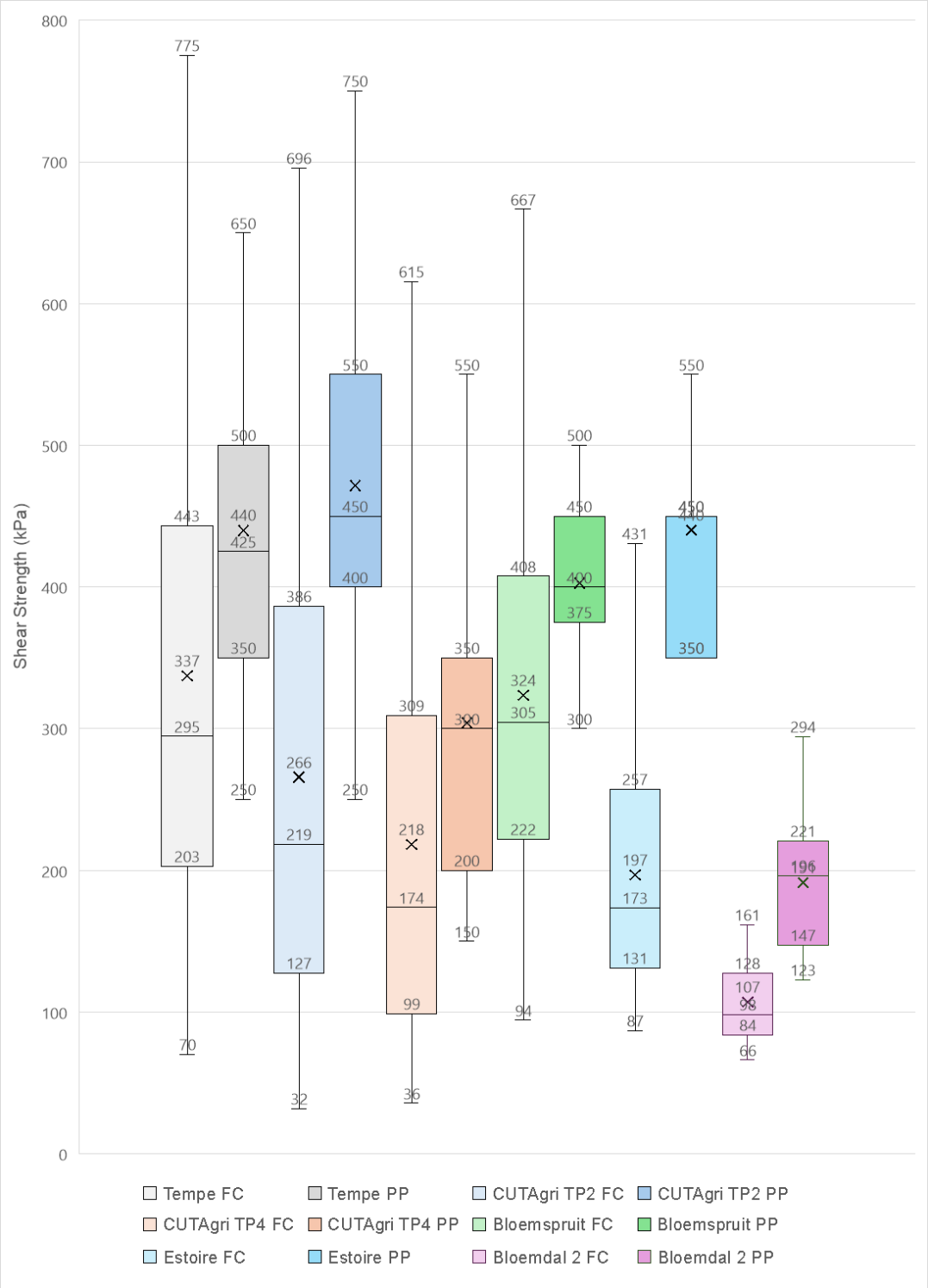


Figure 4-27: Comparative box and whisker diagram

The centre of distribution for all the FC results is lower than the centre of distribution for the PP results. The FC data also indicates a wider spread and range in the data compared to the PP results, except for Bloemdal 2, where the FC data has a smaller spread compared to the PP. For this site, the number of FC values obtained was

very low (23 data points), and the COV was relatively close to the PP COV. This might be due to the higher penetration reading (>4 mm) compared to all the other sites, which had penetration readings less than 4 mm. As discussed in the previous section, 0.1 mm of a penetration reading can have a significant impact on the shear strength readings. All of the distributions are significantly skewed, either positively or negatively, except for Tempe PP, which appears almost symmetrical but slightly skewed to the left.

4.5.6 Violin plots

Figures 4-29 to 4-34 show the FC and PP violin plots for each site. A violin plot is a combination of box plot and kernel density plot. It indicates the median and interquartile range along with peaks in the data and the density of the data. A white dot represents the median and the thick bar represents the interquartile range. A thin line represents the rest of the data set, including the maximum and minimum, except for the outliers. As mentioned in the previous section data outliers were identified and removed using the interquartile range method. On either side of the thin line is a kernel density curve that indicates the width and the shape of the data. Wider sections indicate a higher concentration/frequency of the data. A narrower section or width indicates a lower concentration of data. Figure 4-28 illustrates the anatomy of a violin plot.

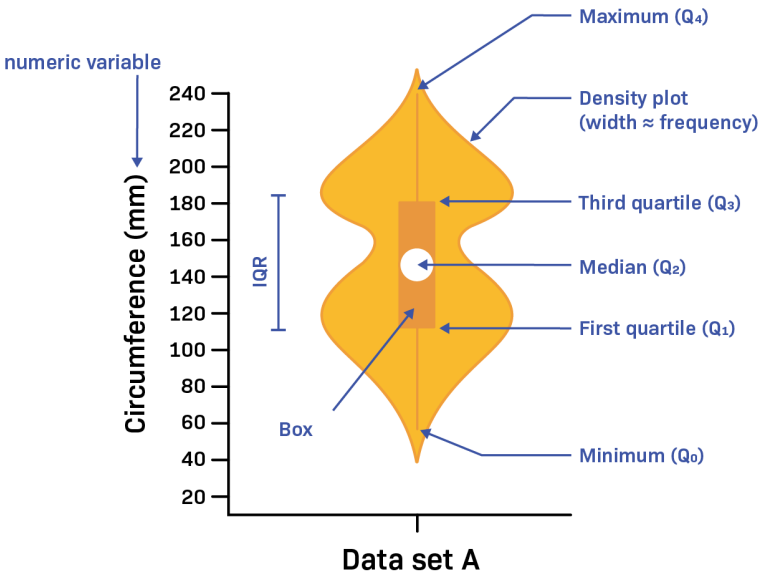


Figure 4-28: Anatomy of a violin plot (Harvard Online, 2021)

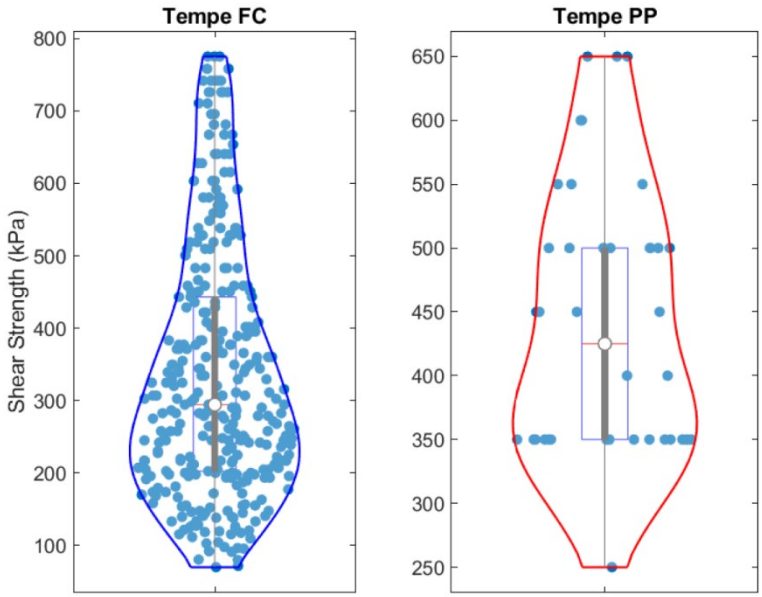


Figure 4-29: Tempe violin plot

For both testing methods, the widest section of the plot is around the first quantile. The FC plot has a narrow tail that stretches from the third quantile to the maximum; in contrast, the PP plot has a wider tail.

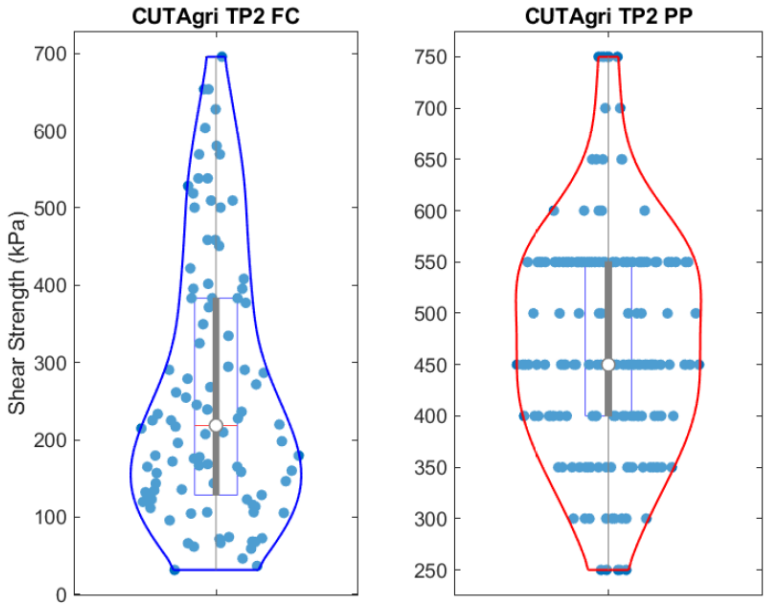


Figure 4-30: CUTAgri TP2 violin plot

The FC plot is again wider around the first quantile, with a narrow tail. The PP plot is widest at the IQR, between the first and third quantiles.

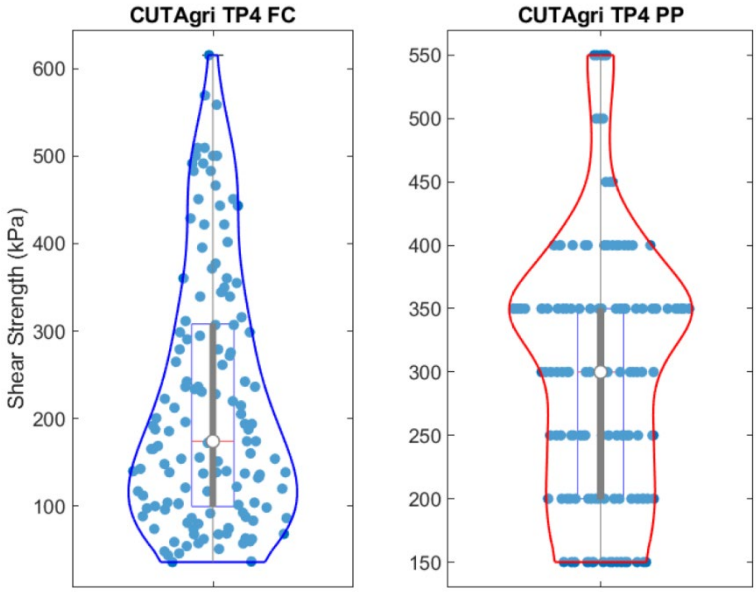


Figure 4-31: CUTAgri TP2 violin plot

The trend continues for the FC plot, with a higher density of data around the first quantile. The narrow tail indicates positive skewness in the data. The PP plot shows a higher density of data around the third quantile, indicating negative skewness.

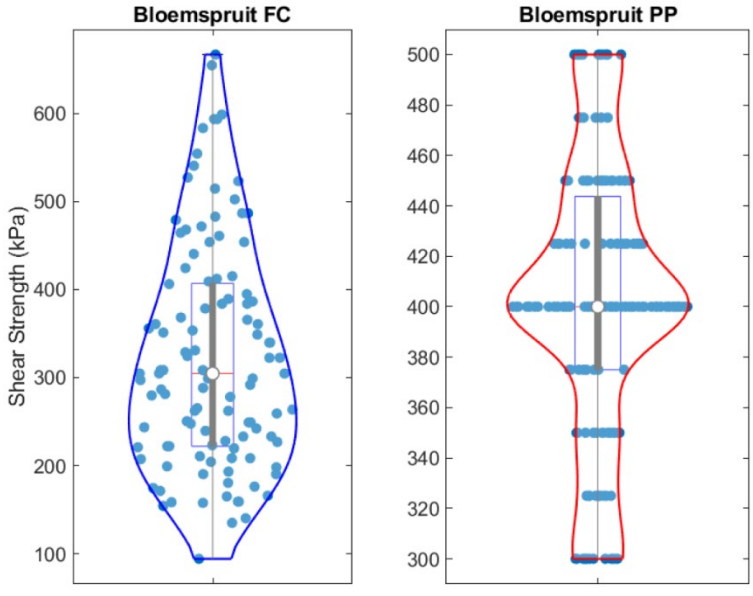


Figure 4-32: Bloemspruit box and violin plot

For Bloemspruit, the data is again more concentrated around the first quantile, with a tail extending to the maximum. The PP plot is widest around the median, with two

tails extending from the first and third quantile to the minimum and maximum, respectively.

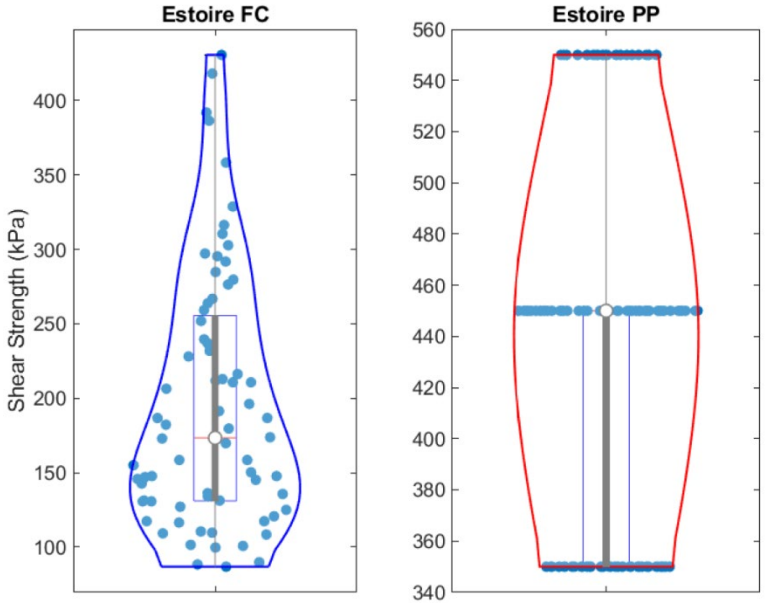


Figure 4-33: Estoire box and violin plot

A similar trend for the FC plot is evident for this site. The PP plot appears to be symmetrical but indicates extreme skewness, with the minimum being equal to the lower quantile. Data points for the PP plot are aligned only at 350, 450, and 550 for this sample because the device measures soil strength in discrete, predefined increments. This results in clustered readings rather than a continuous range, reflecting the instrument's calibration steps rather than subtle variations in soil resistance, which can also depend on the size of the penetration head used.

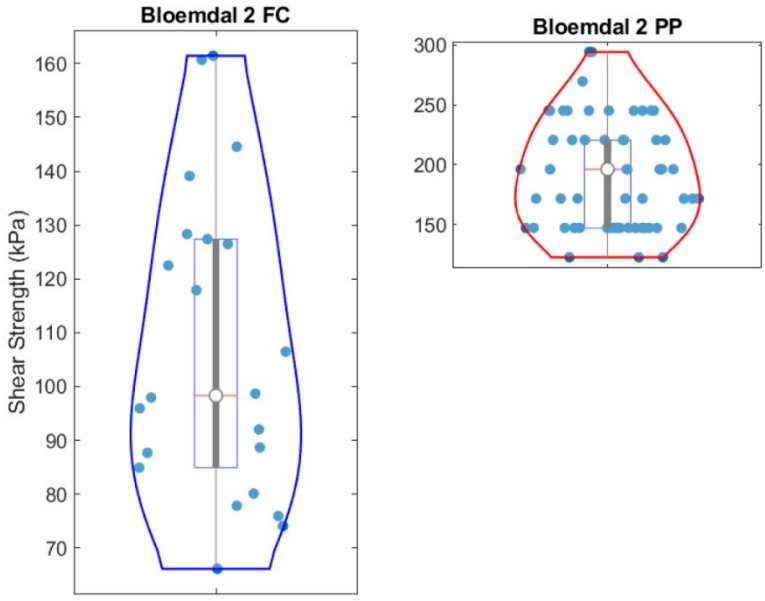


Figure 4-34: Bloemdal 2 box and violin plot

For Bloemdal 2, the FC plot is widest between the median and first quantile, indicating positive skewness. A wider tail is evident compared to the other sites. The PP plot is also widest between the median and first quantile. The median sits towards the top, and the upper quantile is smaller than the lower quantile, indicating negative skewness in the data.

The FC plots follow the same trend across all sites, with the highest concentration of data leaning towards the first quantile, indicating positive skewness. The PP plots, however, do not show any trend or similarities between the sites.

4.5.7 Lognormal probability plots

Using MATLAB, the probability plot, also known as a probability-probability plot, was used to verify whether the test data for the various sites fit a lognormal distribution, as stipulated by Eurocodes. Points that follow the theoretical distribution will lie on the diagonal dashed line. Points that lie above the diagonal dash line indicate discrepancies, such as skewness and heavy tails, or a departure from the theoretical distribution. Figures 4-35 to 4-46 show the probability plot for each site using the FC and PP.

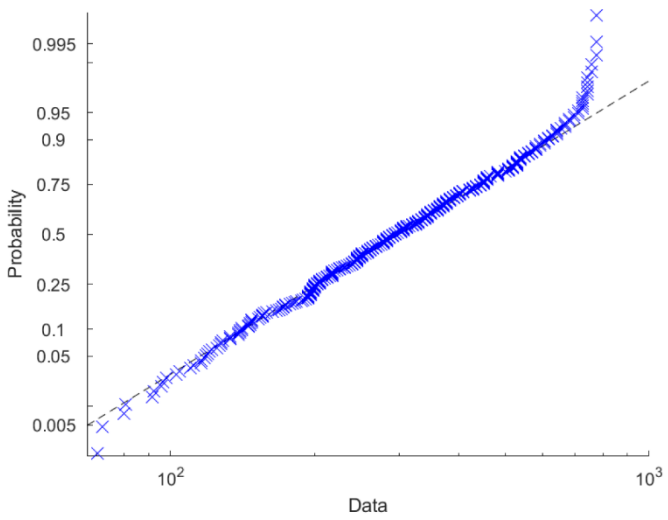


Figure 4-35: Tempe FC probability plot

The curve starts just below the lognormal line and follows it, then bends above the line towards the end, indicating higher variation or a departure from what is expected for a lognormal distribution.

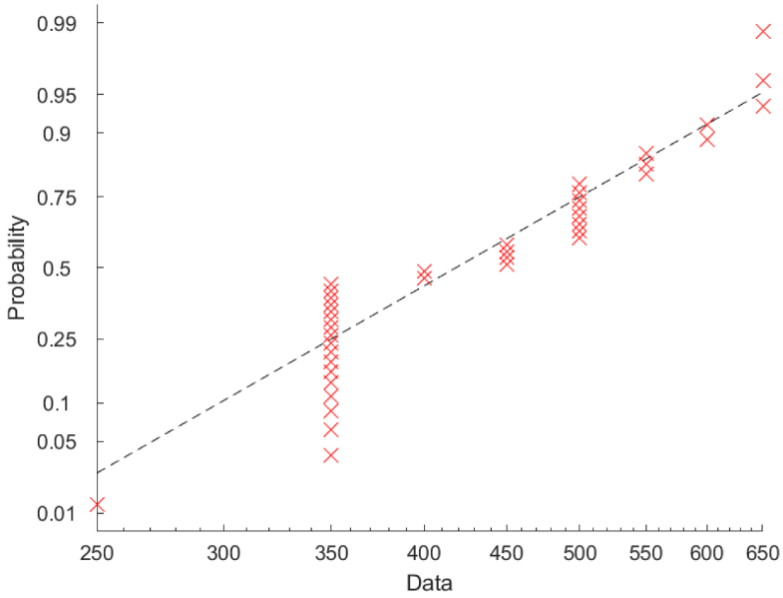


Figure 4-36: Tempe PP probability plot

The data points do not fit for a lognormal distribution with an undefined curve.

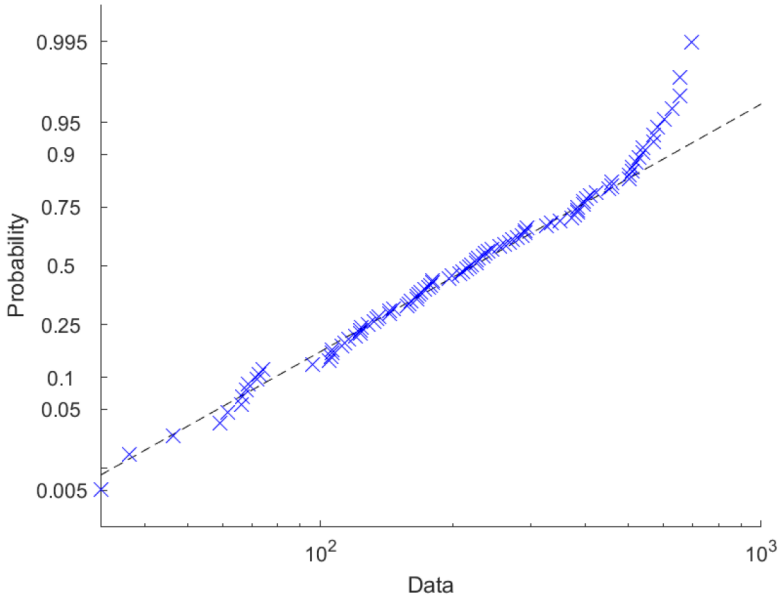


Figure 4-37: CUTAgri TP2 FC probability plot

The curve follows the same trend as the previous sample, starting just below the lognormal line and bending upwards towards the end.

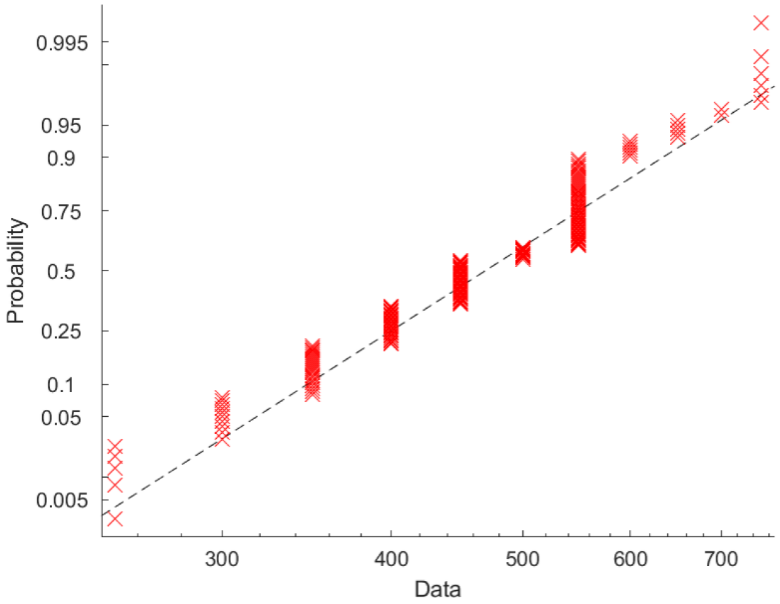


Figure 4-38: CUTAgri TP2 PP probability plot

The plotted points appear to be mostly to the left of the line, with a bend at both the start and end. This suggests a longer tail and skewness in the data.

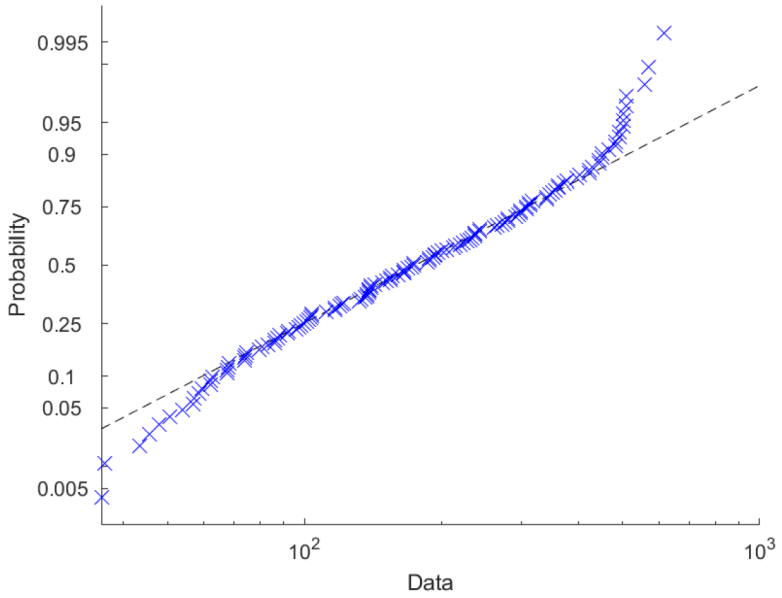


Figure 4-39: CUTAgri TP4 FC probability plot

The points start below the line again and follow it, with an upwards bend at the end, indicating a departure from a lognormal distribution with longer tails.

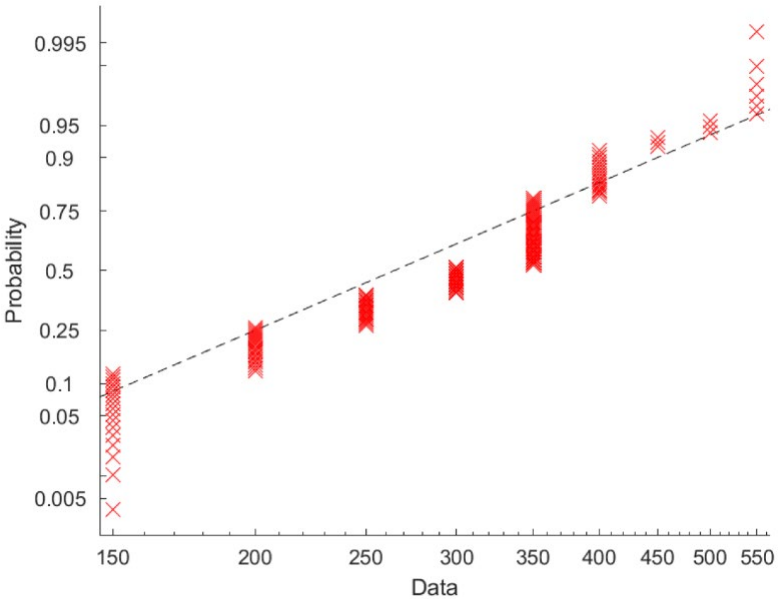


Figure 4-40: CUTAgri TP4 PP probability plot

The curve starts below the line, slightly touches the lognormal line, and then bends below the line curving upwards again at the end, suggesting long tails.

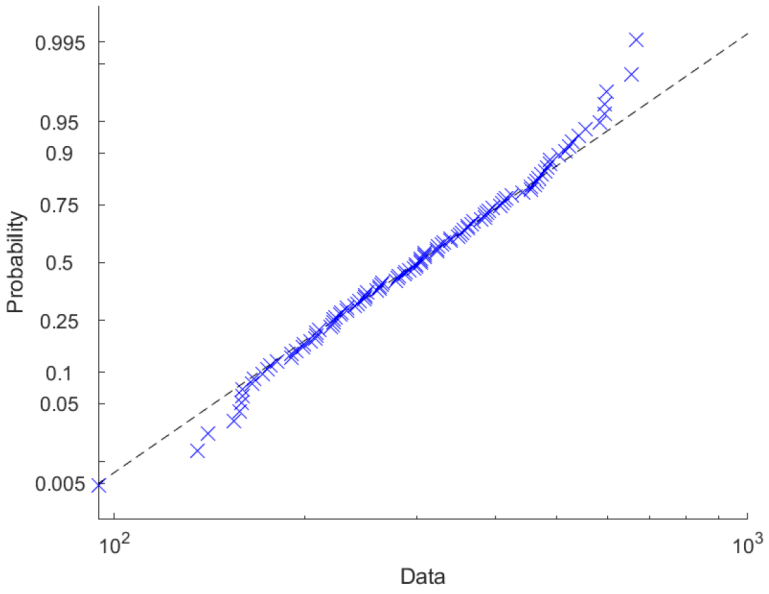


Figure 4-41: Bloemspruit FC probability plot

For this site the FC follows the same trend, indicating higher-than-expected variation for a lognormal distribution.

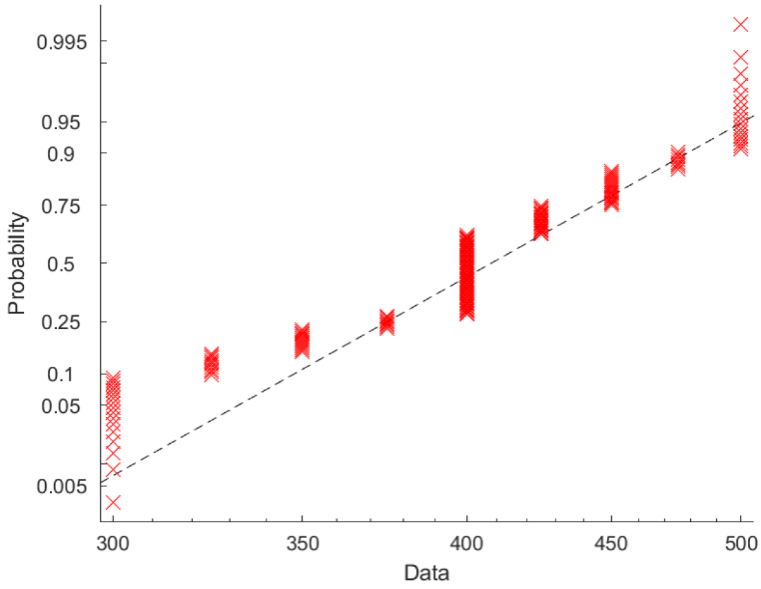


Figure 4-42: Bloemspruit PP probability plot

The curve, or layout of points, appears to bend up and to the left of the line at both the start and end, indicating skewness in the data.

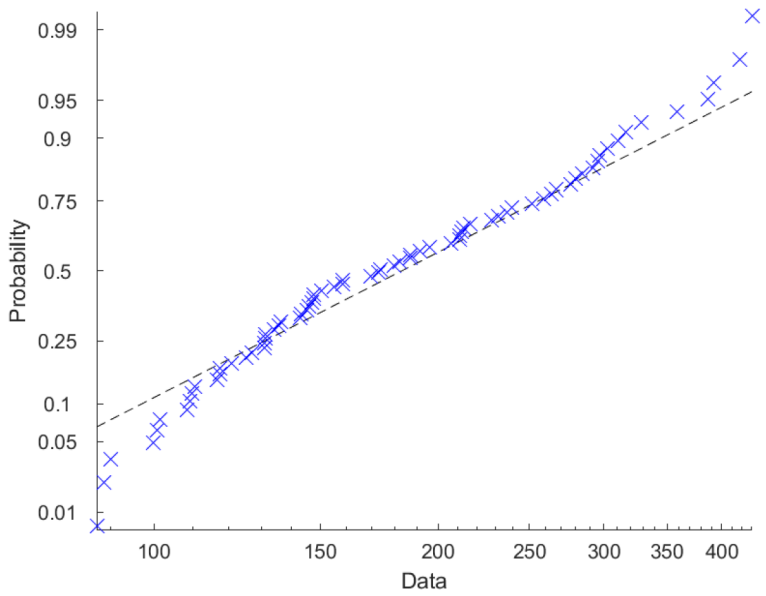


Figure 4-43: Estoire FC probability plot

The FC points for Estoire follow the same trend and pattern as the previous FC plots.

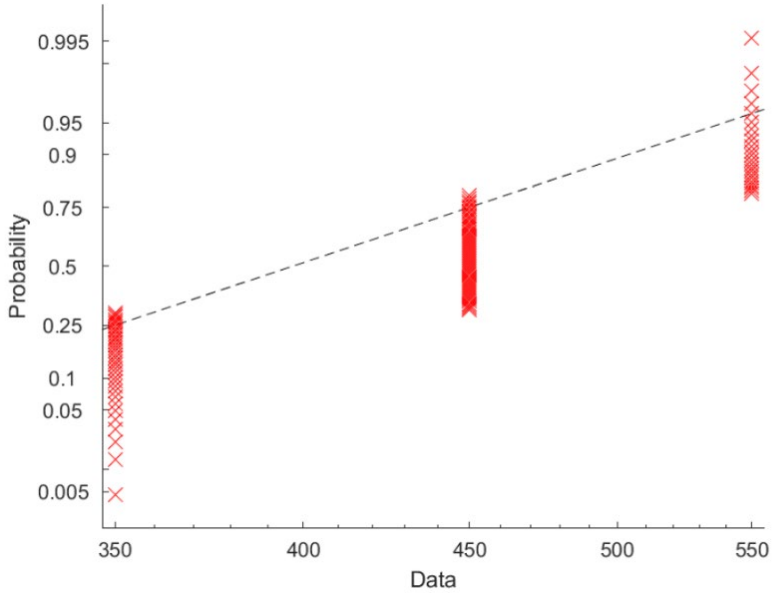


Figure 4-44: Estoire PP probability plot

Most of the points start below the lognormal line and end below it, suggesting skewness in the data.

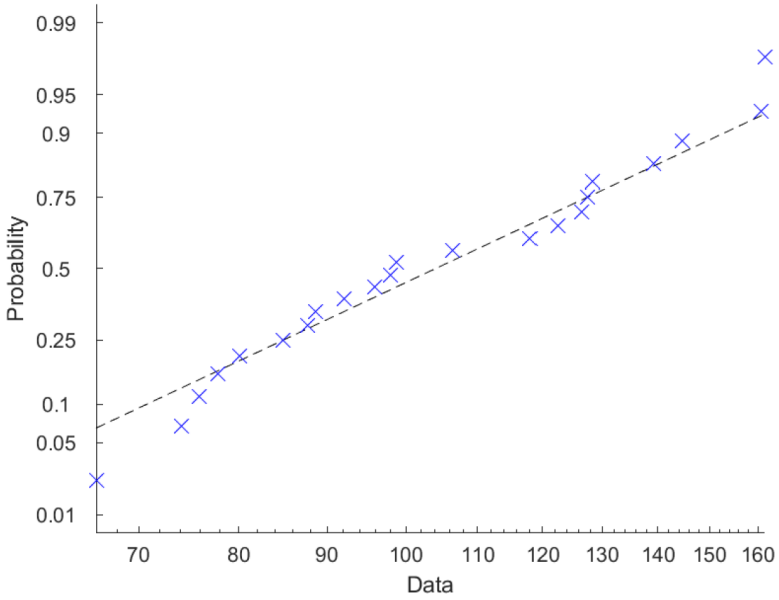


Figure 4-45: Bloemdal 2 FC probability plot

The majority of the points lie on the lognormal line, with only four points deviating from the lognormal distribution, suggesting a better fit to the lognormal distribution compared to the other sites. This may be due to fewer testing points.

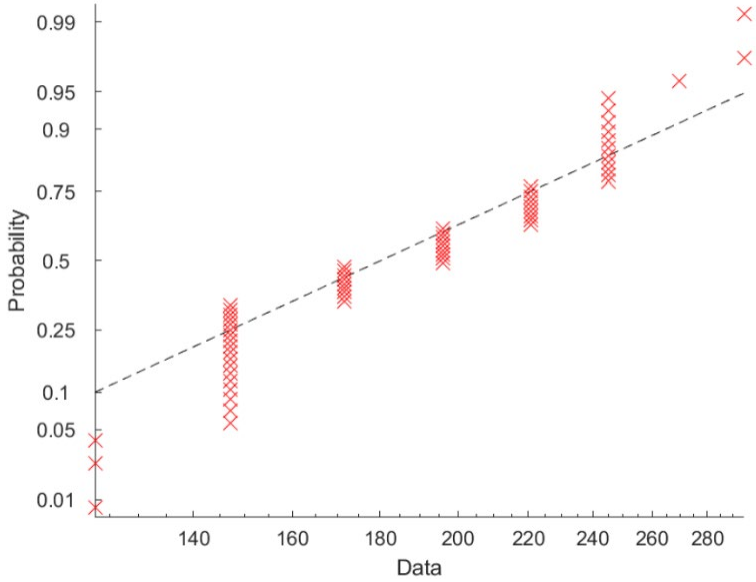


Figure 4-46: Bloemdal 2 PP probability plot

The curve starts below the line and slightly follows it, with and upward bend towards the end, indicating higher-than-expected variation for a lognormal distribution.

Figure 4-47 indicates the overall probability trend for the FC plots.

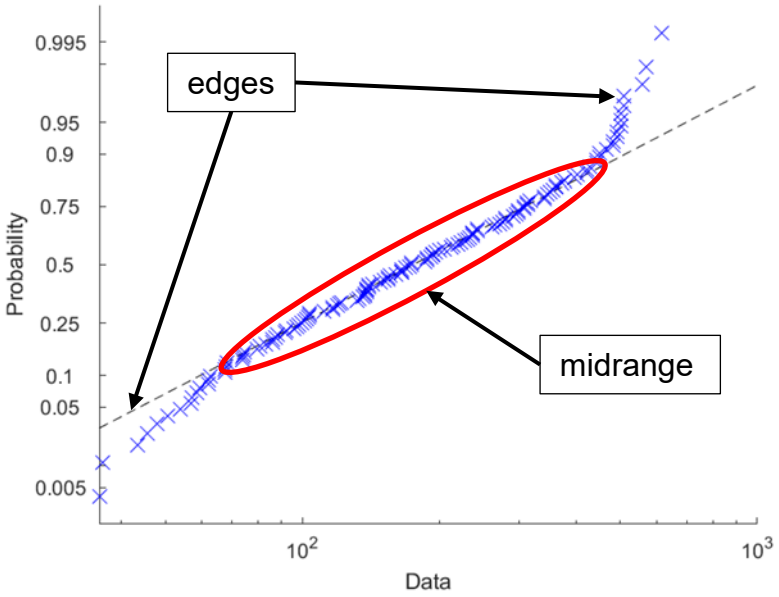


Figure 4-47: FC probability trend

From Figures 4-35 to 4-46, it is evident that most of the samples show a relatively good fit between the dataset and the theoretical lognormal distribution for FC testing. The FC probability plots suggest that, at values close to the midrange, the curves are similar to lognormal, but away from the midrange, the curves resemble a lognormal distribution. However, away from the midrange towards the outer edges, the values deviate from lognormality, as shown in Figure 4-47. Thus, the 5% and 95% fractile values of the curves are not likely to be meaningful. For PP testing, the data is not a good fit for a lognormal distribution.

4.6 Summary of results and findings

Different tools and techniques were used to visualise and analyse the data for the various samples tested using the FC and PP.

Converting penetration readings to shear strength values scatters the data points and introduces higher variation. The PDFs indicate that the measured FC data is a relatively good fit to a lognormal curve. Violin plots show that the highest concentration of points for FC testing lies close to the first quantile, indicating skewness in that direction. The probability plots also suggest that the FC data is a good fit for a lognormal distribution, with small nonconformities at the outer edges.

FC testing produces higher COVs, a greater range (maximum - minimum), and a wider IQR, suggesting higher variability.

The PP PDFs indicate that the measured PP data is not always a good fit for a lognormal distribution. The violin plots show no similarities or patterns between the samples tested, suggesting variability between the samples tested. Probability plots for this test method indicate that the PP is not a good fit for a lognormal distribution. The PP produces a smaller range and IQR compared to the FC indicating more concentrated values around the midpoint and less variation in testing.

CHAPTER 5: CONCLUSION

5.1 Introduction

The previous chapter covered the data analysis and the results of the research study. The challenges of the VS were addressed. The undrained shear strength readings from the FC and PP were analysed and evaluated. In this chapter, the findings of the previous chapters will be reviewed, the research question and objectives will be addressed, and final recommendations will be provided.

5.2 Reflection of the previous Chapters

Chapter 1: The background and research problem highlighted the need to obtain reliable shear strength readings using cost-effective tools for design input.

The main aim, along with the four objectives, were stated and are addressed in the following sections of Chapter 5. A brief methodology section includes a basic flow diagram outlining the research project's methodology. The contribution and benefits of the study provide students, researchers, and industry professionals with a better understanding of obtaining large datasets to effectively account for soil variability when determining the characteristic value for design inputs in a cost-effective and fast way.

Chapter 2: A thorough review of relevant published works and literature was conducted to identify suitable testing procedures for measuring the undrained shear strength of cohesive soils for data analysis. Based on past research, availability, and cost considerations, three testing procedures were selected.

A review of the most common shear strength tests revealed that each test has its own limitations and shortcomings. Most testing procedures are specifically designed and standardised for the outdated WLD approach, which requires an average or representative value derived from no more than three test values.

A review of the newly introduced SANS 10160-5 standard, based on the LSD approach, reveals that a large quantity of test values is required to achieve a 95% confidence level in the test data used for design inputs. This may be unattainable for the majority of available testing procedures. However, the use of cheaper and faster alternatives could help overcome this challenge.

Chapter 3: In this chapter, the research areas were introduced using a Google Earth snippet. The in-field sampling and testing procedures used across all sites were outlined, along with sample preparation process for the FC testing. The components of FC apparatus were listed. The testing procedure and applicable standard and equation used was also specified.

Chapter 4: The challenges associated with the VS and FC were presented. Data analysis and test results for the FC and PP were discussed in this chapter, utilising COVs, violin plots, PDFs, and probability plots for each soil tested. The impact of the low penetration readings and the chosen cone factor value was also discussed.

5.3 Research aim

The research aim, as stated in Chapter 1, can now be answered as follows:

- Three testing methods were analysed and evaluated to identify the best method to account for inherent soil variability. VS testing proved to be challenging and impractical. The FC formula introduces more variation and scatter in the data when using the empirical formula and is more suitable for LL testing rather than undisturbed soil testing. The PP provides consistent results and indicates the degree of soil variability in a quick and economical way.

The majority of the FC probability plots indicate that the data fits a lognormal distribution around the midrange, but values farther out from the midrange, towards the edges, do not. This may not only be due to inherent sample variation but also to the empirical formula, which is logarithmic. The FC test produced significantly higher COVs than the PP when converting penetration values to shear strength values.

None of the PP PDFs presented in Chapter 4 were a good fit to lognormal curves. This may be attributed the testing application, which yields more consistent values with a smaller range and may be better suited for a normal distribution.

5.4 Conclusion

The research objectives stated in Chapter 1 can now be addressed:

5.4.1 Objective (a)

This study aimed to evaluate shear strength PDFs of undisturbed soil samples within the framework of the newly introduced LSD concept in South Africa. A thorough literature review highlighted the shortcomings of the most common shear strength tests currently in use. Most shear strength testing standards cater specifically for the WLD design concept. The LSD procedure can be reasonably applied to obtain design values, provided there is high confidence in the testing results.

5.4.2 Objective (b)

The PP and FC tests demonstrated the ability to generate large volumes of data efficiently and economically, satisfying the requirements for reliable PDF construction. The PP, in particular, yielded consistent results across a range of cohesive soils and effectively captured the inherent variability of the soils within MMM. The FC also provided extensive datasets, but interpretation was complicated by its sensitivity to testing conditions, soil stiffness, and reliance on variable cone factors. The VS test failed to provide a sufficient number of reliable data points, rendering it unsuitable for PDF construction in this region. Overall, the PP method stood out as the most practical and dependable approach for generating PDFs to support LSD applications.

5.4.3 Objective (c)

The analysis showed that PDFs derived from the test data did not consistently follow a lognormal distribution. This finding supports previous research by Stott (2020a; 2020b) and Galeandro et al. (2017), indicating that while some soil samples may approximate a lognormal curve, many deviate from this assumption. This calls into question the universal applicability of Eurocode 7's lognormal assumption for shear

strength in cohesive soils, particularly in regions like MMM. As such, each dataset should be evaluated individually rather than relying on a blanket distribution model. The PDF patterns produced by the different testing devices are not similar. Compared to the PP tests, FC tests consistently produced PDFs with a lower mean and 5% fractile value across all analysed samples.

5.4.4 Objective (d)

The study found that PDF patterns differed significantly between the testing devices. The PP produced more realistic and narrow distributions with lower variability, while the FC displayed wider data ranges and often underestimated shear strength in stiff or dry soils. This inconsistency in FC results was particularly noticeable in samples from Bloemdal 1 and Somerton. The VS could not generate reliable PDFs due to insufficient measurements. Furthermore, the PP consistently yielded higher 5% fractile values and means compared to the FC, indicating a better representation of soil strength variability. These results confirm that the choice of test significantly influences the resulting PDF and its interpretation.

5.5 Summary

The FC is not an ideal testing method to measure and assess the inherent variability of undisturbed soils due to issues with the FC methodology that needs to be addressed. The empirical formula and cone factor values used to estimate the undrained shear strength of soils can vary based on factors such as the soil type, penetration depth, and whether the sample is in a remoulded or undisturbed state. The accuracy of the results also depends on the conversion of the penetration readings to shear strength, which relies on the cone factor and/or the logarithmic equation used. Using a higher cone factor, such as 0.9 or 1.0, may mitigate this issue increasing the shear strength readings for FC testing and aligning them more closely with the PP results. If a value of 0.9 or greater is used, it would mean that the cone factor value used for undisturbed testing is relatively higher than factor values for remoulded FC testing and those indicated in literature.

The PP seems to be an effective testing method for measuring and assessing the inherent variability of soils, as it presents fewer issues with the testing application

and apparatus. No equation or conversions are required, and the test provides a direct shear measurement, estimating the undrained shear strength of soil in a quick and reliable way. The PP produced different results and patterns for each sample tested, highlighting the variability between samples across MMM.

The PP and FC provide the advantage of generating a large dataset of test measurements in a shorter timeframe and are arguably more cost-effective than most other shear strength tests. The VS, however, is not a good device to use in the MMM/Free State area, as the infield cohesive soil shear strength is much higher than what the device can measure. Additionally, seasonal rains and moisture content have a significant impact on VS readings, a factor not considered in this research project. At the time of testing, the province had not experienced significant rainfall. Higher moisture contents would result in higher penetration readings, which in turn would lead to lower shear strength readings.

Of the three testing methods analysed and evaluated, the PP produced the more realistic COVs and consistent readings, with a smaller data range for the soil samples compared to the FC. The PP can quickly indicate the degree of variability by using Harr's (1987) classification method and the COV.

5.6 Recommendations

From the investigated research project, the following topics have been identified for future research:

A. Various uncertainties remain in the selection of the cone factor value, particularly for undisturbed testing as opposed to remoulded sample testing. The type of sampler used, and the impact of sampler disturbance may influence the cone factor value. Additionally, research is needed on the use of a heavier weight (>400 g) for FC testing. This would require modifications to the FC device to accommodate larger or heavier weights.
B. The relationship between soil suction measurements and shear measurements using the VS, FC, and PP should be investigated further. Stott (2020a) found that the resulting PDFs from the VS, FC, and suction shear correlated well. The PDFs between the PP and FC did not show a good

correlation due to the cone factor. Additionally, the VS PDFs were inconclusive due to the low shear capability of the device used. Further testing and analysis are needed on a broader spectrum of soil types, including softer soils and clays, using other brands or types of testing apparatus.

C. The correlation between the FC and undisturbed PP could be further investigated, as the study found somewhat similar undrained shear strength readings. A slight increase in the cone factor may result in more comparable data between the two methods. The empirical formula should be adapted for undisturbed samples with low penetration values to reduce data scatter when converting penetration values to shear values. Additionally, the FC could potentially be correlated with other shear strength tests, such as the conventional shear box and triaxial tests (UU and CU tests).

D. Lastly, the FC could also be correlated with other penetration tests, such as the DCP test.

CHAPTER 6: REFERENCES

Abdi, E., Ghalandarayeshi, S., Mousavi, F. & Page-Dumroese, D. S. 2020. Modeling unconfined compressive strength of fine-grained soils: Application of pocket penetrometer for predicting soil strength. *CATENA* 196 (January).

Adeyeri, J. B. 2015. *Technology and Practice in Geotechnical Engineering*. Information Science Reference, Hershey, USA.

Akbas, S. O. and Kulhaway, F. H. 2009. Reliability-Based Design Approach for Differential Settlement of Footings on Cohesionless Soils. *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 135(12):1779-1788.

Allen, T. M. 2007. *Development of WSDOT Pile-Driving Formula and Its Calibration for Load and Resistance Factor Design (LRFD)*. Report No. WA-RD 610.1. Washinton State Department of Transportation, Seattle, USA.

Arnold, A., Askarinejad, A. & Zhang, W. 2018. Undrained shear strength profile of normally and overconsolidated kaolin clay, *Proceedings of 9th International Conference on Physical Modelling in Geotechnic.,* 119-124. CRC Press, London, UK.

AASHTO. 2010. T 296: Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression. Washington D.C, USA: AASHTO

AASHTO. 2022. T 236-22: Direct Shear Test of Soils under Consolidated Drained Conditions. Washington D.C, USA: AASHTO

ASTM. 2012. ASTM D3080: Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions. West Conshohocken, PA, USA: ASTM International.

ASTM. 2018. ASTM D2488. Standard Practice for Description and Identification of Soils. West Conshohocken, PA, USA: ASTM International.

ASTM. 2020. ASTM D4767 Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils. West Conshohocken, PA, USA: ASTM International.

ASTM. 2020. ASTM D7181. Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils. West Conshohocken, PA, USA: ASTM International.

ASTM. 2024. ASTM D2850: Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils. West Conshohocken, PA, USA: ASTM International.

ASTM. 2024. ASTM D6528. Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Fine Grain Soils. West Conshohocken, PA, USA: ASTM International.

Babalola, Z. 2016. Direct Shear and Direct Simple Shear Tests: A Comparative Study of the Strength Parameters and their Dependence on Moisture and Fines Contents. Published MScEng Dissertation, Department of Civil Engineering, University of Cape Town.

Backus, B. 2020. Soil Direct Sheat Test: Understanding the Process. [Internet]. Global Gilson. Available from: https://www.globalgilson.com/blog/soil-direct-sheartest#:~:text=Direct%20shear%20test%20of%20soil%20is%20a%20relatively%20simple%20soil,for%20characterizing%20basic%20soil%20properties. [Accessed: 18 August 2022].

Backus, B. 2021. Triaxial Shear Testing of Soils, Understanding Methods and Equipment. Global Gilson. [Internet]. Available from: https://www.globalgilson.com/blog/triaxial-shear-test-of-soil. [Accessed: 18 August 2022].

Baecher, G. B. and Christian, J. T. *Reliability and Statistics in Geotechnical Engineering*. John Wiley & Sons Inc. Hoboken, USA.

Baguelin, F., Jezeguel, J. F., Lemee, E. & Le Mehaute, A., 1972. Expansion of cylindrical probes in cohesive soils. *ASCE Journal of the Soil Mechanics and Foundations Division* 98(11):1129-1142.

Bappi, P. 2018. Types of triaxial tests. [Internet]. SlideShare. San Francisco, USA. Available from: https://www.slideshare.net/PsyBappi/unconsolidated-undrained-test. [Accessed: 20 August 2022]

Bareither CA, Benson CH and Edil TB (2008) Reproducibility of direct shear tests conducted on granular backfill materials. *Geotechnical Testing Journal* 31(1): 1–11.

Blight, G. E. 2013. *Unsaturated Soil Mechanics in Geotechnical Practice*. CRC Press, London, UK.

Bobrowsky, P. T. & Marker, B. eds., 2018. *Encyclopaedia of Engineering Geology*. Springer, Cham, Switzerland.

Bogusz, W. and Godlewski, T., 2019. Philosophy of geotechnical design in civil engineering - possibilities and risks. *Bulletin of the Polish Academy of Sciences, Technical Sciences* 67(2):289-306.

Bond, A. and Harris, A. 2008. *Decoding Eurocode 7*. Taylor & Francis, New York, USA.

Bowles, J. E. 1996. *Foundation analysis and design*. McGraw-Hill Companies, Inc., New York, USA.

Braatvedt, I. H. 1995. *A Guide to Practical Geotechnical Engineering in Southern Africa*. FRANKI, Olifantsfontein, South Africa.

Bro, A. D., Stewart, J. P. and Pradel, D. E. 2013. Estimating Undrained Strength of Clays from Direct Shear Testing at Fast Displacement. *Geo-congress 2013-Stability and Performance of Slopes and Embankments IIII* 1(231):106-119.

Brown, P. J. and Huxley, M. A. 1996. The cone factor for a 30° cone. *Ground Engineering*, 29(10), 34-36

BS 1377-2. (1990). BS 1377-2: Methods of test for soils for civil engineering purposes, Part 2: Classification tests. London, UK: British Standards Institution (BSi).

BS 1377-7. (1990). BS 1377-7: Methods of test for soils for civil engineering purposes, Part 7: Shear strength tests (total stress). London, UK: British Standards Institution (BSi).

Canelas, D., Fernandes, I. and Lopes, M. d. G. 2018. Use of Fall Cone Test for the determination of undrained shear strength of cohesive soils, *VI International Scientific Conference "Integration, Partnership and Innovation in Construction Science and Education"*. MATEC Web of Conferences 251, Moscow, Russia.

Chang, N. 2011. Comparison of working stress design and limit state design for pile foundations. *Sabinet African Journals* 19(3):18-20.

Chew, Y. M., NG, K. S., and NG, S. F., 2015. The effect of soil variability on the ultimate bearing capacity of shallow foundation. *Journal of Engineering Sciences* 10:1-13.

Controls Group. 2022. Shear Hand Vane Tester. [Internet]. Controls Group, Milan, Italy. Available from: https://controls-group.com/product/shear-hand-vane-tester-controls/. [Accessed: 15 August 2022].

Cortellazzo, G., 2000. Progettazione delle fondazioni superficiali in base all'Eurocodice 7. *Rivista Italiana di Geotecnica* 2:39-50.

Craig, R. F. 2004. *Craig's Soil Mechanics*. Taylor & Francis, New York, USA.

Das, B. M., 2008. *Advanced Soil Mechanics*. Taylor & Francis, London, UK.

Das, B. M. (2010). *Principles of Geotechnical Engineering* (7th ed.). Cengage Learning.

Das, B. M. and Sobhan, K., 2012. *Principles of Geotechnical Engineering*. Cengage Learning, Stamford, USA.

Dastider, A. G., Chatterjee, S. and Basu, P. 2021. Advancement in Estimation of Undrained Shear Strength through Fall Cone Tests. *Journal of Geotechnical and Geoenvironmental Engineering* 147(7).

Day, R. W. 2012. *Geotechnical Engineer's Portable Handbook*. McGraw-Hill, New York, USA.

Day, R. W. 2024. Appropriate methods of design for use in geotechnical practice, *Proceedings of the 18th Regional African Conference on Soil Mechanics and Geotechnical Engineering*, 3-23. Laboratory of Environment, Water, Geomechanics and Buildings, University of Sciences and Technology Houari Boumediene, Algiers, Algeria.

de Koker, N. and Elvin, A. 2018. Risk-based member reliability in structural design. *Journal of the South African Institution of Civil Engineering* 60(4): 16-24.

Dolinar, B. 2004. Undrained shear strength of saturated cohesive soils depending on consolidation pressure and mineralogical properties. *Acta Geotechnica Slovenica,* 1:5-11.

du Preez, C.C., Kotzé, E., and van Huyssteen, C.W. 2019. Soils, Agriculture and Food. In: Knight, J., Rogerson, C. (eds) *The Geography of South Africa*, Ch. 12, 111 -121. Springer International Publishing, Cham, Switzerland.

Eaton, B.C., Moore, R.D. and MacKenzie, L.G. 2019. Percentile-based grain size distribution analysis tools (GSDtools) - estimating confidence limits and hypothesis tests for comparing two samples. *Earth Surface Dynamics 7*: 789-806.

Eden, W. J. and Law, K. T. 1980. Comparison of Undrained Shear Strength Results Obtained by Different Test Methods in Soft Clays. *Canadian Geotechnical Journal* 17(3):369-381.

Ehnbom, V. and Kumlin, F. 2011. Reliability Based Design of Lime-Cement Columns based on Total Settlement Criterion. Published MScEng Thesis, Department of Civil Architecture and Built Environment, Royal Institute of Technology.

European Committee for Standardisation (CEN). 1990. Eurocode – Basis of Structural Design, Brussels, Belgium.

European Committee for Standardisation (CEN). 2004. Eurocode 7: Geotechnical design – Part 1: General rules. EN 1997-1:2004. Brussels: CEN.

European Committee for Standardisation (CEN). 2018. EN 17892-10:2018. Geotechnical investigation and testing – Laboratory testing of soil – Part 10: Direct shear tests. Brussels: CEN

Feng, T. W. 2005. Reappraisal of the fall cone test, *Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering,* 357-360. Millpress Science Publishers, Osaka, Japan

Fenton, G. A. and Griffiths, D. V. 2010. Reliability-Based Geotechnical Engineering. West Palm Beach, *ASCE GeoFlorida Conference*, 14-52. Geo-Institute of ASCE, West Palm beach, USA.

Ferreira, R. S. and Robertson, P. K., 1992. Interpretation of Undrained Self-Boring Pressuremeter Test Results Incorporating Unloading. *Canadian Geotechnical Journal 29*(June):918-928.

Franke, E., Kiekbusch, M. & Schuppener, B., 1979. A new Direct Simple Shear device. *Geotechnical testing journal* 2(4): 190-199.

Galeandro, A., Doglioni, A. & Simeone, V., 2017. Statistical analyses of inherent variability of soil strength and effects on engineering geology design. *Bulletin of Engineering Geology and the Environment* 2(76):587-600.

Garneau, R. & LeBihan, J. P., 1977. Estimation of some properties of Champlain clays with the Swedish fall cone. *Canadian Geotechnical Journal* 14(4):571-581.

Geoscience. 2008. Dominant lithologies of South Africa. [Internet]. Council for Geoscience, Silverton, South Africa. Available from: https://www.geoscience.org.za/cgs/systems/publications/downloadable-material/. [Accessed: 16 August 2022].

Gylland, A. S., Thakur, V. and Emdal, A. 2016. Extended interpretation basis for the vane shear test, Proceedings of the *17th Nordic Geotechnical Meeting, 233-240.* NGM, Reykjavík, Iceland.

Handy, R. L. and Spangler, M. G. 2007. *Geotechnical Engineering: Soil and Foundation Principles and Practice.* McGraw-Hill, New York, USA.

Hansbo, S. 1957. A new approach to the determination of the shear strength of clay by the fall-cone test, *Royal Swedish Geotechnical Institute Proceedings No.14*, 5-47. Stockholm, Sweden.

Harr, M. E., 1987. *Reliability-Based Design in Civil Engineering*. New York: McGraw-Hill.

Harvard Online, 2021. How to interpret violin plots. [Internet]. LabXchange, Harvard University, Cambridge, United States of America. Available from: https://www.labxchange.org/library/items/lb:LabXchange:46f64d7a:html:1 [Accessed 3 August 2024].

Hernandez, H. 2017. Multivariate Probability Theory: Determination of Probability Density Functions. Report No. 2017-3. ForsChem Research, Medellin, Columbia.

Holtz, R. and Kovacs, W. D. 1981. *An Introduction to Geotechnical Engineering*. Prentice-Hall Inc., Englewood Cliffs, USA.

Houlsby, G. T., 1982. Theoretical analysis of the fall cone test. *Géotechnique* 32(2):111-118.

Hrubesova, E., Lunackova, B. and Brodzki, O. 2016. Comparison of Liquid Limit of Soils Resulted from Casagrande Test and Modificated Cone Penetrometer Methodology. *Procedia Engineering* 142(2016): 364-370.

Huber, M. 2013. Soil variability and its consequences in geotechnical engineering. Published Deng Dissertation, Faculty of Civil and Environmental Engineering, University of Stuttgart, Institute for Geotechnics, University of Stuttgart, Stuttgarrt, Germany.

Humboldt. 2022. Soil Penetrometer, Pocket Type. [Internet]. Humboldt, Elgin, USA. Available from: https://www.humboldtmfg.com/soil-penetrometer-pocket-type.html. [Accessed: 15 August 2022].

IIT Gandhinagar, 2024. Direct Shear Test Arrangement. [Internet]. Indian Institute of Technology Gandhinagar, Gujarat, India. Available from: https://research.iitgn.ac.in/stl/wp/direct-shear/ [Accessed 7 June 2024].

Isik, S. N., Ulusay, R. and Doyuran, V. 2014. Comparison of undrained shear strength by pressuremeter and other tests, and numerical assessment of the effect of finite probe length in pressuremeter tests. *Bulletin of Engineering Geology and the Environment* 74(3): 685-695.

International Organization for Standardization (ISO). 2017. 17892-6. Geotechnical investigation and testing - Laboratory testing of soil - part 6: Fall Cone test, Geneva, Switzerland.

Jackson, R. 2020. Soils: Cohesive versus Cohesionless. [Internet]. Earthworks Environmental. Available from: https://www.earthworksenv.com/posts/soils-cohesive-versus-cohesionless. [Accessed: 27 August 2021].

Karlsson, R. 1961. Suggested improvements in the liquid limit test, with reference to flow properties of remoulded clays. *Proceedings of the 5th international conference of soil mechanics and foundation engineering*, vol. 1, pp. 171–184.

Kjellman, W., 1951. Testing the shear strength of clay in Sweden. *Géotechnique* 2(3):225-235.

Kulhawy, F. H., Phoon, K.-K. and Prakoso, W. A. 1992. Uncertainty in basic properties of geomaterials. *Geotechnical Special Publication* 31:95-115.

Ladd, C. C. and DeGroot, D. J. 2003. Recommended Practice for Soft Ground Site Characterization: Arthur Casagrande Lecture, *Proceedings of the 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering.* Massachusetts Institute of Technology, Cambridge, USA.

Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F. and Poulos, H.G. 1977. Stress-deformation and strength characteristics, *9th International Conference on Soil Mechanics and Foundation Engineering*, 421-494. ISSMGE, Tokyo, Japan.

Leung, C. F., 2003. Soil Mechanics and Geotechnical Engineering (12ARC), *Proceedings of the 12th Asian Regional Conference*. World Scientific Publishing Company, Toh Tuck Link, Singapore.

Llano-Serna, M. A. & Contreras, L. F. 2020. The effect of surface roughness and shear rate during fall-cone calibration. *Géotechnique* 70(4):332-342.

Lo, S. R. & Li, K. S. V., 2007. Characteristic and design soil parameters: use of statistics. *Geotechnical Engineering* 160(3):141-146.

Lunne, T., Robertson, P. K., & Powell, J. J. M. (1997*). Cone Penetration Testing in Geotechnical Practice*. CRC Press.

Lysay, G. J. 1999. Comparison of limit states design with working stress design for shallow foundations. Published MScEng Dissertation, Department of Civil Engineering, The University of British Columbia, Vancouver, Canada.

Mair, R.J. and Wood, D.M., 1987. *Pressuremeter testing: Methods and interpretation.* CIRIA Ground Engineering Report: In-situ Testing. London: Butterworths.

Mahmood, Z. 2020. Reliability-based optimization of geotechnical design using a constrained optimization technique. *SN Applied Sciences* 2(168).

Matthews, C. 1998. 15 - *The 'Schloss Adler' railway – design safety. In: Case Studies in Engineering Design*. Butterworth-Heinemann, London, UK.

McCarthy, D. F. 2006. Essentials of Soil Mechanics and Foundations: Basic Geotechnics. Pentice Hall.

Ménard, L., 1956. *An Apparatus for Measuring the Strength of Soils in Place*. Published MSc Eng Thesis. University of Illinois, Urbana-Champaign, USA.

Mgangira, M. B., Jenkins, K. J., Paige-Green, P. and Theyse, H. L. 2011. Proposed Protocol for Triaxial Testing of Resilient Modulus and Permanent Deformation Characteristics of Unbound and Bound Granular Materials. Stellenbosch: CSIR and Stellenbosch University.

Mitchell, J. K. and Soga, K. 2005. *Fundamentals of Soil Behavior*. John Wiley & Sons, Inc., Hoboken, USA.

Muntohar, A. S. and Hashim, R. 2005. Determination of Plastic Limits of Soils Using Cone Penetrometer: Re-Appraisal. *Jurnal Teknik Sipil* 11(2).

Murthy, V. N. S. 2002. *Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering*. Marcel Dekker Inc., New York, USA.

Olson, R. E. 1989. Types of laboratory apparatus for shear testing of soils. Department of Construction Engineering, Chaoyang University of Technology, Taichung, Taiwan.

Peila, D., Viggiani, G. and Celestino, T. 2019. Tunnels and Underground Cities: Engineering and Innovation meet Archaeology, Architecture and Art, CRC Press, London, UK.

Phoon, K. K. and Kulhaway, F. H. 1997. Characterization of geotechnical variability. *Canadian Geotechnical Journal* 36(4):612-624.

Phoon, K. K. and Kulhawy, F. H. 1996. On quantifying inherent soil variability. *Geotechnical Special Publication* 58(I):326-340.

Phoon, K.K., and Kulhawy, F.H. 1999. Characterization of geotechnical variability. *Canadian Geotechnical Journal*, 36(4), 612-624.

Phoon, K.-K., Kulhawy, F. H. and Grogoriu, M. D. 1995. *Reliability-Based Design of Foundations for Transmission Line Structures*. Report No. TR-105000. Cornell University, Ithaca, USA.

Phoon, K.-K., Simpson, B., Ching, J., Day, P., Wang, Y., and Li, D. 2017. Joint TC205/TC304 Working Group on “Discussion of statistical/reliability methods for Eurocodes”. International Society for Soil Mechanics and Geotechnical. Rotterdam, Netherlands.

Polski Komitet Normalizacyjny (2009) PKN-CEN ISO/TS 17892:2009 Geotechnical investigation and testing - Laboratory testing of soil. Warsaw: Polski Komitet Normalizacyjny.

Powrie, W. 2014. *Soil Mechanics: Concepts and Applications*. CRC Press, London, UK.

Reddy, S. C. 2014. Ultimate and Serviceability Limit State Reliability-based Axial Capacity of Deep Foundations. Published PhD Thesis, School of Civil and Construction Engineering, Oregon State University, Corvallis, USA.

Robertson, P. K. (2016). *Soil Classification Using the Cone Penetration Test.* Canadian Geotechnical Journal, 27(1), 151–158.

Robertson, P. K., & Campanella, R. G. (1983). Interpretation of Cone Penetration Tests—Part I: Sand. *Canadian Geotechnical Journal*, 20(4), 718–733.

Robertson, I. N. & Chock, G. 2017. Overview and technical background to development of ASCE 7-16 Chapter 6, Tsunami Loads and Effects, *Proceedings of the 16th World Conference on Earthquake Engineering*. Chilean Association on Seismology and Earthquake Engineering (ACHISINA), Santigo, Chile.

Rui, Y. & Yin, M., 2017. Interpretation of pressuremeter test by finite-element method. Department of Engineering, University of Cambridge, UK.

SABS. 1995. South African Standard Code of Practice for The Design of Foundations for Buildings. SABS 0161-1980. South Africa: THE COUNCIL OF THE SOUTH AFRICAN BUREAU OF STANDARDS.

SABS. 2012. SANS 633: Soil profiling and rotary percussion borehole logging on dolomite land in Southern Africa for engineering purposes. Pretoria: SABS.

SANS. 1995. Basis of Structural Design and Actions for Buildings and Industrial Structures – part 5: Basis of Geotechnical Design an Actions. SANS 10160-5. South Africa: THE COUNCIL OF THE SOUTH AFRICAN BUREAU OF STANDARDS.

Schneider, H. R. & Schneider, M. A., 2013. *Dealing with uncertainties in EC7 with emphasis on determination of characteristic soil properties*. IOS Press: Online.

Schweckendiek, T. and Calle, E. O. F. 2010. A Factor of safety for Geotechnical Characterisation. The 17th *Southeast Asian Geotechnical Conference*. ISSMGE, Taipei, Taiwan.

Selänpää, J., Di Buò, B., Länsivaara, T. and D'Ignazio, M. 2017. Problems related to field vane testing in soft soil conditions and improved reliability of measurements using an innovative field vane device. In: ed. Thakur, V., L'Heureux, J.-S., Locat, A., *Landslides in Sensitive Clays,* 109-119*,* Springer, Cham, Switzerland.

Spear, Q. and Selenatas, D. 2011. Eurocode decoded: Report on the BGAS Symposium on Eurocode 7 today and tomorrow. *Ground Engineering* (445):22-24.

Stott, P. R. 2020a. The Impact of Variability of Soil Properties on Characteristic Values for Geotechnical Design, *Proceedings, 4th International Conference on Geotechnical Engineering*, March 2020, Proservice, Hammamet, Tunisia

Stott, P. R., 2020b. Quo Vadis Geotechnical Engineering South Africa?. *Journal of the South African Institution of Civil Engineering* 28(3):64-67.

Stott, P. R. and Theron, E. 2016. Variability in soil properties and its consequences for design, *Proceedings of the First Southern African Geotechnical Conference,* 5-6 May, CRC Press, London, UK.

Stott, P. R. & Theron, E., 2019. Investigation of the probability density functions for suction potential of soils, *Proceedings of the 17th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, 7–9 October, Cape Town, South Africa.

Stróżyk, J. and Tankiewicz, M. 2013. Undrained shear strength of the heavily consolidated clay. *Land Reclamation* 45(2):207-216.

SUDAS Design Manual, 2013. Statewide Urban Design and Specifications Program. Iowa State University, Maes, USA.

Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice* (3rd ed.). Wiley.

Theron, E., Stott, P. R., Vosloo, P. and Langroudi, A. A. 2019. Assessment of the suitability of the fall cone method to replace the Casagrande cup for liquid limit determination of South African soils, *Proceedings of the 17th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, 175-179, International Society for Soil Mechanics and Geotechnical Engineering, Cape Town, South Africa.

Thomas, D., 1965. Static Penetration Test in London Clay. *Geo-technique* VX(2):174-179.

Tukey, J.W., 1977. *Exploratory Data Analysis*. Reading, MA: Addison-Wesley.

Uzielli, M., Lacasse, S., Nadim, F. and Phoon, K. K. 2006. Soil variability analysis for geotechnical practice, *Proceedings of the 2nd International Workshop on Characterisation and Engineering Properties of Natural Soils*, 1653-1752. CRC Press, London, UK.

Vanmarcke, E. H., 1977. "Probabilistic Modeling of Soil. *Journal of the Geotechnical Engineering Division* 103(GT11):1227-1246.

VJ Tech. 2021. An Introduction to Direct Simple Shear Testing. Retrieved from VJ Tech Limited: https://www.vjtech.co.uk/an-introduction-to-direct-simple-shear-testing/

Vosloo, P. 2022. Assessment of the suitability of the fall cone method and the Casagrande cup method for determining the liquid limit of South African soils. Published MEng Dissertation, Department of Civil Engineering, Central University of Technology, Bloemfontein, South Africa.

Wahls, H. E. 1983. *Evaluating Strength Parameters of Simple Clays: Geotechnical Consideration of Residual Soils*. Transportation Research Board, Washington D.C., USA.

Waters, S.G. 2022. Development of an information system for geotechnical engineers towards improved decision making. Published MEng Dissertation, Department of Civil Engineering, Central University of Technology, Bloemfontein, South Africa.

Windle, D. and Wroth, C. P. 1977. In situ measurement of the properties of stiff clays, *9th International Conference on Soil Mechanics and Foundation Engineering*, 347-352. Japanese Society of Soil Mechanics and Foundation Engineering, Tokyo, Japan.

Wood, D. M. 1985. Some fall cone tests. *Géotechnique* 35(1):64-68.

Wroth, C. P. and Hughes, J. M. O. 1974. The development of a special instrument for the in-situ measurement of the strength and stiffness of soils, Subsurface Exploration for Underground Excavation and Heavy Construction. ASCE, Henniker, USA.

Wroth, C. P. and Wood, D. M. 1978. The correlation of index properties with some basic engineering properties of soils. *Canadian Geotechnical Journal* 15(137):137-145.

Yasun, A., 2017. Capability of pocket penetrometer to evaluate unconfined compressive strength of Bagdad clayey soil. *Al-Nahrain Journal for Engineering Sciences* 21(1): 66-73.

Zeng, X., Li, Y., Lui, X. Yao, J. and Lin, Z. 2020. Relationship between the Shear Strength and the Depth of Cone Penetration in Fall Cone Tests. Advances in Transportation Geotechnics 2020(9):1-8.

Zreik, D., Ladd, C., and Germaine, J. 1995. A New Fall Cone Device for Measuring the Undrained Strength of Very Weak Cohesive Soils. ASTM International. *Geotechnical Testing Journal* 18(4): 472–482.