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Evaluation of the Clay Cutting Test

Assessment of a new method for measuring undrained shear strength in soils through comparative testing

Master's thesis in Infrastructure and Environmental Engineering

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CHALMERS UNIVERSITY OF TECHNOLOGY
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Cover: Picture of the device used to perform the Clay Cutting Test.

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Abstract

The Clay Cutting Test (CC-Test) is a newly developed laboratory method to assess the undrained shear strength of soil samples. Unlike other tests that provide a single value of strength, the CC-Test presents a continuous strength profile by measuring the resistance as a thin wire cuts through the soil sample. This thesis evaluates the CC-Test through a combination of archive data comparison and in-depth study of strength profiles from clay samples of varying depths.

In the archive compilation, representative values of shear strength from the CC-Test were compared to those obtained with direct simple shear (DSS) and fall cone test. For strengths below 25 kPa a good agreement between the methods was observed. At higher strengths, the fall cone becomes less reliable and the DSS data are limited. A parameter study indicated that density, water content and thread diameter have an impact on the measured resistance. However, in the range of typical soils, the deviation of the parameters is within ± 10% of the measured strength.

The in-depth analysis focused on 12 stiffer clay samples from 7-35 m depth. For each sample, uniaxial compression test (UCT), CC-Test and fall cone test were conducted on the same specimen, to minimize deviation due to natural variation of the soil. The CC-Test yielded higher values than the fall cone and generally lower than the UCT. For samples below 20 m, even the upper quartile of the CC-Test strength profile could be considered conservative compared to UCT. The correlation between the CC-Test measurements and the UCT was strongest near the locations corresponding to the ends of the shear plane. In some tests, oscillating patterns appeared in the measured strength curves, implying an effect of minor material changes within the soil. These patterns were also present in reference tests, indicating they are not caused by prior loading or sample disturbance.

The CC-Test shows potential as a complement to established methods for determining undrained shear strength due to its simple setup, fast execution, and ability to present strength variation within the soil specimen. This makes it particularly useful in quality control of soil samples. For it to be accepted in the industry, clear guidelines on how to interpret the data is crucial. Further development, such as measuring the remoulded shear strength with the CC-Test would further strengthen its applicability.

Keywords: CC-Test, Clay Cutting Test, Clay Cutter, undrained shear strength, soil testing, soil characterization, wire penetration test.

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Karin Bergroth, Gothenburg, June 2025

List of Acronyms

Below is the list of acronyms that have been used throughout this thesis listed in alphabetical order:

CC-Test	Clay Cutting Test
CPT	Cone Penetration Test
CRS	Constant Rate of Strain
DSS	Direct Simple Shear (test)
OCR	Over Consolidation Ratio
SGI	Swedish Geotechnical Institute
UCT	Unconfined Compression Test

Nomenclature

Below is the nomenclature of indices, sets, parameters, and variables that have been used throughout this thesis.

Parameters

A	Area
A_s	Surface area
c	Cohesion
c'	Effective cohesion
c_u	Undrained shear strength
$c_{u,DSS}$	Undrained shear strength from DSS test
$c_{u,fc}$	Undrained shear strength from fall cone test
$c_{u,FV}$	Undrained shear strength from field vane
$c_{u,Hansbo}$	Undrained shear strength from Hansbos empirical relation
$c_{u,SGI}$	Undrained shear strength from SGIs empirical relation
g	Gravitational acceleration
i	Cone penetration depth
K	Cone factor
m	Mass
q_t	Tip resistance in CPT
T	Torque
w_L	Liquid limit
w_N	Water content
τ	Shear strength
μ	Correction factor
ϵ	Axial strain at failure
ρ	Density

σ	Total normal stress
σ'	Effective normal stress
σ_1	Major principal stress
σ_3	Minor principal stress
σ'_c	Preconsolidation pressure
σ_{v0}	Total vertical in-situ stress
σ'_{v0}	Effective vertical in-situ stress
ϕ	Frictional angle
ϕ'	Effective frictional angle

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1

Introduction

Within the field of geotechnical engineering, both in-situ and laboratory testing of soil are essential for making well-informed and precise assessment of soil behaviour (Hunt, 2005). Reliable and accurate testing serves as the foundation for all further calculations and design decisions (Briaud, 2013). As numerical modeling and analysis methods continue to advance, the need for high-quality and accurate input data becomes increasingly important.

There is a variety of commonly used methods for determining soil parameters, where the choice of method depend on factors such as soil conditions, time constraints, budget and specific project requirements (Knappett & Craig, 2012). Continuous improvement of existing methods, as well as developing new methods can improve cost efficiency for geotechnical investigations. Furthermore, it can provide greater insight and understanding of soil behaviour.

1.1 Background

The Clay Cutting test (CC-Test) is a laboratory method to test the undrained shear strength of undisturbed soil samples and is developed by Peter Hedborg. It is a simple and fast method, designed to complement existing methods for shear strength testing (Hedborg, 2025). The specimen is cut with a thin wire, while the resistance to push the wire through the soil is measured continuously trough the sample.

The CC-Test measures and presents the variation of shear strength within soil samples, a feature not possible with other commonly used laboratory methods (Hedborg, 2024). Instead of providing a single value of shear strength, the CC-Test can give a more detailed strength profile, which possibly can contribute to a better understanding of soil behaviour.

The method has previously been validated mainly against the fall cone test as well as the direct simple shear test (DSS) (Hedborg, 2024). The equation for calculating the undrained shear strength from the measured resistance in the CC-Test is empirically derived through comparison to the previously mentioned methods. Therefore the failure mechanism and mathematical correlations have not yet been fully established.

Prior to this project, the results from the CC-Test has shown good agreement to the results of fall cone and DSS tests (Hedborg, 2025). However, the impact of soil properties has not been studied more in detail. Furthermore, a thread diameter of 0.25 mm has been commonly used in the CC-Test (Hedborg, 2024). This diameter does not cause significant deflection while cutting through the soil. Different diameters have previously been tested sporadically in the range of 0.09 - 0.37 mm. However, no further systematic studies have been carried out to investigate the influence of the thread diameter on test result.

1.2 Aim

The aim of the project is to evaluate and validate the CC-Test, as a part of the method development. This includes ascertaining the accuracy and reliability of the CC-Test by comparing its results with those obtained with fall cone test and DSS test. Furthermore, focus will be on investigating the influence of different soil properties and thread diameter to the measurements. The goal is to gain more information regarding the test methods accuracy and limitations.

The project also aims to investigate in greater detail how the results from the CC-Test should be evaluated. As of today, an average value from the steady state part of the test curve is usually adopted as the representative value of the undrained shear strength from the test. The purpose is to evaluate this methodology by investigating how the result of CC-Test relate to fall cone and unconfined compression test (UCT).

1.3 Research questions

The following research questions forms the base of the project.

1. How does the representative value of undrained shear strength obtained from the CC-Test compare to the values from the DSS and fall cone tests?
2. How do soil parameters such as liquid limit, water content, and density influence the correlation between the undrained shear strength values from the CC-Test and the DSS or fall cone tests?
3. How does the diameter of the wire used in the CC-Test influence the correlation between the undrained shear strength values from the CC-Test and the DSS or fall cone tests?
4. How does the strength profile obtained with the CC-Test relate to the values obtained with UCT or fall cone tests?
5. Can the average value of the steady-state part of the strength profile from the CC-Test be a representative value of the undrained shear strength?

1.4 Limitations

The following limitations have been identified in the scope and execution of this study:

- Only commonly used methods for determining undrained shear strength are included in the theoretical background. These are selected based on their relevance to the CC-Test, focusing on standard procedures, assumptions, empirical correction factors, advantages and limitations.
- This study compares the CC-Test results only against results from fall cone, DSS and UCT. Other in-situ or laboratory methods are not included in the comparison.
- The natural soil samples used in the analysis are limited to sites within the Gothenburg region. All samples are assumed to be undisturbed and were retrieved using a piston sampler. However, minor sample disturbance can not be completely ruled out and may influence results.
- This study does not investigate the influence of penetration rate on CC-Test results. Only tests performed at the standard rate of 40 mm/min are included in the analysis.
- The archive compilation is dominated by soft, shallow clays, while the in-depth study focuses on deeper, stiffer soils. These differences in material behaviour may influence the comparability and interpretation of test results.

2

Shear strength of soil

Shear strength is defined as the maximum shear stress a material can resist before it fails by shearing. Shear stress causes different parts or layers of a material to slide relative each other (Shaker, 2021). This sliding motion is caused by forces acting parallel to each other, but in opposite directions. In geotechnical engineering, this shearing is the most common mode of failure, meaning the shear strength controls the ultimate loads that can be carried by the soil (Briaud, 2013).

Understanding the shear strength of soil is thereby essential in design and stability analysis of foundations, slopes, embankments, and retaining walls (Knappett & Craig, 2012). The shear strength of the soil is determined by the effective stress state, and the resistance to shear forces is provided by frictional forces in the contact between the particles and cohesion (Knappett & Craig, 2012). Cohesion is a bond of electrochemical attractions between the particles which is more significant in fine-grained soils like clays (Hunt, 2005).

The frictional resistance appear due to normal forces pressing the particles against each other. Since the magnitude of friction is controlled by the normal forces, the shear strength of soil is controlled by the normal forces, represented by the effective stress. This relationship is summarized in the Coulomb failure criterion, which is a widely used model to describe soil strength, see Equation 2.1 (Verruijt, 2018).

$$\tau = c + \sigma tan\phi \tag{2.1}$$

where
τ = Shear strength
c = Cohesion
σ = Normal stress
ϕ = Internal friction angle

Equation 2.1 is expressed in total stress terms, which is typical for undrained analysis in fine-grained soils such as clays, see Section 2.1. In undrained conditions, shear strength is commonly assumed to be a constant value, c_u, as effective stress changes can not be accounted for during rapid loading (Knappett & Craig, 2012). For effective stress analysis, the Coulomb criterion is expressed as Equation 2.2.

$$\tau = c' + \sigma' tan\phi' \tag{2.2}$$

where
τ = Shear strength
c' = Effective cohesion
σ' = Effective normal stress
ϕ' = Effective internal friction angle

Based on the theory of stresses, Mohr presented a circle that graphically could describe all stresses action on a soil element on all possible planes (Verruijt, 2018). Each point on the circle corresponds to a specific plane orientation. The Coulomb failure envelope can be plotted together with the Mohr circle forming the Mohr-Coulomb failure criterion. This criterion is used to assess the stress state that will result in plastic failure, meaning the soil deforms irreversibly. Failure is assumed to occur when the Mohr circle first touches the failure envelope. At that point, the shear stress on a specific plane is equal to the shear strength of the soil. If the stress state lies outside the envelope, it indicates that the soil has already failed (Verruijt, 2018).

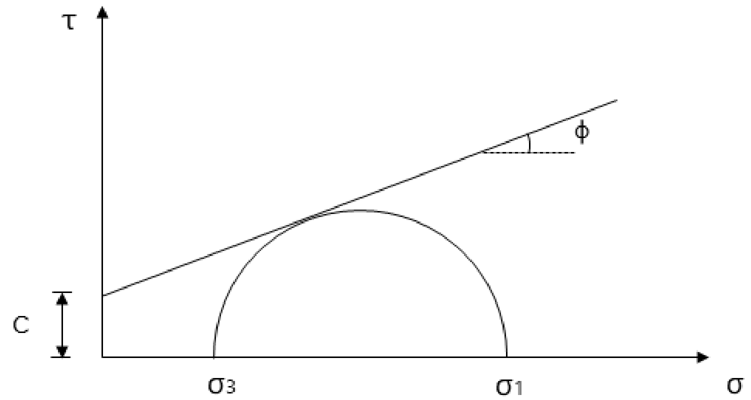


Figure 2.1: Mohr Coulomb failure criterion. The circle represents all stresses acting on a soil element on all planes and the straight line is the failure envelope described in Equation 2.1, based on Knappett and Craig (2012)

2.1 Undrained shear strength

The term undrained concerning soil refers to the state of soil where no consolidation occurs due to limited drainage of the pore water. This state occurs because of low permeability of the material and rapid loading, or because of impermeable boundaries limiting the drainage (Verruijt, 2018). In coarse grained material, such as sand and gravel, water drains rapidly due to high permeability. As a result, the undrained shear strength of coarser soils is rarely relevant in real world cases, since these material behave drained under normal loading conditions (Knappett & Craig, 2012)

For fine grained soils the permeability is lower and the material drains over a much longer time. When a load is applied on such soil, excess pore pressures develop. This initial state represents the more critical condition concerning stability as part of the applied load is carried by the water in the pores (Verruijt, 2018). Over time, as the water drains and pore pressure dissipate, more of the load is transferred to the soil skeleton. The effective stress and in turn the shear strength is increased and the soil structure is more stable (Knappett & Craig, 2012). Since the undrained case is the more critical concerning stability, the undrained shear strength is often the most relevant.

The undrained shear strength is not a constant property of the soil. Besides the fact that it is stress dependent as mentioned previously, it is also controlled by the load direction. Typically there are three zones to describe the different types of shear strength; active, direct and passive (Larsson et al., 2007). The loading cases are presented in Figure 2.2. Each zone or loading case has corresponding laboratory tests which are more explained in detail in the following chapters. In the active zone the compression triaxial test is the test that gives best values. In the passive zone, the extension triaxial test is most relevant as the soil is deformed in this way. In the direct zone the direct simple shear test (DSS) is the most representative method.

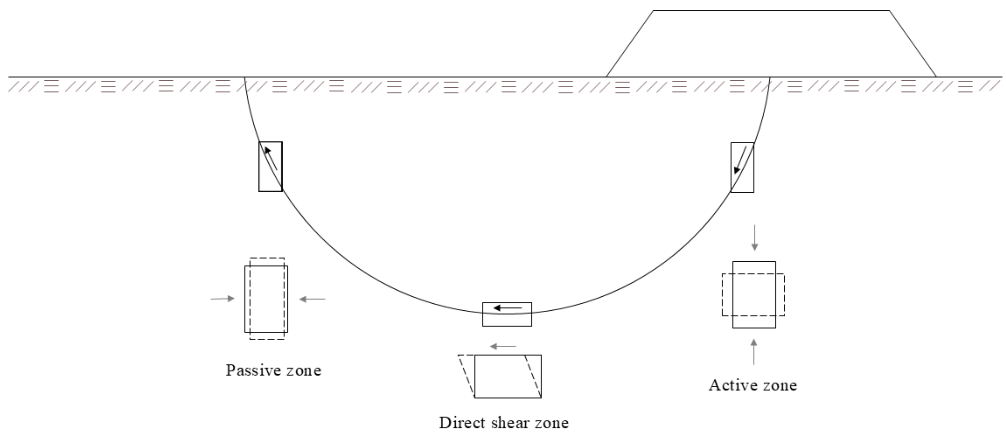


Figure 2.2: Conceptual slip surface and corresponding stress situations; active, passive and direct, based on Larsson et al. (2007).

Besides the triaxial and DSS test there are additional established methods for assessing the shear strength of soil which are all in common use. The reason for the diversity of methods is that they all come with advantages and limitations (Briaud, 2013). Certain methods are suitable for specific soil types, some are more accurate in homogeneous soil while others are more suitable in stratified conditions. Additionally, some methods are fast whereas some are more precise. Often multiple methods are used, and a trend is evaluated (Sällfors et al., 2017).

2.2 Empirical relations

Empirical relations can be used as a first assessment of the undrained shear strength, however real test are always more reliable (Larsson et al., 2007). Based on the fall cone and vane test, Hansbo developed a relationship between the liquid limit and preconsolidation pressure to the undrained shear strength. The complete relation is presented in Equation 2.3. The value of strength can be directly comparable to the unreduced values of strength obtained with fall cone or field vane, see Section 2.3.1 and 2.4.1. For normally consolidated and slightly over consolidated Scandinavian clays the relation is relatively accurate (Larsson et al., 2007). However, when comparing the empirical values to those obtained by field vane, some significant scatter is common. Empirical values should thereby be used with caution.

$$c_{u,Hansbo} = \sigma'_c * 0.45 * w_L \quad (2.3)$$

where
$c_{u,Hansbo}$ = Undrained shear strength obtained with Hansbos formula [kPa]
σ'_c = Preconsolidation pressure [kPa]
w_l = Liquid limit [%]

Swedish Geotechnical Institute (SGI) has also presented an empirical equation to assess the undrained shear strength, see Equation 2.4 and 2.5 (Larsson et al., 2007). The equation is empirically calibrated against results from triaxial and direct simple shear test (Larsson & Åhnberg, 2003). In addition to the parameters used in Hansbos equation, SGIs recommendation also considers soil type and mode of loading by determining the material parameters a and b.

$$c_{u,SGI} = a * \sigma'_c * OCR^{(1-b)} \quad (2.4)$$

or

$$c_{u,SGI} = a * \sigma'_{vo} * OCR^b \quad (2.5)$$

where
$c_{u,SGI}$ = Undrained shear strength obtained with SGI relation [kPa]
a = Material parameter [-]
σ'_{vo} = Effective in-situ stress [kPa]
σ'_c = Preconsolidation pressure [kPa]
OCR = Overconsolidation ratio [-]
b = Material parameter [-]

Concerning clay or silty clay, the values of a can be obtained by the following relations depending on the mode of loading and the liquid limit (Larsson et al., 2007).

Active shear a ≈ 0.33
Direct shear a $\approx 0.125 + 0.205 w_L/1.17$

Passive shear a $\approx 0.055 + 0.275w_L/1.17$

If the organic content is over 2% in the soil, the value of a are increased accordingly (Larsson et al., 2007). For active shear, a is set in the range 0.33-0.5 increasing linearly between 2 and 6% organic content. For a higher organic content than 6% a value of 0.5 should be adopted. For direct shear a is increasing linearly up to 0.4 between 2 and 20% organic content. If the organic content is higher than 20% the value of a is set to 0.4. The value of b depends on the mode of loading and can vary between 0.7-0.9, but typically 0.8 can be adopted (Larsson et al., 2007).

2.3 In-situ methods to test shear strength of soil

Methods used on site has the advantage of testing the soil in its natural state with accurate stress levels (Larsson et al., 2007). However, there is a possibility that the equipment and driving causes disturbance of soil giving misleading results.

2.3.1 Field vane test

The vane shear test is a widely used in situ method for determining the undrained shear strength of fine-grained soils (Swedish Institute for Standards, 2020). The ultimate strength as well as the remoulded can be determined and from that the soil sensitivity can be derived. The test is especially suitable in assessing soil strength in soft clays, as these soils are highly sensitive to disturbance.

The setup consists of a vane with four thin plates placed at 90-degree angles that is inserted into the soil at a specified depth, ensuring minimal disturbance to the surrounding soil (Sällfors, 2013). Once the vane is positioned, it is rotated at a constant rate, and the torque is measured until failure in the soil.

In the field vane test, shear failure is assumed to develop along a cylindrical surface surrounding the vane, with a diameter corresponding to the vane width (Knappett & Craig, 2012). At failure, the shear strength is assumed to be fully developed and distributed evenly along the perimeter and across the ends of the cylinder. From the recorded maximum torque, the peak undrained shear strength is calculated by Equation 2.6 (Knappett & Craig, 2012).

$$T = \pi C_{u,fv} \left(\frac{d^2 h}{2} + \frac{d^3}{6} \right) \tag{2.6}$$

where
$c_{u,fv}$ = Peak shear strength value from field vane [kPa]
T = Torque at failure [Nm]
d = Vane diameter [mm]
h = Vane height [mm]

In a Swedish context Equation 2.7 is commonly used, provided by Swedish Institute for Standards (2020). Usually the values are corrected with a factor, μ, based on the liquid limit of the soil.

$$c_{u,fv} = \frac{6000000}{7\pi D^3} T_{\max} \tag{2.7}$$

where
$c_{u,fv}$ = Peak shear strength value [kPa]
T_{max} = Maximum torque [Nm]
d = Vane diameter [mm]

The friction from the testing rod contributes to the measured torque which is problematic especially in deeper layers as the contribution becomes significant. A slip coupling is often used to minimize the friction between the testing rod and the surrounding soil, ensuring that the measured torque is mainly from the shear strength of the soil and not rod friction (Selänpää et al., 2017).

Soil sensitivity is determined by performing the test in both undisturbed and remoulded soil. The soil is remolded by rotating the vane a specific amount of times (Swedish Institute for Standards, 2020). Sensitivity is defined as the ratio of undisturbed shear strength to remoulded shear strength (Knappett & Craig, 2012).

The vane shear test has some limitations, especially in stiffer or overconsolidated soils. Since the test applies shear directionally, it may also be affected by anisotropy, meaning different strength in different directions (Hunt, 2005). Furthermore, the test gives discrete shear strength values at specific depths and it can be less effective for soils with layering or significant variation in strength over short vertical intervals (Larsson et al., 2007).

2.3.2 Cone Penetration Test

The Cone Penetration Test (CPT) is a widely used method were a probe is pushed down into the ground at a constant penetration rate, providing continuous measurements throughout the penetration. This provides the advantage of identifying variations in soil properties and detecting layers where these properties change (Larsson, 2015). Compared to methods that provide discrete data points, such as the vane shear test this can give greater insight into the detailed soil profile.

The CPT method can be used in a range of different soil types, from fine grained cohesive soils to more sandy soils with occasional gravels. The limit is usually where it is possible to drive down the cone without hammer blows or rotation (Larsson, 2015). The probe is cylindrical with a conical tip of 60 degrees. When the probe is pushed into the soil, it measures two main parameters: cone tip resistance q_c and sleeve friction f_s. Additionally, the pore pressure, u_2,is often measured behind the cone tip. When this is included the method is called CPTu or piezocone (Lim et al., 2020). The measured cone tip resistance q_c is often corrected for pore pressures u_2 with Equation 2.8 and reported as total cone resistance, q_t. The factor a is the net area ratio (Hunt, 2005).

$$q_t = q_c + (1 - a) \cdot u_2 \tag{2.8}$$

where
q_t = Total cone resistance [kPa]
q_c = Measured cone tip resistance [kPa]
a = Net area ratio [-]
u_2 = Measured pore pressures [kPa]

From the measurements the undrained shear strength can be estimated using empirical correlations. Commonly Equation 2.9 is used which is based on corrected cone resistance, q_t, the total overburden stress, σ_{v0}, and the N_{kt} factor. $N_{kt}$is a correction factor typically in the range of 10 to 18 (Robertson & Cabal, 2022). Generally the value decreases as the soil sensitivity increases. On the other hand, the values of N_{kt}increases with increasing plasticity (Hunt, 2005).

$$c_{u,CPT} = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{2.9}$$

where
$c_{u,CPT}$ = Undrained shear strength from CPT [kPa]
q_t = Total cone tip resistance [kPa]
σ_{vo} = Total vertical is-situ stress [kPa]
N_{kt} = Correction factor [-]

In a Swedish context, where plasticity is rarely evaluated, the correction is instead based on liquid limit and OCR summarized in Equation 2.10 according to Larsson (2015).

$$c_{u,CPT} = \frac{q_t - \sigma_{v0}}{13.4 + 6.65 w_L} \left(\frac{OCR}{1.3} \right)^{-0.2} \tag{2.10}$$

where
$c_{u,CPT}$ = Undrained shear strength from CPT[kPa]
w_L = Liquid limit [%]
q_t = Total cone tip resistance [kPa]
σ_{vo} = Total vertical is-situ stress [kPa]
OCR = Overconsolidation Ratio [-]

As mentioned, the method has the advantage of continuous measurements compared to vane test where only discrete points are measured. Nevertheless, interpretation should be made with caution. CPT is sensitive to handling errors and the quality of the results depend on the execution (Larsson, 2015). Because of this, the undrained shear strength from CPT is recommended to be validated against other methods to verify the results.

2.3.3 T-bar test

The T-bar test is a type of full-flow penetrometer which is mainly used in offshore geotechnical investigations to assess soft clayey soils (Liu et al., 2019). During testing, a bar shaped like a "T" is pushed into the soil at a constant rate while the penetration resistance is measured. The T-bar test is similar to the CPT but is preferred in very soft soils, where it engages a larger volume of soil and thereby can provide more accurate measurements of resistance (DeJong et al., 2011). Unlike the CPT, which primarily displaces soil, the T-bar allows soil to flow around the bar, reducing the impact of anisotropic stress conditions on the penetration resistance. Furthermore, the total vertical stress does not have to be accounted for in the evaluation (Lunne et al., 2005). The undrained shear strength from T-bar test can be calculated with Equation 2.11

$$c_{u,T-bar} = \frac{q_{T-bar}}{N_{T-bar}} \tag{2.11}$$

where $c_{u,T-bar}$ = Undrained shear strength from T-bar test[kPa]
q_{T-bar} = Measured resistance [kPa]
N_{T-bar} = T-bar factor [-]

The value of N_{T-bar} depends on the roughness of the T-bar, where analytical analysis gives the values of 9.14 for a fully smooth, and 11.94 for fully rough interface (Randolph & Houlsby, 1984). Usually 10.5 is adopted as a value to be used if no further information is available.

An advantage of the T-bar test is the ability to assess remoulded shear strength through cyclic penetration. By repeatedly inserting and extracting the bar, the soil structure is disturbed and the degradation curve, derived from these cycles, provides information about the soil sensitivity (Einav & Randolph, 2005). The number of cycles required to reach a stable remoulded shear strength value varies with soil sensitivity, but 10 cycles is a commonly used value in offshore geotechnical practice. (Yafrate et al., 2009).

2.4 Laboratory methods to test shear strength of soil

Results from laboratory test can be more accurate than in situ methods, however the quality is also relying on the quality of the soil specimen. Correct collection and handling of the soil specimen is of greatest importance to achieve reliable test results (Larsson et al., 2007). Additionally even if the accuracy is high, the small sample size limits the gained information.

2.4.1 Fall cone test

The fall cone test is widely used in Scandinavian geotechnical engineering for determining the undrained shear strength of fine-grained soils (Swedish Geotechnical Society, 2018). Internationally the method is primarily regarded as an index test rather than a direct measure of soil strength (Sällfors et al., 2017). However, when used to determine the liquid limit, the fall cone test is a preferred method worldwide, as it provides consistent and repeatable results compared to other methods (Koumoto & Houlsby, 2001).

In the fall cone test, a cone with a specific angle and weight is released from a position where it just touches the soil surface. Typically the cones are of weight 10 g, 60 g, 100 g or 400 g, where the two lighter have an angle of 60 degrees and the heavier of 30 degrees (Swedish Geotechnical Society, 2018). The reason for the variety of cones is to allow the method to be used in soils of various types. Heavier cones are required for stiffer soils, and lighter for softer soils. When released, the cone penetrates the soil, and the depth of penetration is recorded and used in empirical equations to estimate the undrained shear strength, see Equation 2.12. The value of K varies depending on soil plasticity and has been calibrated based on comparisons with the vane shear test (Larsson et al., 2007).

$$c_{u,fc} = \frac{K \cdot mg}{i^2} \tag{2.12}$$

where
$c_{u,fc}$ = Undrained shear strength from fall cone [kPa]
K = Cone factor [-]
m = Weight of cone [g]
g = Gravitational acceleration, 9.82 [m/s^2]
i = Cone penetration depth [mm]

Both undisturbed and remoulded samples can be tested, allowing for the derivation of soil sensitivity and the liquid limit. The correction factor, μ, often applied to fall cone results, is derived from empirical relationships between the liquid limit and results from alternative strength tests (Larsson et al., 2007).

Although the test setup is relatively simple, achieving reliable results with the fall cone relies on the execution. Proper positioning of the cone at the soil surface before release and precise measurement of penetration depth are critical (Swedish Geotechnical Society, 2018). Even small errors can significantly impact the calculated shear strength, particularly in very soft soils. The test is also highly sensitive to shallow penetration depths, where minor misreadings causes larger errors. Furthermore, the fall cone test becomes less reliable at depths below 15 m, as it has not been calibrated for such conditions (Larsson et al., 2007).

While the fall cone test is reliable for clayey soils, its performance is less accurate in silt and sand layers, where penetration resistance tends to be overestimated (Larsson et al., 2007). Furthermore, in stiffer soils a heavier cone is required to achieve a measurable penetration depth, but the fall cone method has not been calibrated for such conditions.

2.4.2 Direct simple shear test

The Direct Simple Shear (DSS) test is a laboratory method used to determine the shear strength of fine-grained soils where a shear force is directly applied to a soil specimen (Hunt, 2005). The specimen is prepared by placing a rubber membrane and metal rings round the cylindrical sample to restrain it from deforming radially during the test and only allowing vertical deformation. The specimen is placed between two shear plates in the DSS apparatus, and the upper plate is connected to a displacement transducer, while the lower plate is typically connected to a load cell (Briaud, 2013). Firstly, the specimen is consolidated under a normal stress in the vertical direction just below the assumed preconsolidation pressure (Larsson et al., 2007). Secondly the vertical pressure is reduced to the magnitude of in situ conditions. This procedure is executed to simulate the soils previous state and reduce the possible disturbance and relaxation due to retrieval of the sample.

Once the specimen is consolidated, the upper plate is gradually displaced horizontally. The shear force required to cause failure in the soil is measured and the shear stress is calculated according to Equation 2.13. The applied shear force is divided by the shear area of the specimen, which is determined based on the specimen's dimensions (Larsson, 2004).

$$c_{u,DSS} = \frac{F}{A} \tag{2.13}$$

where
$c_{u,DSS}$ = Undrained shear strength from DSS test[kPa]
F = Force applied at failure [kN]
A = Cross sectional area of sample [m^2]

One of the advantages of the DSS test is its ability to simulate in situ conditions of soil under undrained shear more effectively than other methods (Briaud, 2013). The consolidation phase brings back the soil to the in situ conditions and the application of shear stress is applied directly. Furthermore, the results from the test doesn't require corrections with empirical factors as with the fall cone and vane shear test.

2.4.3 Triaxial test

The triaxial test is a laboratory method where the soil is tested under conditions that resembles its natural state the most (Briaud, 2013). The specimen is put in a chamber with a radial pressure from the liquid in the chamber, resembling the horizontal pressure in situ. Furthermore, a vertical load is applied, corresponding to vertical load in situ. Because the soil is exposed to conditions similar to its in-situ state, the behaviour during loading will be more accurate than for the other laboratory methods, producing more accurate test results (Knappett & Craig, 2012).

The triaxial test can be conducted under either drained or undrained conditions. For the drained case, the water is allowed to drain in the sample, implying that the pore pressure will not increase or decrease extensively (Knappett & Craig, 2012). Because of this the volume of the sample is allowed to change. In the undrained case, on the other hand, the water is not allowed to enter nor exit the specimen. Instead there will be a change in the pore pressure as the external pressure is increased and the volume constant (Westerberg et al., 2012).

The triaxial test is commonly conducted through two phases; consolidation phase and shearing phase. During the consolidation phase, axial and radial pressures are applied on the specimen, commonly resembling those assumed in situ (Westerberg et al., 2012). The strains during this phase is usually not of great interest, but can give an indication of the quality of the sample.

Secondly, the shear phase is introduced, which can be either compression or extension, where compression is the most commonly used in the industry (Knappett & Craig, 2012). During the compression triaxial test, the vertical load is increased with a constant deformation rate, forcing the specimen to expand radially until failure. During this shearing state the radial pressure is maintained constant. This setup resembles the active shearing directly under a load, see Section 2.1.

The passive shearing state, resembling increased horizontal pressure or reduced vertical load is simulated by performing the extension triaxial test (Westerberg et al., 2012). During this setup the sample is allowed to extended until failure by having a higher horizontal than vertical pressure.

To interpret the undrained shear strength, c_u, from triaxial test it is assumed to be the shear stress at failure of the specimen (Westerberg et al., 2012)

$$c_{u,Triax} = \frac{\sigma_1 - \sigma_3}{2} \tag{2.14}$$

where
$c_{u,Triax}$ = Undrained shear strength from Triaxial test [kPa]
σ_1 = Major principal stress [kPa]
σ_3 = Minor principal stress [kPa]

The triaxial test is costly and time consuming. Nevertheless, it is commonly used in the industry due to its ability to produce accurate results by replicating in situ conditions (Knappett & Craig, 2012).

2.4.4 Unconfined compression test

An unconfined compression test (UCT) can be regarded as a simplified triaxial test without the radial confining pressure ($\sigma_3 = 0$) (Knappett & Craig, 2012). A cylindrical soil sample is placed on a plate. The height of the specimen should be within the range of 9 - 12.5 cm to make sure that the top and bottom does not interfere with the shear plane as it propagates (Swedish Institute for Standards, 2018).

In the UCT, the top plate is fixed while a vertical load is applied by moving the bottom plate upwards. The typical strain rate is between 1% and 2% per minute, which corresponds to approximately 1-2 mm/min for a sample of 10 cm (Swedish Institute for Standards, 2018). A shear plane will typically propagate diagonally through the sample at failure.

The maximum vertical stress that the soil can withstand before failure in shear, is calculated by dividing the peak vertical load by the corrected cross sectional area, as shown in Equation 2.15. The undrained shear strength is then calculated using Equation 2.17.

$$\sigma_v = \frac{P}{A_0}(1-\varepsilon) \tag{2.15}$$

$$\varepsilon = \frac{d_f}{H} \tag{2.16}$$

$$c_{u,UCT} = \frac{\sigma_v}{2} \tag{2.17}$$

where

σ_v = Axial stress at failure [kPa]
P = Axial load at failure [N]
A_0 = Initial cross-sectional area of the specimen [mm^2]
ε = Axial strain at failure [-]
d_f = Axial deformation at failure [mm]
H = Initial height of the specimen [mm]
$c_{u,UCT}$ = Undrained shear strength [kPa]

Since no radial pressure is applied during testing, excess pore pressures could theoretically be dissipated, allowing some consolidation (Briaud, 2013). However, the test is considered to be undrained for fine grained soil because the test is performed rapidly, not allowing a significant drainage process.

3

CC-Test

The Clay Cutting Test (CC-Test) is a laboratory method developed to determine the undrained shear strength of soil samples. The aim of the method is to complement current and established methods. The test has a straightforward approach where a tensed wire is pushed through an undisturbed soil specimen while the resisting force is measured throughout the profile. Through an empirically derived equation, see Section 3.3, the undrained shear strength is calculated based on the measured resistance.

3.1 Setup of CC-Test

The setup consist of a wire with a round cross section, typically with a diameter of 0.25 mm. The wire is tensed in a holder that keeps it in place. With screws on the holder the tension in the wire is built up, minimizing its deflection when cutting the soil sample. The load cell is placed on the holder and measures the required force to penetrate the soil, see the setup in Figure 3.1.

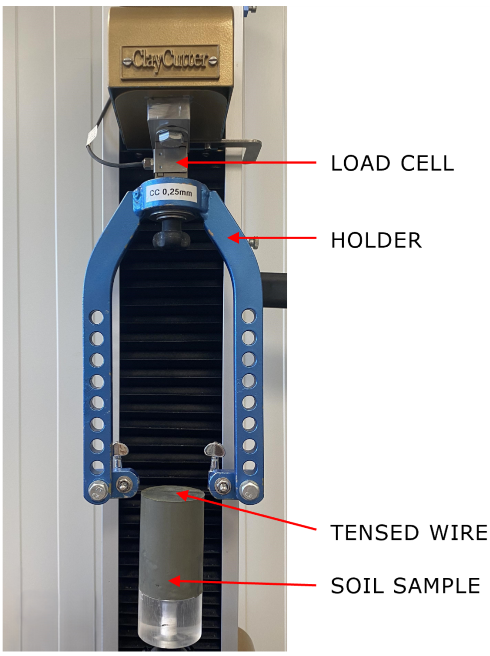


Figure 3.1: Setup of Clay Cutter

The wire is lowered to touch the surface of the soil specimen before starting the test. The penetration rate is 40 mm/min which is relatively fast compared to the speed of other laboratory tests such as UCT, which typically has a rate of 1 mm/min. However, previous systematic testing has shown that within the range of 20-60 mm/min the measured resistance remains consistent (Hedborg, 2024).

Rates slower than the given range tend to give increased resistance. A proposed explanation is that at lower speed the soil has time to collapse behind the wire and increasing the resisting force (Hedborg, 2024). Within the presented range of 20-60 mm/min, there will instead be a gap of air behind the thread allowing the thread to run through the sample without additional resistance.

The Clay Cutter has an internal displacement sensor that records the position of the thread over time. Based on these measurements the actual penetration rate can be calculated. When evaluating over a 1 second interval, the intended rate of 40 mm/min varies within ± 3 mm/min. When using a 2 second interval, the variability reduces to ± 1 mm/min. This is assumed to be accurate enough since it is well within the range of 20-60 mm/min where previous tests has shown little variation as mentioned previously.

To assess the mechanical repeatability of the setup, tests were performed on a melamine foam sponge, a homogeneous elastic reference material. As shown in Figure 3.2 these tests presented minimal variation between runs, indicating that the device itself produces stable measurements. However, this does not account for natural variability in soils, but provides confidence in the mechanical performance of the test equipment and measurement system.

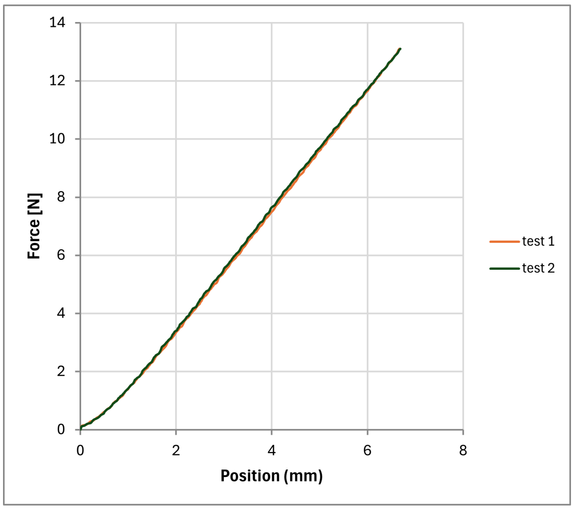
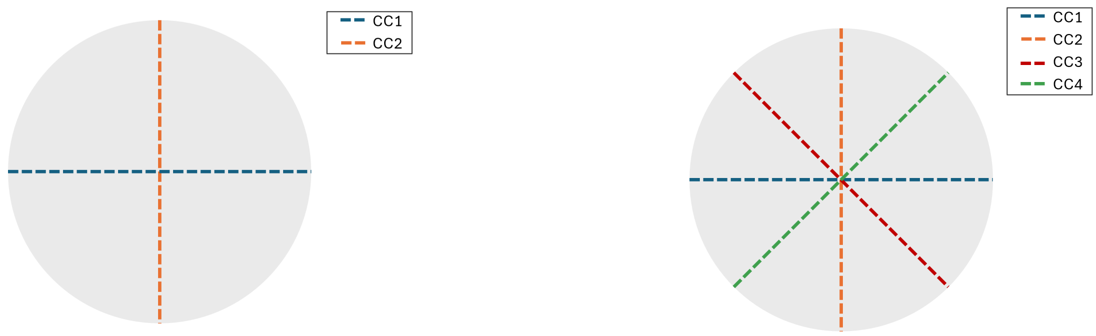


Figure 3.2: Repeatability test of the equipment in melamine foam, an elastic reference material.

3.2 Standard execution of CC-Test

The sample is prepared by being carefully extracted out of the sampling tube. A minimum sample height of 2 cm is required for testing, but heights of up to 10 cm can be used. During standard procedure the wire cuts the sample through the entire profile, the specimen is then rotated 90 degrees and cut again through the profile (Hedborg, 2025). The first cut is called CC1 and the second CC2, see Figure 3.3 for placement. Previous testing has shown that the initial penetration does not influence subsequent measurements and more information can be collected from a single specimen this way. Furthermore, two additional tests, CC3 and CC4 respectively, can be performed if rotated 45 degrees and then 90 degrees, resulting in a total of four cuts. The four tests are spread evenly with 45 degrees, see Figure 3.3.



(a) CC-Test with two cuts

(b) CC-Test with four cuts

Figure 3.3: Placement of CC-Test in the sample, seen from above.

3.3 Evaluation of CC-Test results

The undrained shear strength of the sample is evaluated by Equation 3.1, where the measured resistance is divided by a factor. The factor is set based on empirical testing through a "best-fit" approach based on roughly 400 CC-Tests compared to mainly the fall cone, but also DSS. For a thread diameter of 0.25 mm the factor was determined to be 19.63 (Hedborg, 2024). Since this value corresponds closely to half the surface area of the thread, or the assumed contact area with the soil, the factor in the equation is expressed as $0.5 * A_s$.

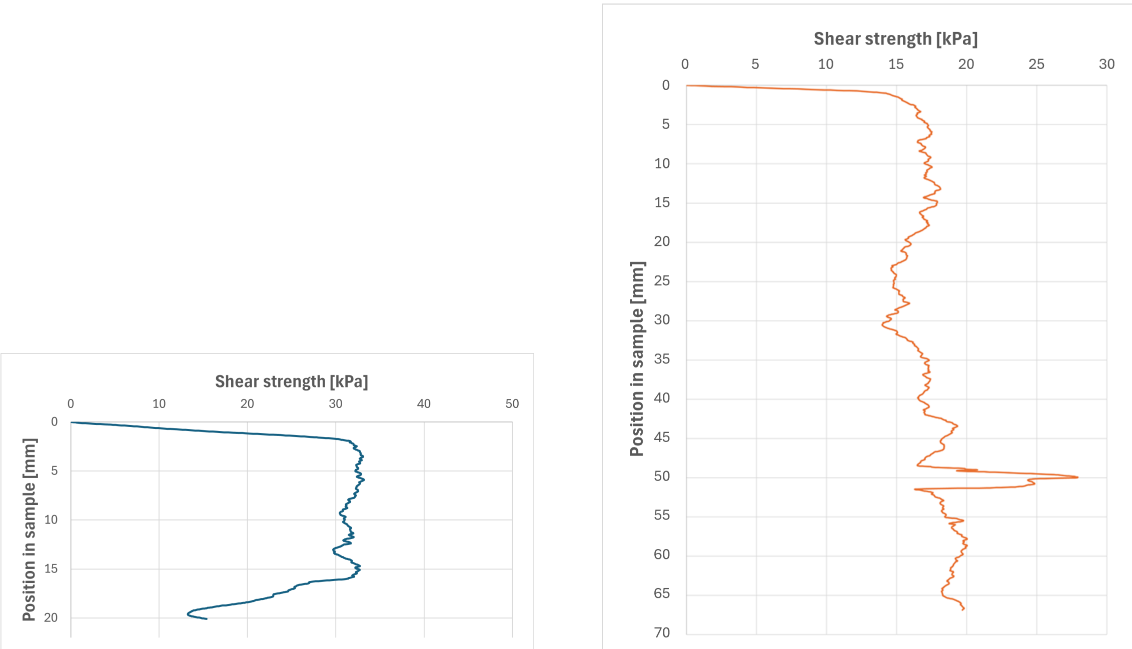
$$c_{u,CC} = \frac{F}{0.5 * A_s} \tag{3.1}$$

where
$C_{u,CC}$ = Undrained shear strength from CC-Test [kPa]
F = Measured resistance [kN]
A_s = Surface area of the thread [m^2]

CC-Test is performed on undisturbed samples as a complement to fall cone, especially suitable in layered soils where variations in properties may occur within small changes of depths. The CC-Test gives the strength profile of the sample by forcing the failure in a smaller area close to the thread, rather than creating a global failure in the soil sample. Because of this, the interpretation of the CC-Test should be done similarly as with the CPT, where the stratification and variation are of interest.

As the thread starts to penetrate the soil, there will be a gradual increase of resistance, where the thread builds up enough force to break the surface of the soil. During this build up the deformations are mainly elastic until the strength is build up to reach plastic deformation as plastic failure occurs, see beginning of curves in Figure 3.4. At the end of the test the base act as a backstop and therefore the last part of the measurement, when the thread gets close to the base, should not be considered when evaluating the curve. In stiffer soils, cracks may form in the lower part of the sample and because of this the required force to continue driving will be lower than the actual shear strength due to the previous failure, see end of curve in Figure 3.4 (a).

The middle part of the CC curve is often the part of interest. In a homogeneous and perfect sample this part presents a constant value, however in natural soil this is not the case as can be seen in Figure 3.4. Typically small changes or variations can be observed even if the natural soil is fairly homogeneous as in (a). The variations are more significant in soil with layers of different properties. If the thread touches shells, gravels or other objects in the soil, peaks may occur. An example is presented in (b) where a shell causes the significant peak at the end.



(a) Short test with little variability

(b) Long test with variability and a peak due to a shell in the soil

Figure 3.4: Examples of two different type curves measured with the CC-Test. The undrained shear strength is plotted against position in the sample measured from the top.

Currently, the average value of the middle part of the CC curve is regarded as the representative value of undrained shear strength for the test. This excludes initial build-up and effects near the base, as well as any peaks due to objects such as shells.

The CC-Test could also be performed on consolidated soil specimens after Triaxial test or DSS to validate the results as well as present variations in the specimens. Especially interesting and useful for soil samples where a distinct shear crack emerges in a part of the sample, a higher shear strength can be found in other areas providing more information of the soil.

3.4 Theoretical comparison of CC-Test with CPT and T-bar

The CC-Test is empirically based and its theoretical basis is not sustained. For this reason, comparing it to established penetration based methods, specifically CPT and T-bar, can help evaluate the possibilities and limitations of applying a similar theoretical framework to the CC-Test. The methods may have some differences, however they all share the core principle of relating a measured resistance during penetration to an estimate of undrained shear strength.

For both the CPT and the T-bar test, a measured resistance to penetration in the soil is related to the undrained shear strength through a factor, N_{kt} for CPT and N_{T-bar} for T-bar. Regarding CPT the factor is mostly empirically derived due to the complexity of failure around the cone. The T-bar on the other hand, provides a robust theoretically derived factor.

A key difference between the CPT and T-bar is the flow mechanism around the penetrating object. In CPT the soil experiences both compression and shear, resulting in complex failure mechanism. The T-bar allows the soil to flow around the cylindrical bar, simplifying the analysis by assuming that the soil behaves similarly above and below the bar. Because of this, the T-bar can be modelled as a cylinder submerged completely in the soil represented by a rigid plastic medium (Einav & Randolph, 2005). The factor mainly impacting the results will be the roughness of the T-bar surface. Through derivation of an upper bound and lower bound plasticity solution gives the values of N_{T-bar} to range from 9.14 for a fully smooth surface to 11.94 for fully rough surface (Randolph & Houlsby, 1984).

The CC-Test is generally not evaluated with an N-factor. Instead, the surface area of the wire in contact with the soil is used, rather than the conventional cross sectional area. This in turn implies that there is indeed a factor corresponding to the ratio of cross sectional area of the wire to the contact area with the soil. This would be similar to an N factor. The factor of N_{cc} would then be pi*0.5 or 1.57 for all diameters of thread.

Comparing that to the roughness factor for T-bar it is quite low. However, one important difference of the CC-Test is the fact that it assumes that the contact area is only half the surface area of the thread, compared to the full flow surrounding the T-bar. Due to the high penetration rate of the wire the soil doesn't have the time to flow fully. A gap of air is instead developed behind the thread as it penetrates the soil. This is a major difference impacting the approach. However, a roughness factor will still most probably be involved, even though it may not be possible to derive based in the same methodology as for the T-bar.

There are cases where the T-bar has a similar gap of air behind the bar and that is for shallow depths. Research by White et al. (2010) has shown that for shallow embedment, where only a part of the total surface area is in contact with the soil, the roughness factor is in fact lower than the lower bound plasticity solution of 9.14. The roughness factor increases exponentially from zero up to the factor of the fully submerged, with depth (White et al., 2010). The reason for this is the buoyancy effects and the different failure mechanism closer to the surface. These solutions are derived through large deformations finite element analysis and not the robust analytical solution for fully submerged.

4

Methodology

Two areas of interest is defined for the project where the first one is a data compilation of an archive of previous CC-Test in comparison to DSS and fall cone. In the second one, tests are made on soil samples, where the CC curves are analysed more in-depth to find out how it relates to other methods. Both parts contributes to the purpose of evaluating the CC-Test as a method of presenting the undrained shear strength of soil samples. An overview of the methodology is visually presented in Figure 4.1.

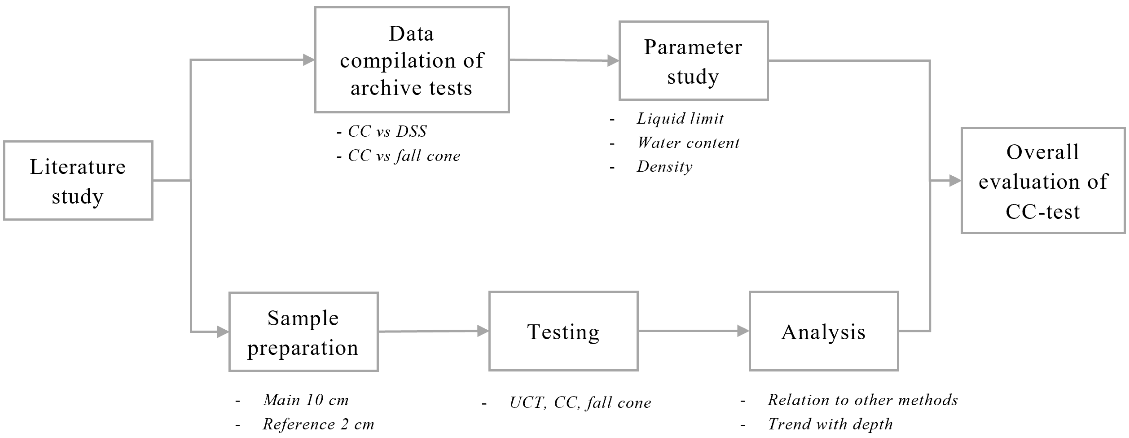


Figure 4.1: Overview of the methodology

4.1 Literature review

In order to better understand the area of study a literature review was conducted. The review focused on information regarding existing methods to test undrained shear strength as well as the theory behind each method. The aim was to provide an overview of existing testing methods as well as identify theories possible to strengthen the reliability of the CC-Test. Information was mainly gathered from online databases of scientific papers but also books in relevant areas.

Since the theoretical basis of the CC-Test is not well-established, a literature review also included the physical processes that could occur in the soil during testing. This includes soil deformation, shear failure and the forces acting on the cutting wire. Comparing the CC-Test with established testing methods and their theoretical foundation was the focus in order to distinguish both similarities and limitations.

4.2 Archive compilation of CC-Test

Previous results from an archive collection of CC-Test were compared to results from fall cone and DSS. All tests were conducted at AFRY Geotechnical Laboratory where the fall cone test and DSS test were executed according to standard execution. The available CC-Test were executed with different penetration rate and thread diameter since they were executed when the standard rate and thread diameter was not yet sustained. As the impact of rate is not included in the scope of this report, only tests with the sustained standard rate of 40 mm/s were included. Regarding thread diameter, multiple thread diameters were included in the analysis but the impact is also studied further in the report. The soil samples were from a total of 17 sites in the Gothenburg area. The soil types and number of samples for each soil type are presented in table 4.1. The soil type was determined by visual assessment.

Soil type	Abbreviation	No. of samples
CLAY	Cl	17
CLAY shells	Cl sh	1
gyttja-bearing CLAY	gyCl	9
somewhat gyttja-bearing CLAY	(gy)Cl	8
silty CLAY	siCl	18
gyttja-bearing silty CLAY	gysiCl	23
somewhat gyttja-bearing silty CLAY	(gy)siCl	3
gyttja-bearing sandy CLAY	gysaCl	3
gyttja-bearing sandy CLAY sh	gysaCl sh	1
somewhat sandy CLAY	(sa)Cl	1
GYTTJA	Gy	1
clayey GYTTJA	clGy	9
silty GYTTJA	siGy	1
gyttja-bearing SILT	gySi	1
clayey SILT	clSi	5
gyttja-bearing sandy SILT	gysaSi	1

Table 4.1: Soil types, abbreviations, and number of samples for tests included in the archive compilation

For all soil samples CC1 and CC2 were available, see Section 3.3 for explanation of the execution. For some samples CC3 and CC4 were also available. All soil samples were tested with the fall cone and for some, DSS results were also available. The total number of tests from the different methods are presented in Table 4.2.

Test method	Number of tests
CC-Test	290
fall cone	102
DSS	41

Table 4.2: Number of tests included in the archive compilation for each method

Both DSS and fall cone gives a single value of shear strength whereas the CC-Test provides the variability of shear strength throughout the sample. Variations may be due to layers in the soil and variation of its properties, whereas larger peaks will be registered when objects such as shells are present in the sample. If peaks are due to objects, the registered force and in turn the undrained shear strength will be misleadingly high.

To avoid this in the comparison, a representative part of each CC-Test was chosen. The representative part of the test was defined as a 10 mm or longer section of the sample that did not contain peaks due to shells or larger grains such as gravel. The determination of whether the peaks were due to shells or larger fractions were based on pictures of the cross section when available, or if mentioned in the laboratory protocol. Variations due to other soil properties were not excluded. For the cases where a representative part was not possible, they were excluded from this comparison. A total number of 8 CC-Tests were excluded due to this.

As multiple curves were available for each tested specimen, an average of all the curves were evaluated as a single representative value. The representative value of undrained shear strength from CC-Test was plotted against the value from fall cone test and DSS test in separate diagrams. Furthermore the undrained shear strength from CC-Test was normalized with the results of fall cone test and the DSS test respectively. The normalized values are presented against different soil properties such as density, liquid limit and water content.

The impact of thread diameter on the measured strength was also studied in the same way. The normalized shear strength was presented against the thread diameter used in the CC-Test. This to highlight the impact of thread diameter on the measured strength.

4.3 In-depth analysis of strength profiles

A systematic in-depth analysis of 12 undisturbed soil samples from three different sites was conducted. To complement the archive compilation, which is dominated by soft soil with undrained shear strength below 25 kPa, stiffer soils were selected for this study. The samples used in the analysis were from depths 7-35 m. Three methods; UCT, CC-Test and fall cone, were all tested in the same specimen of soil to avoid deviating values due to natural variation of the soil.

Site	Depth [m]	Soil type
A	8	sulphide mottled CLAY
	12	sulphide mottled CLAY
	15	sulphide mottled silty CLAY
B	10	sulphide mottled CLAY
	12	sulphide mottled CLAY
	20	sulphide mottled CLAY
	24	sulphide mottled CLAY
	30	sulphide mottled CLAY
	35	sulphide mottled CLAY
C	7	CLAY
	8	silty CLAY
	10	CLAY

Table 4.3: Sample distribution for in-depth study by site, depth, and soil type

Firstly a 10 cm sample was prepared and tested with UCT. To not disturb the sample too much, only a maximum of 3% strain was allowed. All samples reached the peak shear strength within that limit and a clear shear crack was created, which was the information of interest in this case. The appearance of the shear cracks was documented through photographs.

After the UCT, the CC-Test was conducted in the same soil sample. CC1 was placed with the shear-plane, and CC2 was rotated 90 degrees, see Figure 4.2. Pictures were taken of the cross-section after cutting to present any visual variations of the soil.

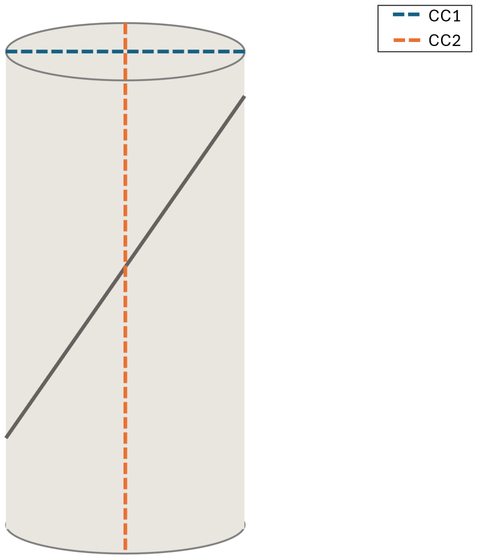


Figure 4.2: Placement of CC1 and CC2 in the soil specimen in relation to the shear plane, marked with the grey line.

A third method, the fall cone was executed last on the same specimen. A total of six measurements, were taken in three different levels of the specimen, see Figure 4.3 for placement in the sample. The six measurements were combined to an average for the sample. The water content was recorded for each of the three planes where the fall cone was released. This could further highlight variations within the sample.

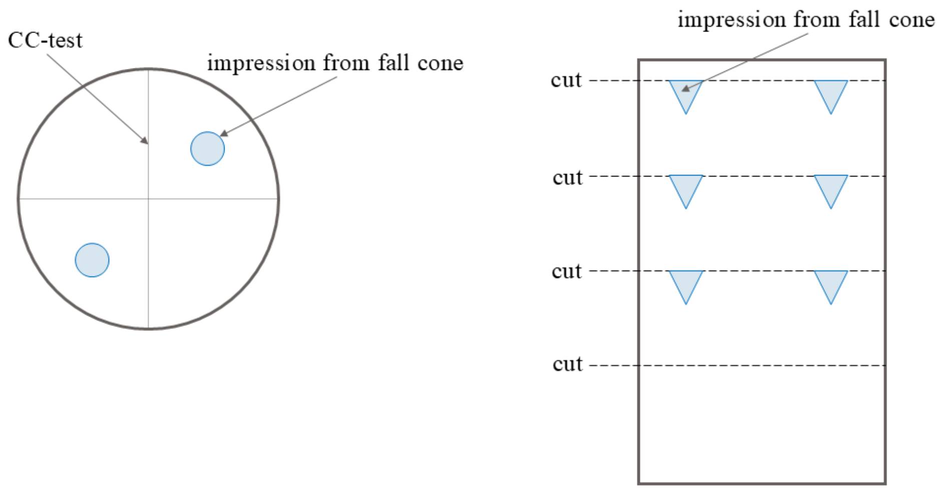


Figure 4.3: Placement of fall cone tests in the soil specimen after UCT and CC-Test from above and from the side.

Because of the low strain limit in the UCT compared to standard execution, the samples were assumed to have similar properties as before the UCT. However, to sustain this an additional 2 cm sample was prepared from the same tube as the 10 cm sample. For Site B, not enough material was available for this additional testing so the results are only available for Site A and Site C. In the 2 cm sample, only the CC-Test was performed. These additional results were compared to CC-Test from the previously UCT-tested samples to confirm that prior tests did not significantly alter the soil behaviour.

The results from the three methods were analysed in-depth to find systematic behaviour of the CC-Test in relation to the two other tests. Furthermore this also provided greater insight in how to evaluate the test results from CC-Test.

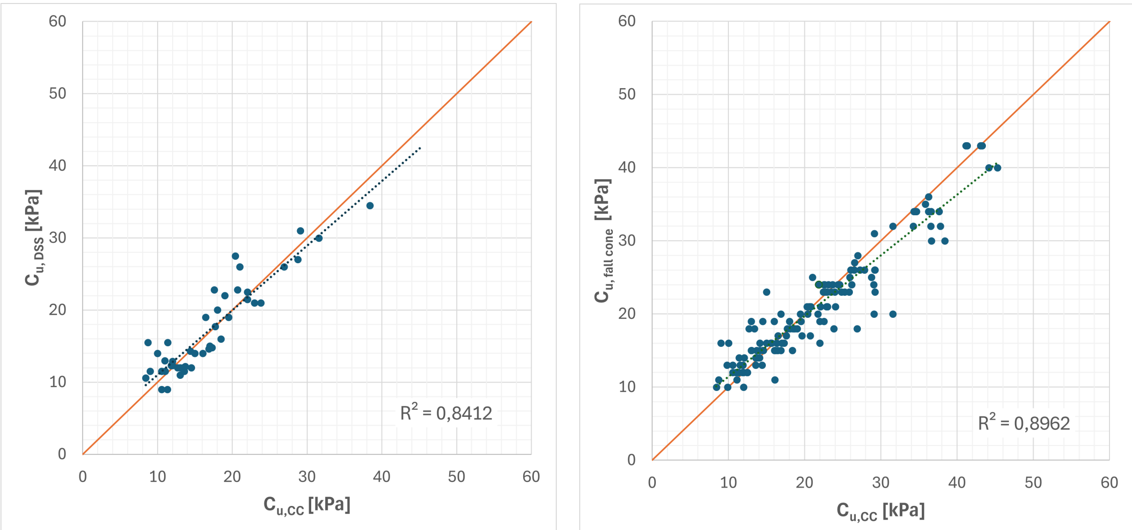
5

Results

This chapter presents the results from the compilation of previous CC-Tests, DSS and fall cone tests. Furthermore it includes the in-depth study of CC-Tests from Site A, Site B and Site C which are compared to results of fall cone and UCT.

5.1 Comparative testing on archive data

The representative value of undrained shear strength obtained from the CC-Test are compared to those from DSS and fall cone respectively, see Figure 5.1. In the figure the orange line represents a perfect, 1:1 correlation between the compared methods. The dotted blue line represents the linear trend of the data.



(a) DSS versus CC-Test

(b) Fall cone versus CC-Test

Figure 5.1: Comparison of shear strength values from the CC-Test with (a) DSS and (b) fall cone. Each point represents the value of undrained shear strength from the two methods. The orange line indicates a 1:1 ratio and the dotted blue line the linear trend of the data.

As can be seen in the figure, the results from CC-Test show a satisfactory agreement with both the DSS and the fall cone. The trend lie close to the 1:1 reference line, implying that the CC-Test provides comparable values of undrained shear strength to the established methods.

Some deviation between the methods are expected due to the natural variability of soil. Regarding the fall cone, it is well known that its reliability decreases at depths below 15 m, see Section 2.4.1. This limitation partly explains why the values of CC-Test tends to yield higher shear strength values compared to the fall cone at strengths above 25 kPa, as the shear strength generally increases with depth.

In Figure 5.2 a reference plot is presented comparing the values of undrained shear strength obtained from the fall cone and DSS. The deviation between the methods is greater in the reference plot than in the corresponding comparisons that include the CC-Tests, especially at shear strengths above 20 kPa where the DSS consistently yields higher values than the fall cone.

The coefficient of determination, R^2, indicates that the data in Figure 5.1 shows a stronger correlation than that in Figure 5.2. This suggests that the CC-Test shows more consistent agreement with the established methods fall cone and DSS, for estimating undrained shear strength.

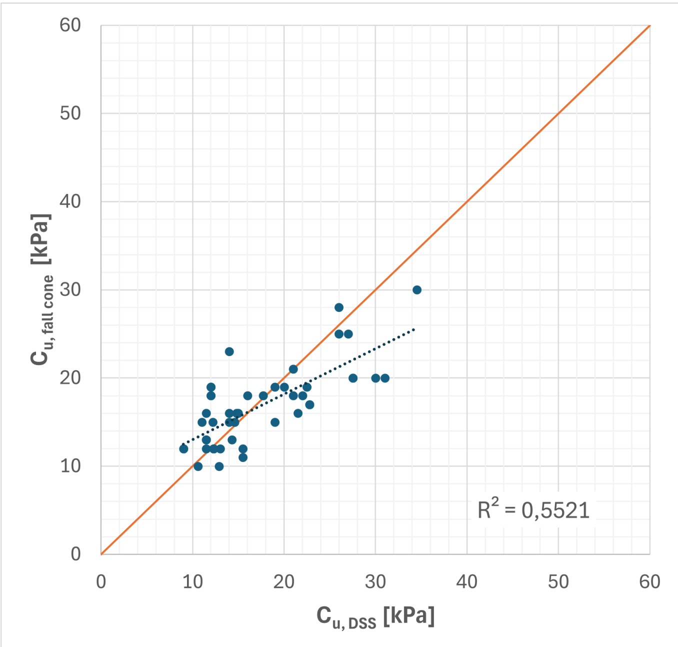
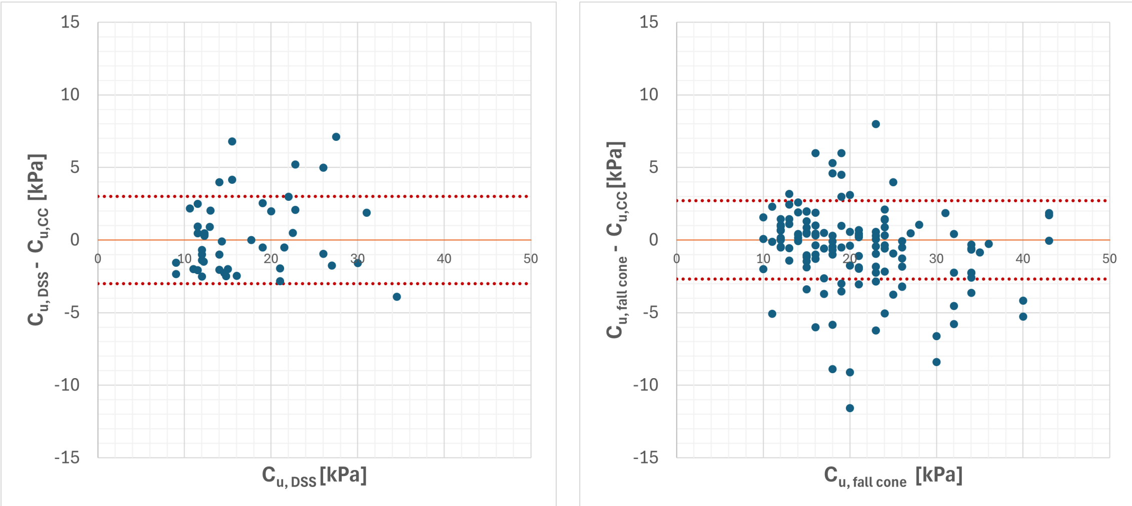


Figure 5.2: Reference plot of undrained shear strength obtained from DSS and fall cone. Each point represents the value of undrained shear strength from the two methods. The orange line indicates a 1:1 ratio and the dotted blue line the linear trend of the data.

The standard deviation of the difference between CC-Test and the DSS is 3.0 kPa, while the difference between the CC-Test and fall cone is 2.7 kPa. As the mean value of the undrained shear strength measurements is 22 kPa, these values corresponds to coefficients of variation of 14% and 12% respectively. Figure 5.3 presents the difference between the CC-Test and DSS or fall cone plotted against the corresponding values from the mentioned methods. In the plot, the standard deviation is presented as dotted red lines.



(a) DSS as reference

(b) fall cone as reference

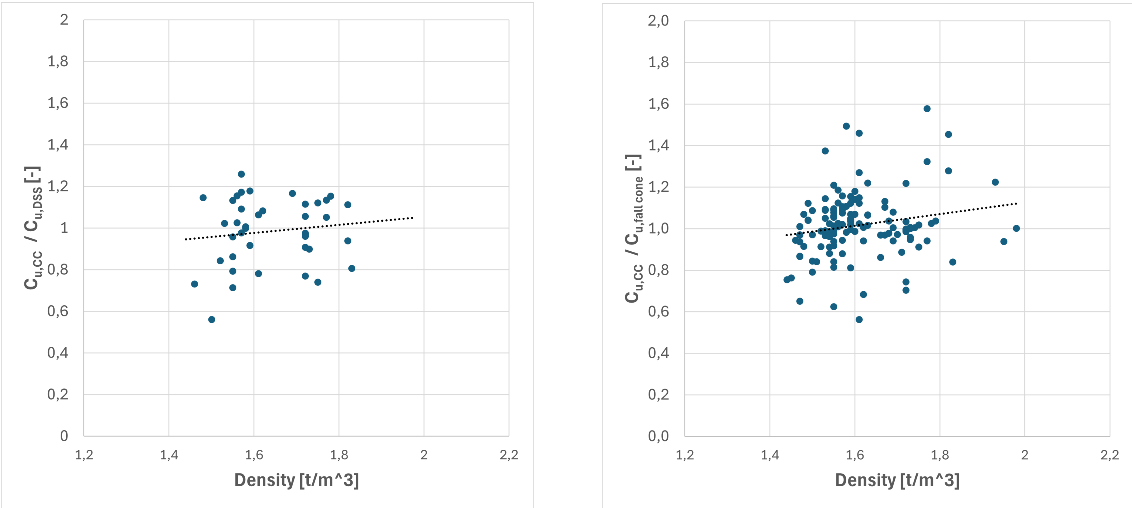
Figure 5.3: Comparison of the shear strength from the CC-Test with (a) the fall cone test and (b) the DSS test. Each point represents the difference between methods against the strength from the reference method. The red dotted lines are the standard deviation.

As observed in Figure 5.3 deviations beyond the standard deviation for the DSS are typically positive, while those for fall cone are mainly negative even if positive occurs as well. This indicates that the CC-Test tends to yield higher values than the fall cone and lower values than the DSS. In other words, the values obtained from the CC-Test is positioned between the two.

The DSS is regarded as a more reliable method for determining shear strength than the fall cone, especially at higher strengths. Therefore it is favourable that the CC-Test provides more conservative results in comparison to the DSS and not overestimate the strength of the soil.

5.2 Parameter study

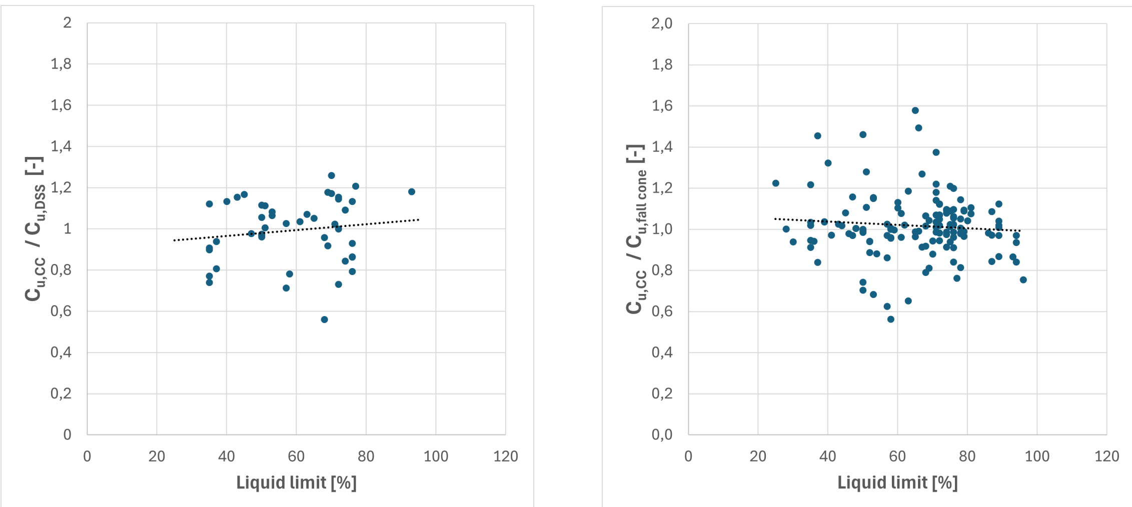
The undrained shear strength from CC-Test, normalized with fall cone and DSS respectively, is presented against different soil characteristics. In Figure 5.4, density is plotted against the normalized shear strength for fall cone and DSS respectively. The dotted line represents the trend line for the data. When fall cone is used as reference the general trend shows that higher density corresponds to higher shear strength measurements with the CC-Test. Within the density range of 1.5-1.8 t/m^3, which is the common range for most soils, the deviation is below $\pm$ 10%. However, for the DSS as reference method the observed trend is less pronounced and the deviation of the trend is $\pm$ 3% in the common range of soils.



(a) CC-Test normalized with DSS **(b)** CC-Test normalized with fall cone

Figure 5.4: The shear strength values from the CC-Test normalized by (a) DSS and (b) fall cone, versus density. The dotted blue line represents the linear trend.

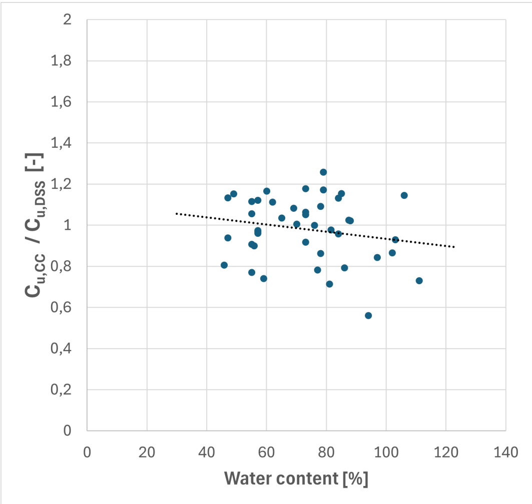
The liquid limit, w_L, versus the normalized shear strength is plotted in Figure 5.5. Only slight trends are presented, opposite depending on the reference methods. With DSS as reference method, a higher liquid limit corresponds to a higher measured shear strength with the CC-Test. In contrast, the fall cone shows the opposite trend where a higher liquid limit corresponds to a lower measured shear strength. The deviation is minor, below $\pm$ 3% for the common range of liquid limit within typical soils, indicating limited impact on measurements. Due to the weak and opposing trends, no clear correlation can be established.



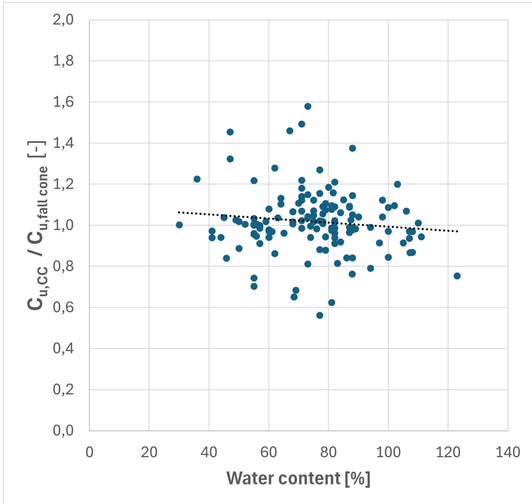
(a) CC-Test normalized with DSS **(b)** CC-Test normalized with fall cone

Figure 5.5: The shear strength values from the CC-Test normalized by (a) DSS and (b) fall cone, versus liquid limit (w_L). The dotted blue line represents the trend.

The water content, w_N, versus the normalized undrained shear strength is presented in Figure 5.6. The same trend can be observed with both fall cone and DSS as reference, although it is more pronounced with DSS as reference. The general trend shows that a higher water content corresponds to a lower measured value of shear strength with the CC-Test. Within the typical range of water content in soils, the deviation is roughly ± 5%.



(a) CC-Test normalized with DSS



(b) CC-Test normalized with fall

Figure 5.6: The shear strength values from the CC-Test normalized by (a) DSS and (b) fall cone, versus water content (w_N). The dotted blue line represents the trend.

The parameter study indicates that both density and water content provide consistent trends, when both DSS and fall cone is used as reference. In general, higher density and lower water content corresponds to higher measured shear strength values in the CC-Test. On the other hand, the liquid limit shows weak and opposing trends depending on the reference method, and no clear correlation can be established.

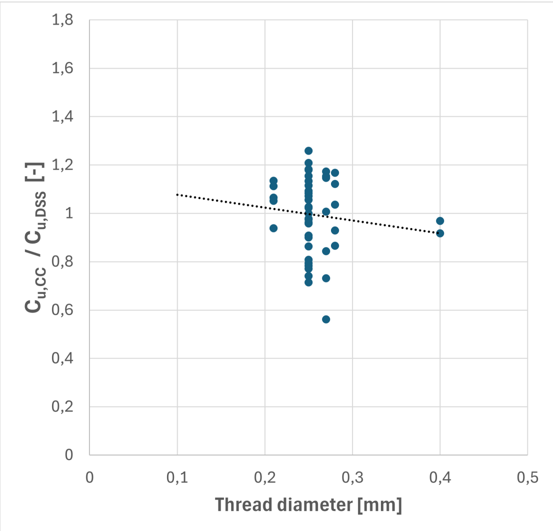
These observations align with the findings in the study by Valadão Junior et al. (2014), who investigates the influence of density and water content for the measured resistance in CPT. In the study it is concluded that soils with higher density and lower water content generally present greater resistance during penetration. That correlates well with the results in this report, as higher measured resistance corresponds to higher obtained shear strength values.

5.3 Thread diameter impact

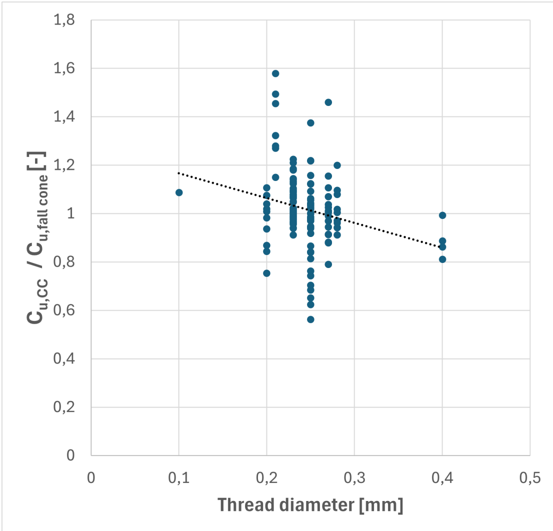
The diameter of 0.25 mm is regarded as the standard thickness for the wire of the CC-Test. However, since threads of multiple thicknesses are used in the archive tests, the impact of the thread diameter was studied. The results are presented in

Figure 5.7. As can be clearly observed, a larger thread diameter corresponds to lower measured undrained shear strength of the CC-Test. Whereas a smaller thread diameter presents a higher measured undrained shear strength compared to both fall cone and DSS.

As the data is not evenly distributed among the different thread thicknesses, the result may be somewhat misleading. However, since the trend for both reference methods imply similar trends, this pattern likely reflects the true behaviour.



(a) CC-Test normalized with DSS



(b) CC-Test normalized with fall cone

Figure 5.7: The shear strength values from the CC-Test normalized by (a) DSS and (b) fall cone, versus thread diameter. The dotted blue line represents the trend.

Comparing to studies investigating the effect of cone size for the CPT the results present similar findings. In the study by Valadão Junior et al. (2014), it was found that larger diameter cones measures lower resistance, and smaller cones measure higher resistance. The reason to this is explained by the fact that the CPT creates cracks ahead of the cone and for a larger tool the emerging cracks are larger. This would allow the cone to travel through soil with existing cracks for a longer distance and thereby presenting less overall resistance.

A similar mechanism could explain the trend observed in the CC-Test. As the thread diameter increases, larger cracks are formed ahead of the thread, allowing the thread to go through soil with existing cracks for a longer distance. This would in turn generate lower resistance measurements.

It is also important to consider the ratio of thread diameter to grain size of the soil. As the thread diameter increases it gets closer to the grain size of the soil which likely impacts the failure mechanism and the measured resistance. For instance, if the thread diameter is doubled, from 0.2 mm to 0.4 mm, the ratio of thread diameter to grain size will also change. In the case of clay, with a maximum grain size of 0.002 mm, a 0.2 mm thread corresponds to about 100 times the grain size

(1%), whereas a 0.4 mm thread, corresponds to about 400 times the grain size (0.5%).

On the other hand, for silt, where the grain size is maximum 0.06 mm, the ratio is significantly different. A 0.2 mm thread corresponds to 3.3 times the grain size (30%), and a thread of 0.4 mm thread corresponds to about 6.7 timed the grain size (60%). This change in ratio likely influence how the thread interacts with the soil structure, and impacts the measured resistance.

5.4 In-depth analysis of strength profiles

In the in-depth study, UCT, CC-Test and fall cone are executed in the same soil specimen. UCT stress-strain curves as well as pictures of the specimen before and after UCT is presented in Appendix. The test data from Site A can be found in Appendix A, Site B in Appendix B and Site C in Appendix C.

5.4.1 Site A

The values of undrained shear strength from UCT, fall cone and CC-Test is compiled in Figure 5.8 for the three samples from Site A with depth. The average value of the CC curve during the steady state is evaluated, as well as the upper and lower quartile to present the variation. No overall trend can be found regarding how the average from the CC-Test relates to the other methods as it varies from being the highest of the three, to the lowest.

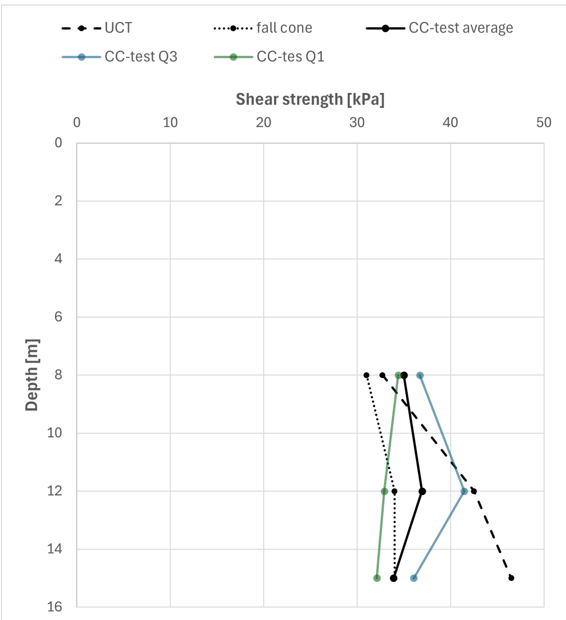


Figure 5.8: Results of shear strength from UCT, fall cone and CC-Test plotted with depth for Site A. The average value of the CC-Test as well as the upper and lower quartile (Q3 and Q1 respectively) are presented.

The interquartile range (IQR) is the difference between Q3 and Q1, or the range of which 50% of the data lie in. The IQR for Site A does not follow a clear trend.

The entire strength profile for the CC-Test as well as UCT and fall cone for the 8 m sample are presented in Figure 5.9. The reference CC-Test, tested in an additional specimen from the same tube, is plotted below the main CC-Tests in the figure to represent its actual placement in the sample tube.

The value of shear strength obtained with the fall cone is generally lower than the CC-Test. On the other hand, the value from UCT represents approximately an average value for the later part of the CC-Test. For the remaining sections of the profile, the UCT value is lower than the CC-Test. In the middle of the sample there is an area of higher strength, however that does not seem to have a large impact on the overall strength obtained with the UCT. Instead the last part of the strength profile, with lower strength, controls the failure.

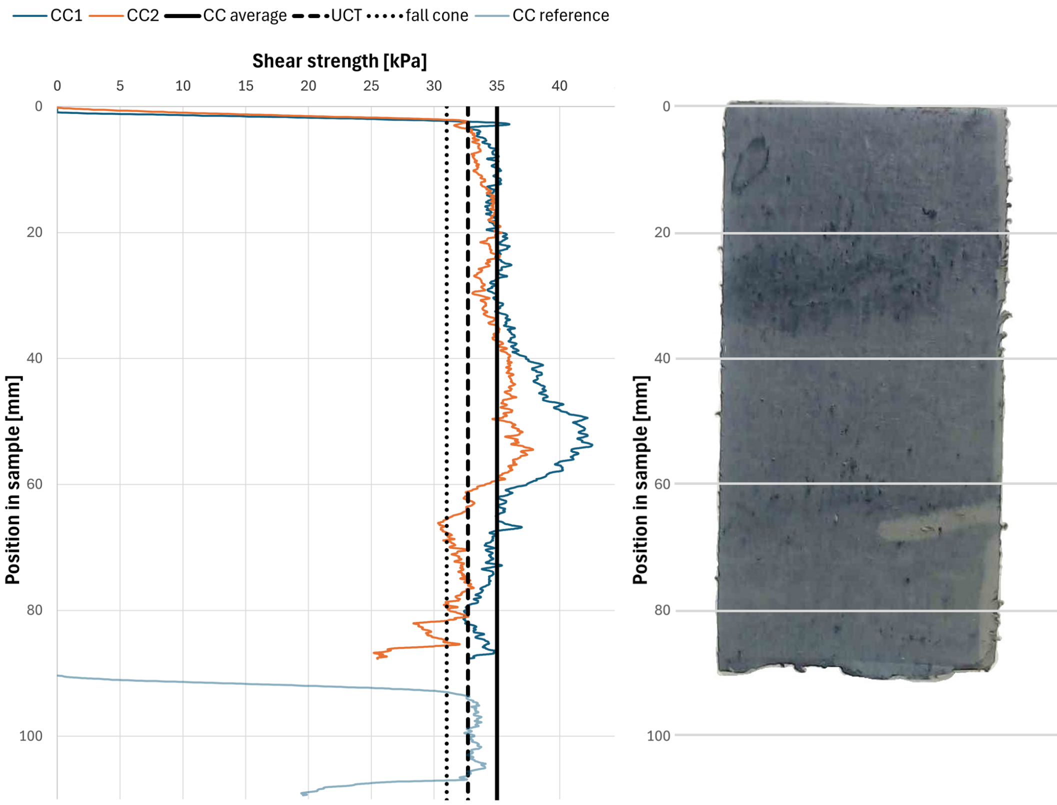


Figure 5.9: Shear strength profiles for the sample from 8 m depth at Site A, with position measured from the top of the sample. The reference CC-Test is plotted below the main CC curves, corresponding to the actual placement in the sample tube. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

In Figure 5.10 the sample from 12 m is presented. For this sample the undrained shear strength from the fall cone matches the parts of lower strength of the CC curve. The UCT however, finds higher values of the CC curve. The area in the middle of the sample with lower strength is not as included in the overall strength obtained with the UCT. Again the areas close to the top and bottom of the specimen is where the UCT and CC-Test intersects. The strength in the middle of the sample, whether higher or lower than at the ends, does not seem to significantly impact the shear strength measured with the UCT.

This might have to do with the formation of the shear plane. For both the 8 m and 12 m samples, the position of the initiation of the shear plane, measured from the top of the specimen, correlates well with the positions of where the UCT and the CC-Test intersects. This suggests that the UCT is governed mainly by the local shear strength at the diagonal failure plane, rather than the minimum from the vertical strength profile obtained with the CC-Test.

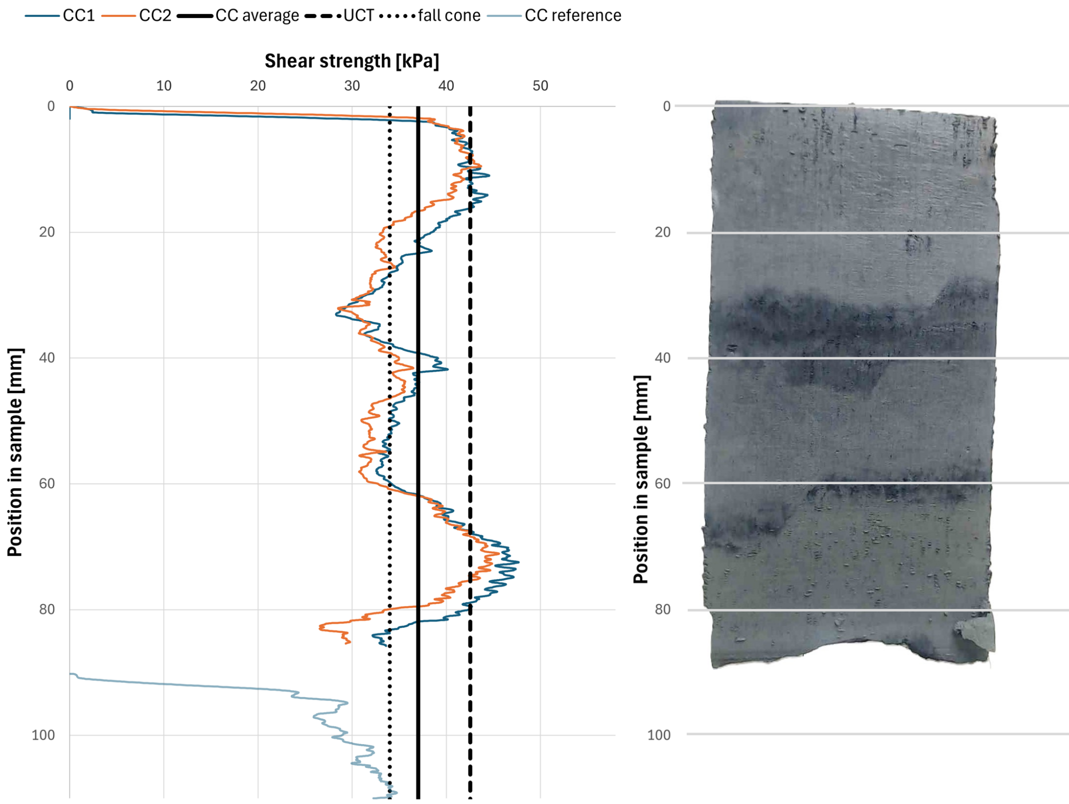


Figure 5.10: Shear strength profiles for the sample from 12 m depth at Site A, with position measured from the top of the sample. The reference CC-Test is plotted below the main CC curves, corresponding to the actual placement in the sample tube. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

In Figure 5.11 the 15 m sample is presented. The average of the CC-Test is very close to the value of the fall cone as could be observed already in Figure 5.8. This is the reason why you the plotted line of fall cone is not visible in 5.11. Furthermore, the UCT and CC-Test never intersect. This behaviour is different from the two previous depths where the fall cone generally corresponds to the lower measurements with the CC-Test and the UCT intersects the CC curve at some point.

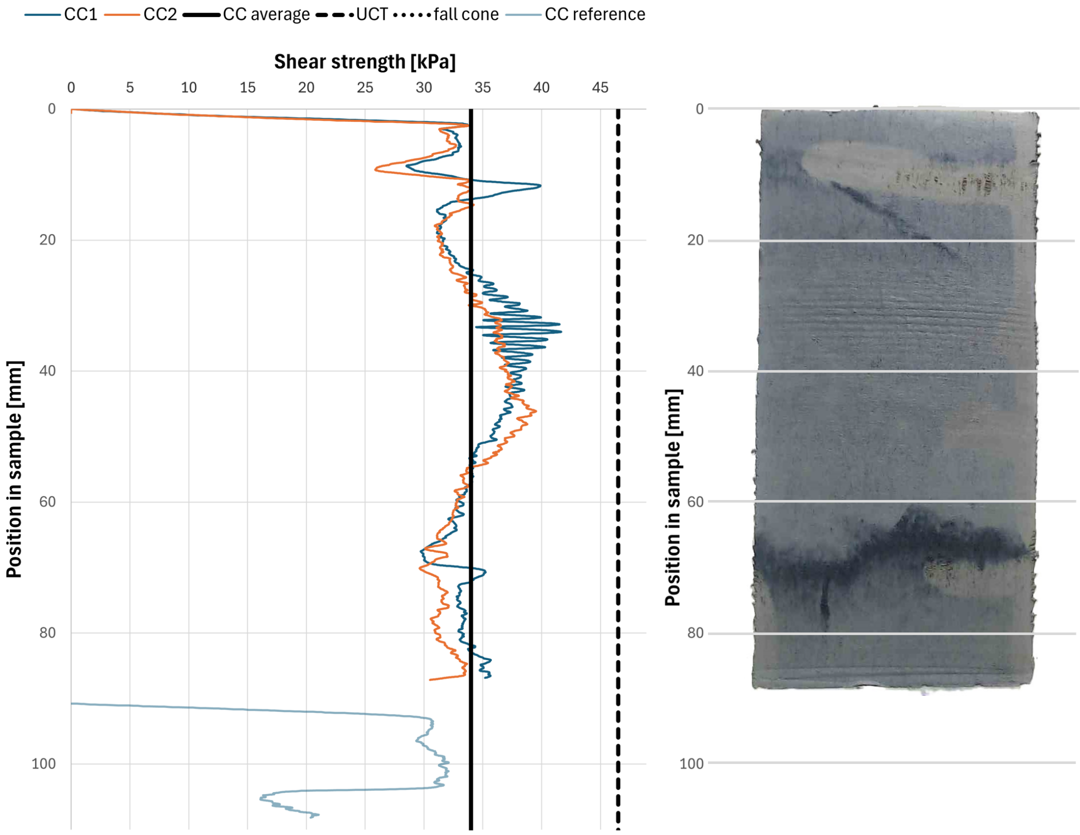


Figure 5.11: Shear strength profiles for the sample from 15 m depth at Site A, with position measured from the top of the sample. The reference CC-Test is plotted below the main CC curves, corresponding to the actual placement in the sample tube. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

It is observed that the 15 m sample had a horizontal crack initially, situated close to the top, see Figure A.6 in Appendix A. During the UCT the horizontal crack becomes more pronounced, but a diagonal shear plane is initiated as well. From the cross section in Figure 5.11 it is evident that in the top part there is an area with a different colour, possibly indicating a different material. This observation is supported by the water content from the three levels of the fall cone, see Table A.1 in Appendix A. The top piece has a lower water content, 47% compared to 60 % and 56% for the middle and bottom level, respectively, which indicates a coarser material.

During the UCT this upper material may behave differently than the rest of the sample, allowing more redistribution of grains and in turn have an effect on the measured strength. The 15 m sample is stated to be sulphidic silty clay, whereas the other depth is sulphidic clay, based on visual assessment. The silt may impact the measurements with the CC-Test and be one contributing reason to why the measurements are lower than UCT.

When evaluating the reference CC-Test, conducted in an additional piece of soil below the main sample, no indication that the soil was disturbed by the UCT can be found. The CC-Test prior to UCT generally follows the same shape as the main CC-Test, thus sustaining its accuracy.

5.4.2 Site B

The samples from Site B consist of six samples and the compilation are presented in Figure 5.12. Overall the samples are sulphide-bearing clay. For this site the average value from the steady state part of the CC-Test is consistently higher than the fall cone but lower than the UCT. With depth the average value from the CC-Test deviates more away from the UCT and towards the fall cone. Furthermore, the upper quartile is for the most part lower than the UCT.

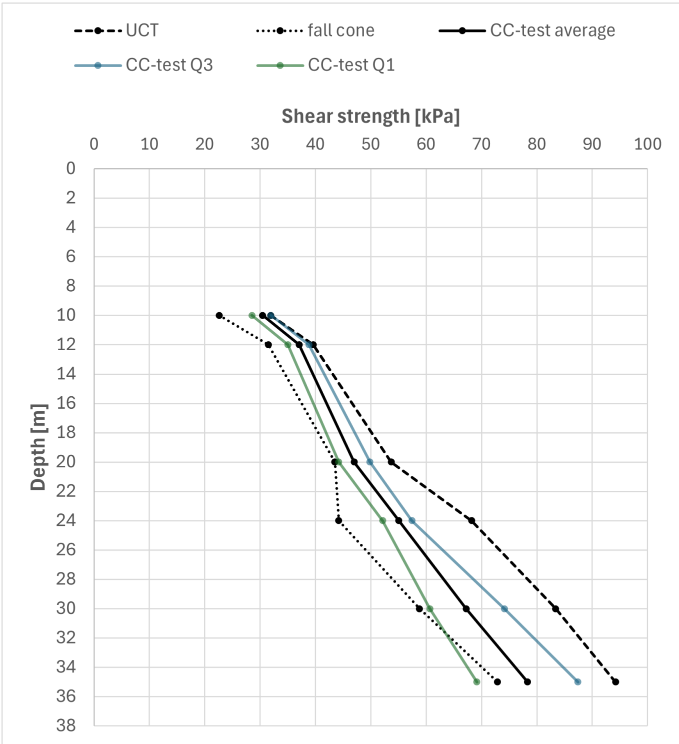


Figure 5.12: Results of shear strength from UCT, fall cone and CC-Test plotted with depth for Site B. The average value of the CC-Test as well as the upper and lower quartile (Q3 and Q1 respectively) are presented.

Comparing the three most shallow tests of 10 m, 12 m and 20 m, generally the same trend can be found. Depth 12 m is presented in Figure 5.13 to state the general example for the three depths and the other two can be found in Appendix 2 Figure B.3 and B.6, respectively. For these tests the value from the fall cone intersects the CC curve in the weaker part, representing roughly a minimum of the CC-Test. The UCT on the other hand, correlates more with the higher strength measured with the CC-Test.

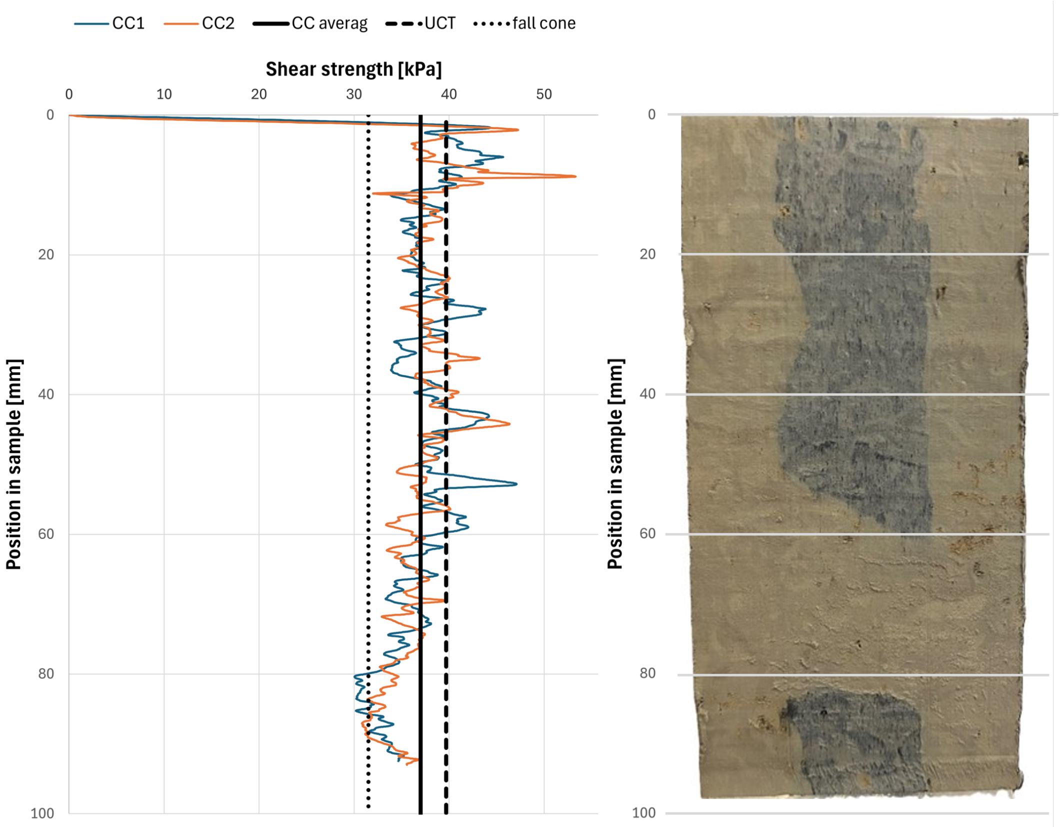


Figure 5.13: Shear strength profiles for the sample from 12 m depth at Site B, with position measured from the top of the sample. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

Comparing with the observation from Site A where the value from UCT correlated with the CC-Test at the endpoints of the shear plane, a similar pattern can be observed in Site B for the three shallow depths. The soil does not necessarily fail along the weakest horizontal section of the specimen, that would imply that the crack would be initiated in the weakest section presented by the CC-Test.

Instead the failure mechanisms is more complex, involving the overall stress distribution in the sample and finding the weakest diagonal plane. This does not necessarily correlate with the horizontal planes assessed by the CC-Test and thereby making it difficult to predict the value of UCT based on the CC curve. However, this difference is due to the fact that the CC-Test measures strength in horizontal planes and the actual failure tends to occur along diagonal shear planes. Notably, the values at the intersection between the endpoint of the shear crack and the CC curve are still relatively close.

Depth 24 m is presented in Figure 5.14. On this depth a new behaviour of the CC-Test starts to emerge. An oscillating pattern is measured by the CC-Test in some sections of the sample. The pattern is most pronounced in two regions of the sample, 3-20 mm and 40-53 mm measured from the top. In these areas the measured variation is more extreme with abrupt changes that are strangely uniform. From the picture of the cross section the oscillating pattern can be observed as impressions by the thread creating lines as it moves vertically through the sample.

However, some parts are not impacted by this and follows the behaviour more like the shallow depths. The fall cone generally presents a value of shear strength lower than the CC curve, whereas the UCT mostly is higher. The intersection of the CC curve with the UCT and fall cone are in the maximum and minimum values of the CC curve.

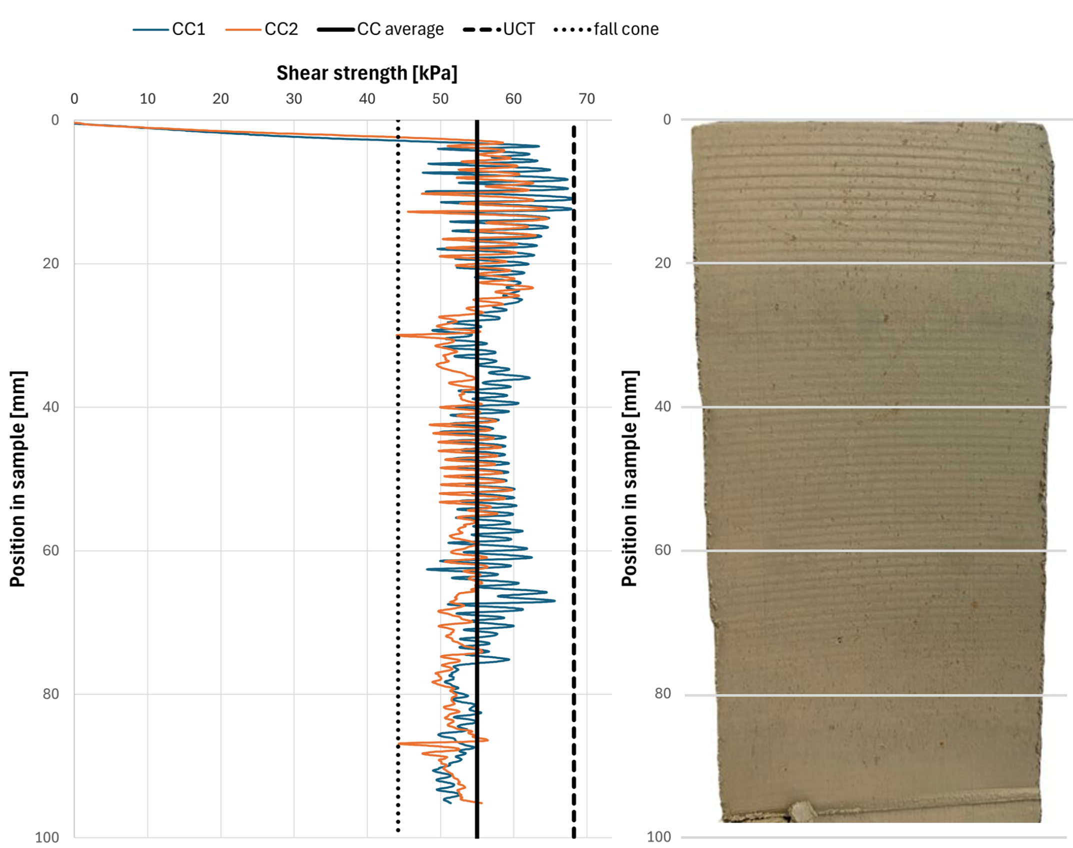


Figure 5.14: Shear strength profiles for the sample from 24 m depth at Site B, with position measured from the top of the sample. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

For the following depths, 30 m and 35 m, the oscillating pattern is even more extreme, see Figure 5.15 for depth 30 m and Figure B.18 in Appendix B for 35 m. This oscillating behaviour causes large variations in the CC curve and is probably why the IQR increases with depth as noted from Figure 5.12

For the depth of 30 m, CC1 presents this oscillating behaviour over the entire specimen, but the CC2 has an area where the oscillating pattern is minimized, see Figure 5.15. That is between 40 - 60 mm in the sample, measured from the top. Interestingly, this is in the area where the thread crosses the failure plane created by the UCT. So when the thread crosses this disturbed zone the oscillating pattern can not be found.

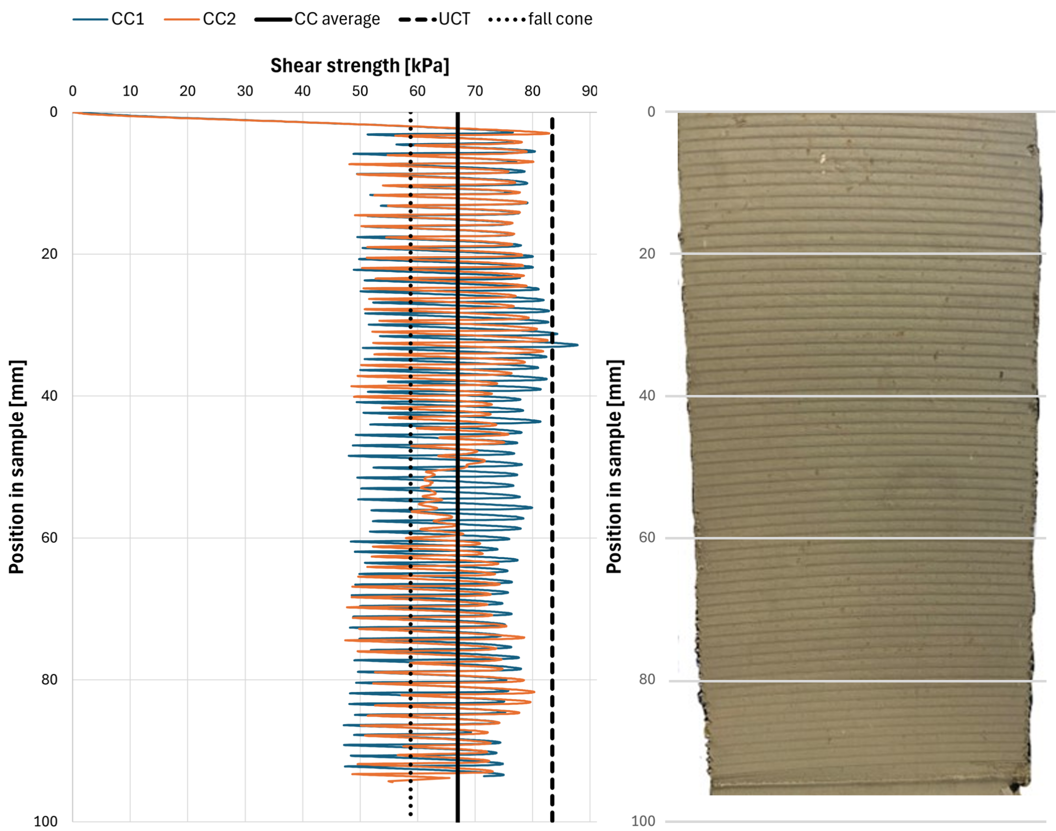


Figure 5.15: Shear strength profiles for the sample from 30 m depth at Site B, with position measured from the top of the sample. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

In Figure 5.16 a smaller part of the curve is presented. Note that the x- and y-axes do not start at 0. It is observed from the figure that the curve isn't oscillating evenly back and forth. Instead it builds up strength, gradually, to then get a rapid failure, to then start building up strength again. In the figure the data points are marked to emphasis this pattern. Observations of the thread during tests when this pattern emerges shows that the thread deflects slightly as it moves down. As the soil fails, the deflection of the thread is straightened and then it starts over again by the thread deflecting.

Since there is no difference of the thread or setup of the Clay Cutter compared to previous tests, the behaviour is likely due to variations of certain soil properties affecting the interaction with the thread. Since the layers are less than 2 mm, there are difficulties measuring specific properties of each layer. Furthermore, it is reasonable to assume that the behaviour comes from minor differences is the soil such as grain size, moisture content or structural arrangement. As these minor variations follow a cyclic pattern, it is possible that they appear due to seasonal variations.

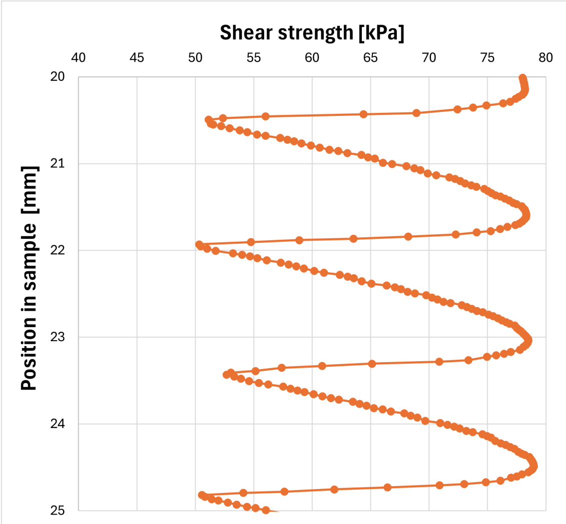


Figure 5.16: CC2 from 30 m close-up of oscillating pattern

5.4.3 Site C

The compilation of the three methods with depth for Site C is presented in Figure 5.17. The CC-Test generally presents undrained shear strength values between the fall cone and UCT. However, the value obtained with UCT for the depth of 8 m deviates from the trend, where it is the same value as the average for the CC-Test.

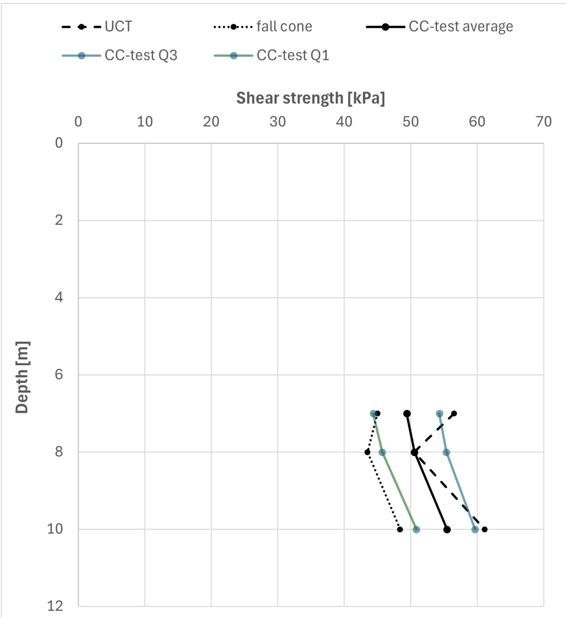


Figure 5.17: Results of shear strength from UCT, fall cone and CC-Test plotted with depth for Site C. The average value of the CC-Test as well as the upper and lower quartile (Q3 and Q1 respectively) are presented.

The complete CC curves, UCT and fall cone for depth 7 m in Site C is presented in Figure 5.18. As can be seen in the figure, the CC-Test presents similar oscillating patterns as in the deeper samples from Site B, over large parts of the profile. From the cross section, diagonal layers can be noted, possibly indicating previous landslide and redistribution of the soil layers.

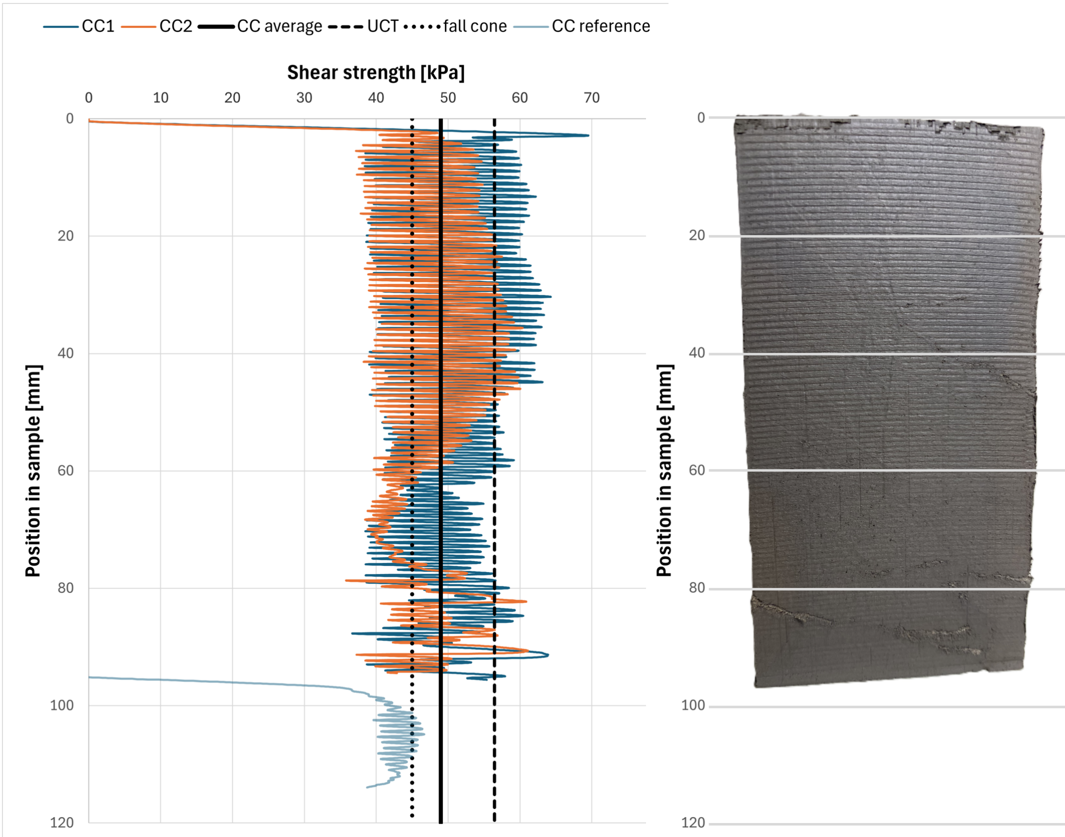


Figure 5.18: Shear strength profiles for the sample from 7 m depth at Site C, with position measured from the top of the sample. The reference CC-Test is plotted below the main CC curves, corresponding to the actual placement in the sample tube. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

The measurements for 8 m is presented in Figure 5.19. The oscillating pattern can be observed but only in certain areas. Furthermore, the sample from 7 m shows a more clear oscillating pattern than that of 8 m. The CC-Test presents extreme values of the same magnitude, minimum just below 40 kPa and maximum roughly 65 kPa, for depths of 7 m and 8 m. However, the strength profiles are different, where the 7 m oscillates back and forth between these extreme values on a large part of the test, whereas the 8 m has sections with higher and lower strength. In a way the 7 m sample is more homogeneous, whereas the 8 m has more layering. From the UCT it is evident that the 8 m provides a lower overall strength compared to the 7 m.

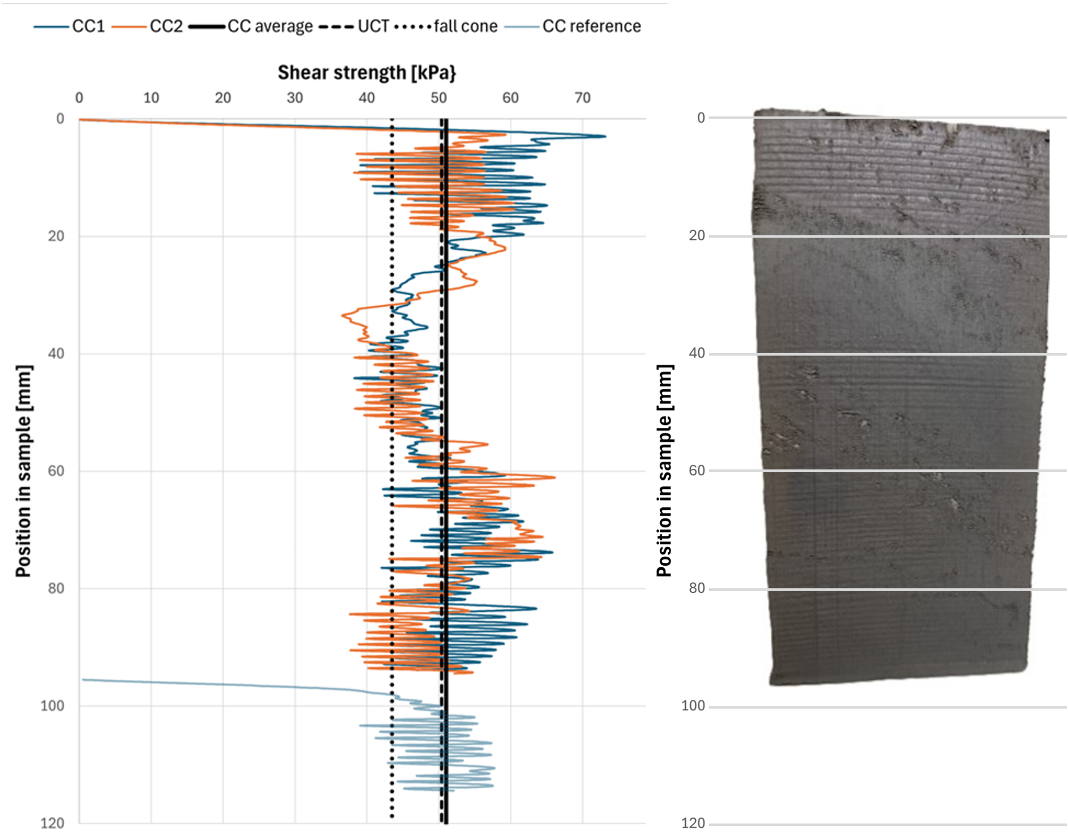


Figure 5.19: Shear strength profiles for the sample from 8 m depth at Site C, with position measured from the top of the sample. The reference CC-Test is plotted below the main CC curves, corresponding to the actual placement in the sample tube. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

In the results from Site B it is obvious that the oscillating behaviour emerges at a certain depth. However, since the strength increases with depth, it could be that the behaviour starts at a certain strength of the soil. However, for Site C the oscillating pattern can be found both in areas of higher and lower shear strength which indicates that it is not only above certain strength that this appears.

The reference CC-Test (executed before UCT) clearly shows the oscillating pattern. This means that the behaviour is not caused by the loading in the UCT since it is already present in the natural samples.

The results from 10 m is presented in Figure 5.20. The specimen has two distinct parts, where the first has a more extreme oscillating behaviour, whereas it becomes less pronounced towards the second half. The average value is higher than the two previous depths.This highlights that it is not the depth or the shear strength, exclusively, that determines the oscillating behaviour.

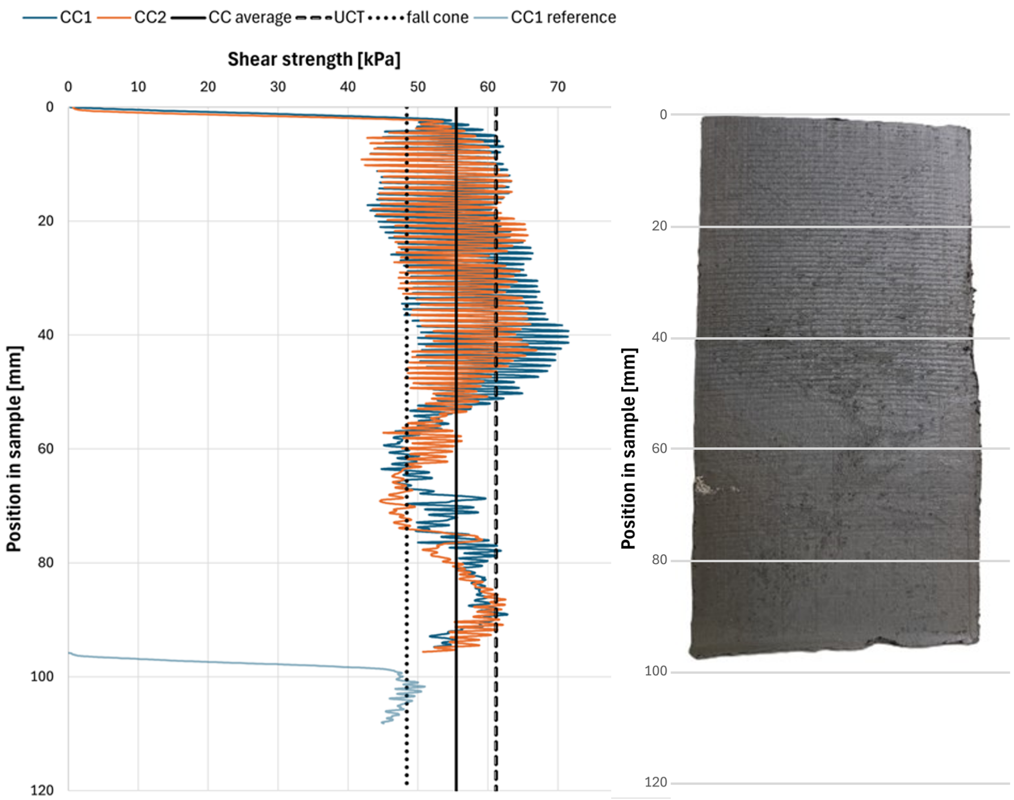


Figure 5.20: Shear strength profiles for the sample from 10 m depth at Site C, with position measured from the top of the sample. The reference CC-Test is plotted below the main CC curves, corresponding to the actual placement in the sample tube. To the right, a cross sectional image from CC1 is aligned with the corresponding vertical position in the sample.

5.4.4 Summary of in-depth analysis

Concluding from the in-depth analysis of curves, the CC-Test generally presents higher values than the fall cone in the tested depth range (7 m-35 m). This is to some extent expected as the fall cone is generally known to present values on the lower side for depths below 20 m, see Section 2.4.1.

The average value of the CC-Test is generally lower than from the UCT, indicating that using the average value from CC-Test would result in a conservative estimate of undrained shear strength. However, for samples taken from depths below 20 m, the difference between the average from CC-Test and UCT values becomes significant. In these cases, even the upper quartile could be regarded as a conservative value compared to the UCT.

When observing the CC curves more in detail it is concluded that the value of UCT correlates well with the CC-Test at the position in the sample corresponding to the endpoints of the shear plane. A higher or lower strength may appear in other areas of the sample, but will not contribute significantly.

In some samples, an oscillating pattern in the CC-Test emerges. For Site B this behaviour starts to emerge below a depth of 24 m. At Site C, on the other hand, the oscillations are found in both higher and lower parts of the curves, and is less pronounced in deeper samples. Based on the reference CC-Test, the oscillating curves can be found in the undisturbed pieces of the sample, highlighting that the behaviour is not induced by the UCT.

Furthermore, based on the reference tests in Site A and Site C no significant difference between the CC-Test after UCT and undisturbed can be found. However, a slight indication that the curve generally is somewhat higher after UCT can be noted. This may be due to the natural variation of soil and not necessarily be caused by the UCT.

6

Discussion

6.1 Test executions

Concerning the archive compilation, the tests were executed prior to this study and only documented information can be regarded. For some CC-Tests, it is not explicitly stated whether the tested soil is from the same tube as the fall cone or DSS. All compared tests are considered to have been retrieved from the same depth, but could thereby be from the upper, middle or lower tube at that depth. The potential for deviation due to natural variation of soil increases if the samples are from different tubes, which may contribute to deviating values in the results.

Furthermore, the CC-Tests are for some of the tested samples performed in the uppermost of the three retrieved sample tubes. This sample is generally considered to be of lower quality compared to the middle and lower tube, primarily due to increased risk of disturbance during extraction. Since disturbed samples tend to present lower strength values, this may contribute to underestimating the shear strength obtained with the CC-Test.

For some CC-Tests in the archive compilation a clear difference of the magnitude of CC1 and CC2 can be observed, see Figure 3.3 for explanation of placement in the sample. While anisotropic conditions in the horizontal plane are not necessarily incorrect, they indicate that the soil strength varies with direction. As a result, the placement of CC1 and CC2 relative to the anisotropy will influence the measured strength values. Such anisotropy may be caused by surrounding conditions in-situ, for example if the sample is retrieved from a slope. Although the testing setup partly compensates for this by conducting CC1 and CC2 at 90 degree angles and evaluating the average of the two, the anisotropy can still potentially lead to some deviations.

Concerning the in-depth study, the UCT, CC-Test and fall cone are all executed on the same day for each sample. This was done to minimize the effect of soil relaxation between the test, which could otherwise affect the results.

The strain in the UCT was limited to a maximum of 3% which is lower than standard procedure which is typically 15%. This alteration was made to avoid disturbing the soil sample too much. Since all UCT tests showed clear shear failure and measured a clear drop in strength, it is assumed that the peak undrained shear strength was indeed recorded.

However, one possible source of uncertainty for the CC-Test is the fact that the UCT was conducted in the same specimen prior to the CC-Test. Even though the strain is limited, it could still disturb the soil structure, causing the CC-Test to measure a different resistance than if executed in the sample first.

To investigate this uncertainty, reference tests were carried out on additional pieces of soil from the same sampling tubes at Site A and Site C. These tests are not extensively deviating from the main tests, but a tendency that the main CC curve presents slightly higher values than the reference can be noted. This may be explained by the UCT compressing the sample, thereby increasing the resistance in the following CC-Test. However, it is not possible to rule out that the deviation is due to the natural variation of soil.

The soil samples used in the in-depth study are generally quite brittle in their character, as most show clear shear planes when performing UCT. In two cases, additional vertical cracks are observed as well as the diagonal shear plane, further indicating brittle behaviour. The samples are of varying soil types, but they are from areas with glacial clay which has experienced significant loading. This overconsolidation likely affects the soil structure and mechanical response.

6.2 Use of the CC-Test

The purpose of the CC-Test is not to replace any of the current existing methods. Instead, it is developed with the aim of complementing other methods by providing more information about the soil samples. As the fall cone can present very different values of undrained shear strength within the same sample, the CC-Test presents the strength profile which can help explain such variation.

Furthermore, the more advanced methods only present a single value, the maximum load the soil can withstand in a small piece of soil. As presented in this report, that value can correspond to a maximum, a minimum or an average value of the CC curve. By conducting the CC-Test after an advanced test, it is possible to present if there are areas within the sample of higher or lower strength than what the advanced test has shown. The CC-Test, can reveal if the soil is relatively homogeneous, supporting the same value given in the advanced method, or if significant variability exists, that could be accounted for when evaluating the overall shear strength for the depth.

6.2.1 Interpretation of the CC-Test

For the CC-Test to be accepted and used in the industry as a method of determining absolute strength, clear guidelines on how to evaluate the test is essential. The current recommendation of evaluating the CC-Test is to choose the average value of the steady state part of the curve, taking into account CC-Tests of all directions in the same specimen. For soils with strengths up to 25 kPa the results from this study show that the value correlates well to the DSS and fall cone. In particular for soft soils below 20 kPa where the agreement is strong to the sustained methods.

For stiffer soils, which are more studied in-depth in this report, the representative value from CC-Test is often higher than the fall cone, but conservative in comparison to the UCT. Even the upper quartile of the measurements from the CC-Test provides values lower than the UCT. This supports the use of the average value as conservative in comparison. If additional, more advanced tests present higher values than the average, the evaluation of the CC-Test can be made more towards the higher strength.

For the fall cone tests, the unreduced value of shear strength is used for comparison in this thesis. This raises the question whether a similar reduction should be applied to the CC-Test results as fall cone values are typically corrected based on the liquid limit. However, no correlation between the liquid limit and the measured undrained shear strength was identified in this study that would justify such a correction. Furthermore, the correlation between the values obtained with the CC-Test and DSS, which is not reduced, is satisfactory. Because of that, applying a correction based on the liquid limit is not recommended.

6.2.2 Potential applications of the CC-Test

Besides determining absolute strength values or present the variation of strength within a sample, there could be other areas of usage for the CC-Test. Due to its simplicity and speed, it could be useful in assessing sample quality between tubes from the same depth. Index testing are normally performed on the bottom tube, while more advanced testing is conducted on the middle or bottom tube. Quality assurance often consists of measuring the density and water content in the middle tube before more advanced testing. The CC-Test could complement this procedure by providing a quick assessment of strength between different tubes, thereby helping verify homogeneity and similar properties.

Currently, there is no standardized procedure for determining soil sensitivity with the CC-Test but it is under development. Cyclic penetration of the thread is possible with the Clay Cutter and could in the future create the remoulded state of the soil to be tested. Preliminary tests has shown positive results concerning degradation of the strength during cyclic cutting. If the CC-Test can reliably determine the remoulded undrained shear strength, allowing for the calculation of sensitivity, it would present a significant advantage. This method would be more time-efficient and potentially more cost-effective than current approaches.

CC-Test is not developed to replace the fall cone as an index test, however in the future it has the possibility to provide similar information; undrained shear strength, the remoulded strength and sensitivity. Furthermore, the remoulded CC-Test could possibly present the rapidity, the rate of remoulding. Which is not possible with the fall cone.

6.3 Limitations of the CC-Test

A limitation of the CC-Test is the lack of control concerning the stress state of the soil. This is a crucial point as the shear strength depends on the current stress state. When the soil sample is retrieved from the ground, it is exposed to unloading which leads to a tendency for the soil to expand. This expansion is to some extent limited by the sampling tubes during transport and storage, but cannot be entirely prevented. Since the test is performed without any reconfining pressure, it is assumed that the stress conditions in the sample are still representative of the in-situ state. In reality, stress relaxation and sample disturbance may alter the strength of the sample. As a result, the measured shear strength may underestimate the true in-situ strength.

Furthermore, during the test procedure, the specimen is not laterally confined and can theoretically deform radially during the test procedure. Whether such radial deformation actually occurs, and to what extent it influenced the measured resistance, remains uncertain. No visible deformation was observed during the CC-Tests, but this does not fully exclude the possibility of minor changes. Further investigation is required to fully understand this behaviour and its implications for the accuracy and reliability of the results.

7
Conclusion

When evaluating the accuracy of the CC-Test, it can be concluded that the average value of the steady state part of the CC-Test corresponds well to the values obtained with the fall cone and DSS in the range 0-25 kPa. This is similar to the range of which the fall cone is calibrated and generally considered reliable. For the DSS there are only a few samples with strengths above 25 kPa, so while the indication is positive, the conclusion can not be made with confidence due to the limited amount of data.

In terms of material properties, higher density and lower water content tend to produce slightly higher CC-Test values, while lower density and higher water content result in lower values. A correlation can be found for thread diameter where larger diameter corresponds to a lower measurement of resistance. However, in the range 0.2 mm-0.3 mm only a maximum of 5% deviation can be found. Concluding a range that is suitable for standard testing.

For samples between 7 and 35 m depth, the average value of the steady state part of the CC curve can for some cases be a good representative value of shear strength. Compared to the fall cone results, the average of the CC-Test is consistently higher, but compared to UCT it presents lower values for most cases. For most tested samples even the upper quartile presents a conservative value compared to UCT. This tendency gets more pronounced with increasing depth and higher strength. This indicates that at higher strength, the average of the CC-Test deviates from the UCT by presenting consistently lower values.

One of the greatest benefits of the CC-Test is presenting local variation within the sample. Identifying zones of higher or lower strength, or stratification gives new possibilities of interpretation. Rather than giving only a single value, the CC-Test can contribute to the understanding by presenting a strength profile. While the mean value of the test curve at the steady state can be regarded as a representative value, the curve contains valuable information that should not be neglected. Such variation is an important aspect to bring in further evaluation and design processes. Due to its simplicity and rapid execution the CC-Test can also be used as a quality assurance, supporting other methods as well as sustain homogeneity among sample tubes.

In conclusion, the CC-Test shows strong potential as a simple and efficient method for assessing undrained shear strength. Some aspects still need to be developed, like interpretation standards and a stronger theoretical foundation. However, the method stands out by providing detailed strength profiles and present internal variability. With continued refinement, it could become a valuable tool within research as well as the geotechnical industry.

7.1 Further research

In this project, few samples of coarser material is used. It remains unknown how the CC-Test performs in soils containing more silt and sand which could be an interesting aspect to investigate. As the grains get larger, they are closer to the diameter of thread, which will most probably impact the failure mechanism and the measured resistance.

As the CC-Test is performed on the soil samples directly, without any reconsolidation, the impact of relaxation from sampling to the time of testing is of interest. Furthermore, the measurements may be influenced by the fact that the specimen is not restrained to deform laterally during penetration, which would also be a topic for further research.

The failure mechanism for the soil around the thread is not sustained. Numerical analysis could investigate this closer, particularly in determining an appropriate factor for calculating the undrained shear strength from the measured resistance. The approach could be similar to the T-bar test in shallow depth and under partially submerged conditions. Analytical derivation may not be directly applicable, but numerical modelling might be a possible alternative.

Further development of deriving the remoulded shear strength and the sensitivity with the Clay Cutter would present a great advancement. If successful, it could significantly reduce testing time and offer economic benefits.

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A
Appendix 1

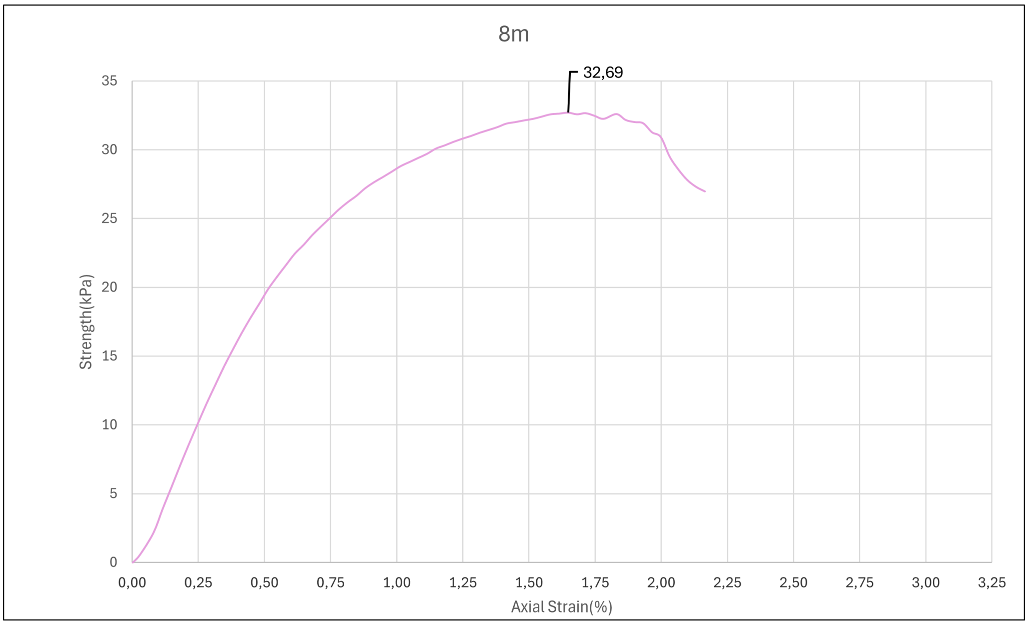

Figure A.1: UCT measurements from 8m depth in Site A with the maximum strength marked

(a) Before UCT

(b) After UCT

Figure A.2: Specimen from 8m before and after UCT, where the shear plane can be observed after UCT.

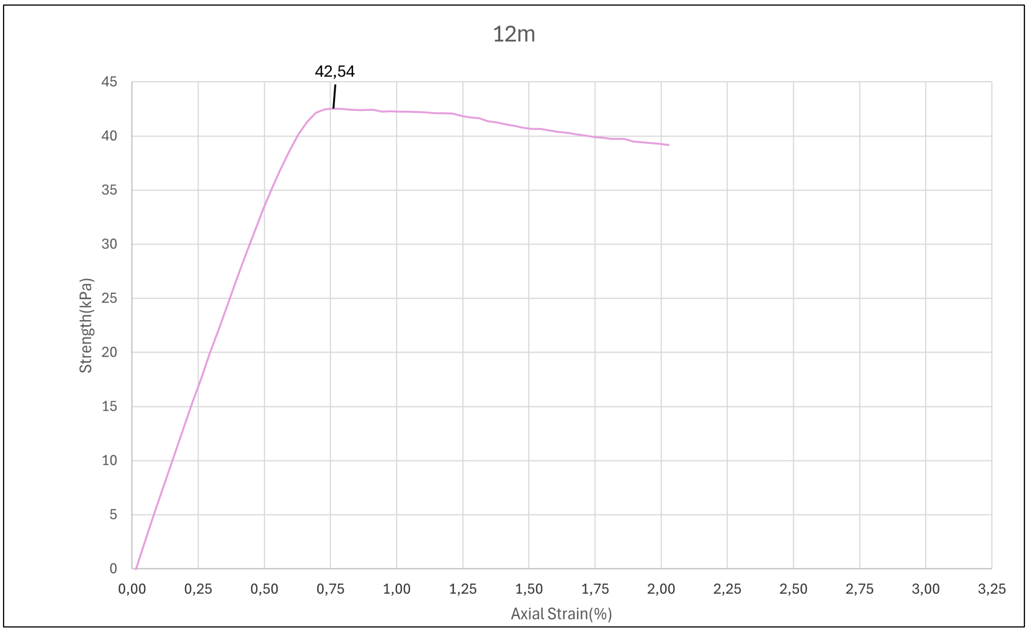

Figure A.3: UCT measurements from 12 m depth in Site A with the maximum strength marked

(a) Before UCT

(b) After UCT

Figure A.4: Specimen from 12 m before and after UCT, where the shear plane can be observed after UCT.

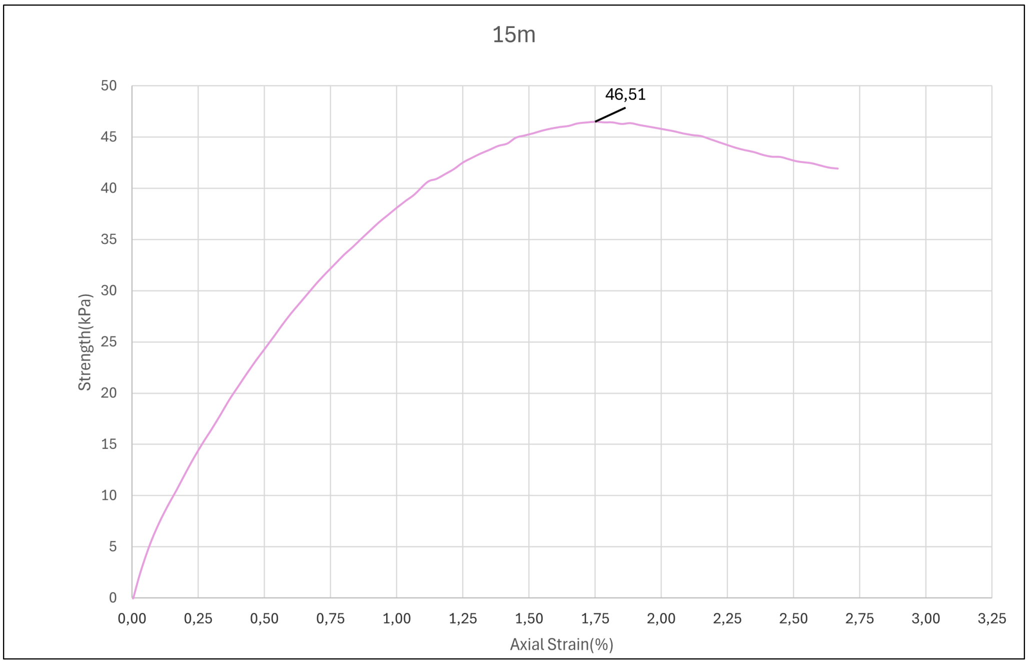

Figure A.5: UCT from 15m depth with the maximum strength marked

(a) Before UCT

(b) After UCT

Figure A.6: Specimen from 15 m before and after UCT, where the shear plane can be observed after UCT.

Depth [m]	**Level**	**Cone impression [mm]**	**Cone impression [mm]**	**Water content [%]**
8	top	11.5	11	71.5
	middle	11.5	11	72.2
	bottom	11	11.5	74
8	reference	-	-	72.2
12	top	10	9.8	65.1
	middle	11	11.5	60.7
	bottom	11	11.5	67.8
12	reference	-	-	65.1
15	top	11	10.5	47.4
	middle	10.5	10.3	60.3
	bottom	11.5	11	56.1
15	reference	-	-	60.3

Table A.1: Data from fall cone test for Site A

B
Appendix 2

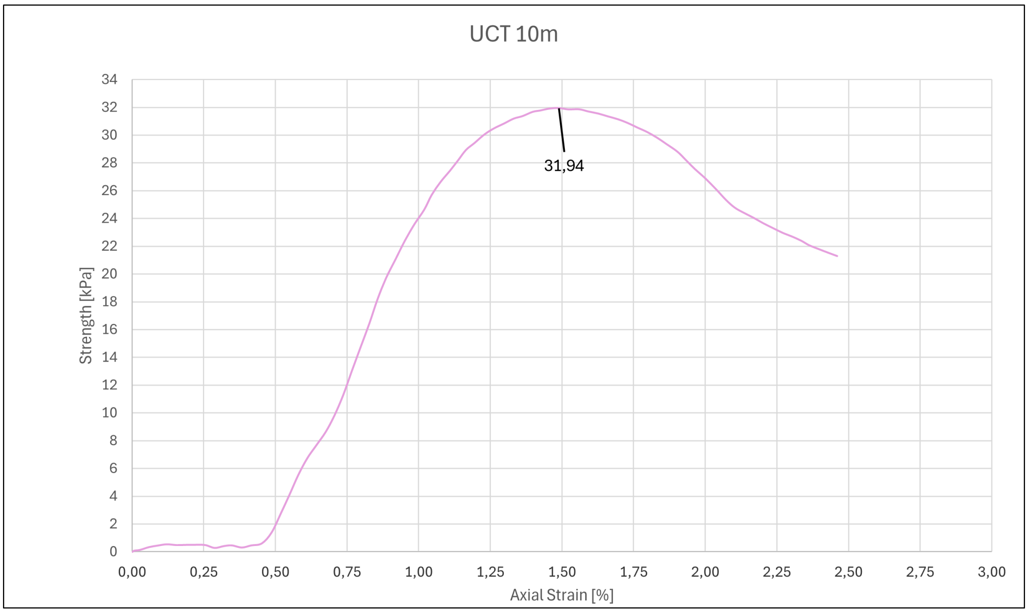

Figure B.1: UCT measurements from 10 m depth in Site B with the maximum strength marked

(a) Shear plane **(b)** Vertical cracks

Figure B.2: Specimen after UCT from 10 m at two different sides showing the shear plane and vertical cracks

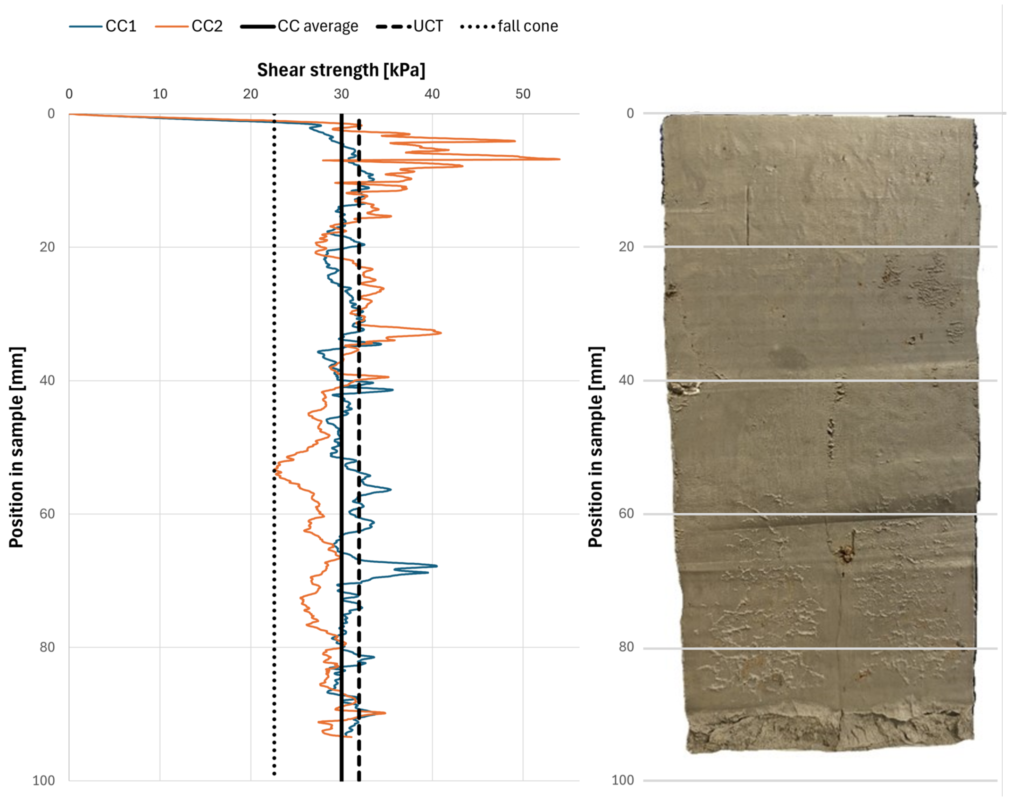

Figure B.3: CC Test and cross section from depth 10 m of Site B

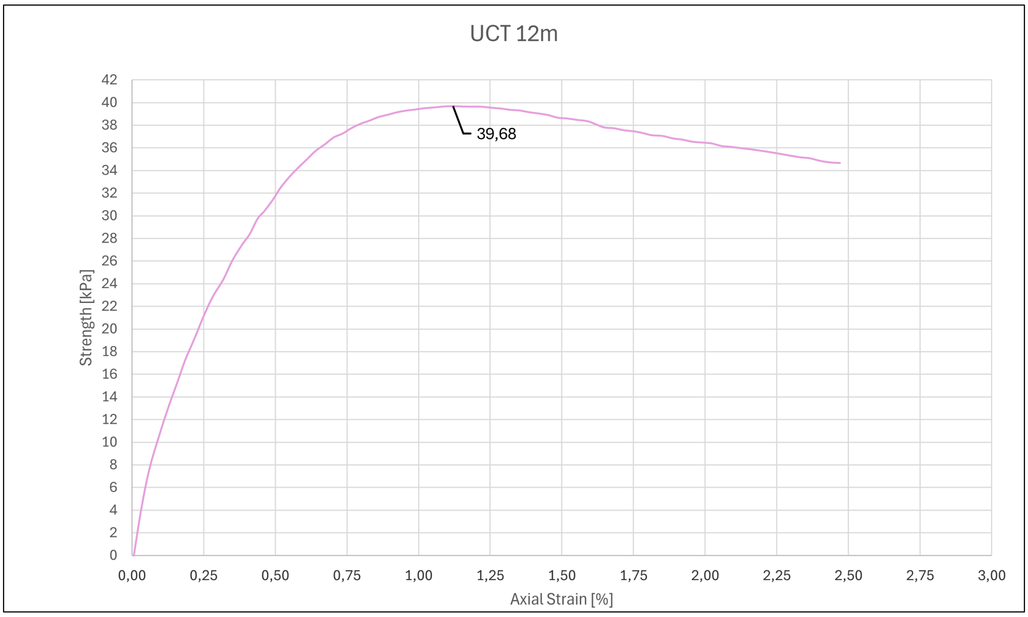

Figure B.4: UCT measurements from 12 m depth in Site B with the maximum strength marked

Figure B.5: Specimen after UCT from 12m showing the shear plane

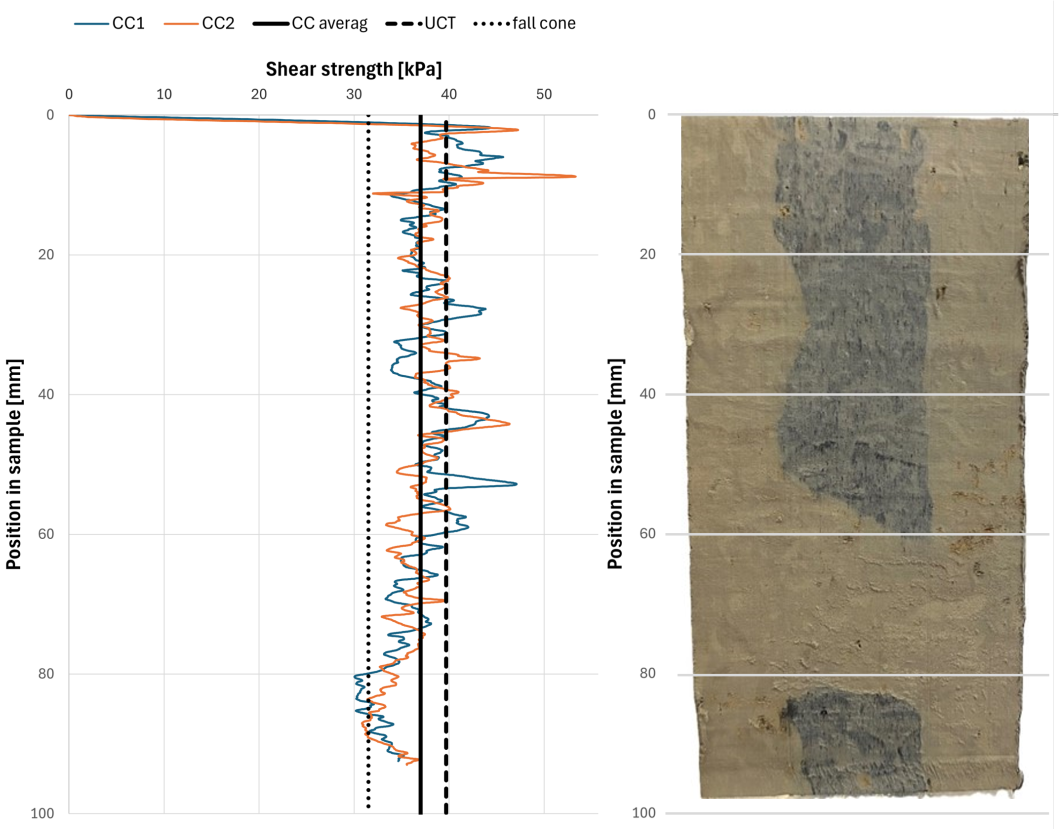

Figure B.6: CC Test and cross section from depth 12m of Site B

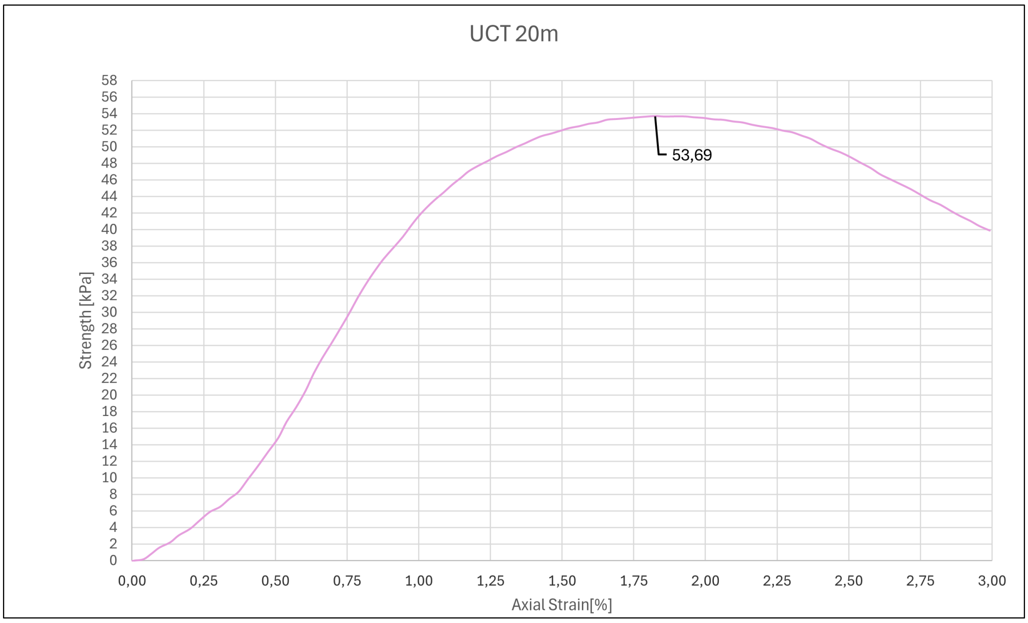

Figure B.7: UCT from 20m depth with the maximum strength marked

Figure B.8: Specimen after UCT from 20m showing the shear plane.

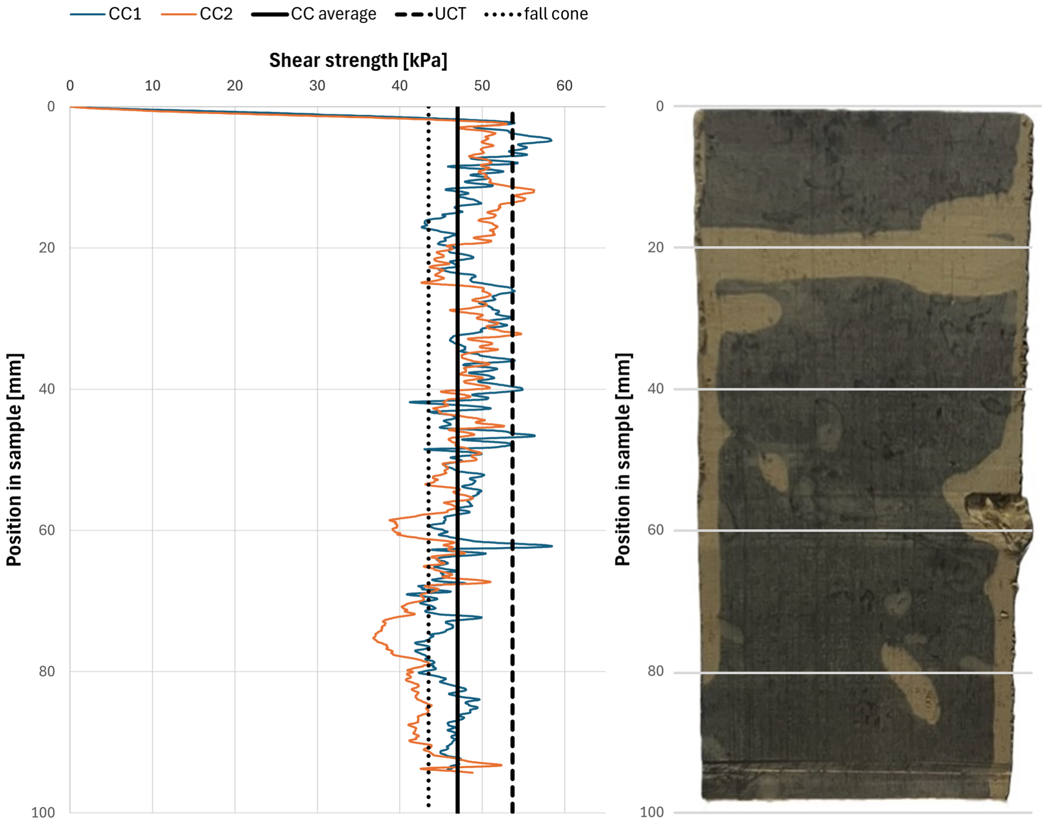

Figure B.9: CC Test and cross section from depth 20 m of Site B

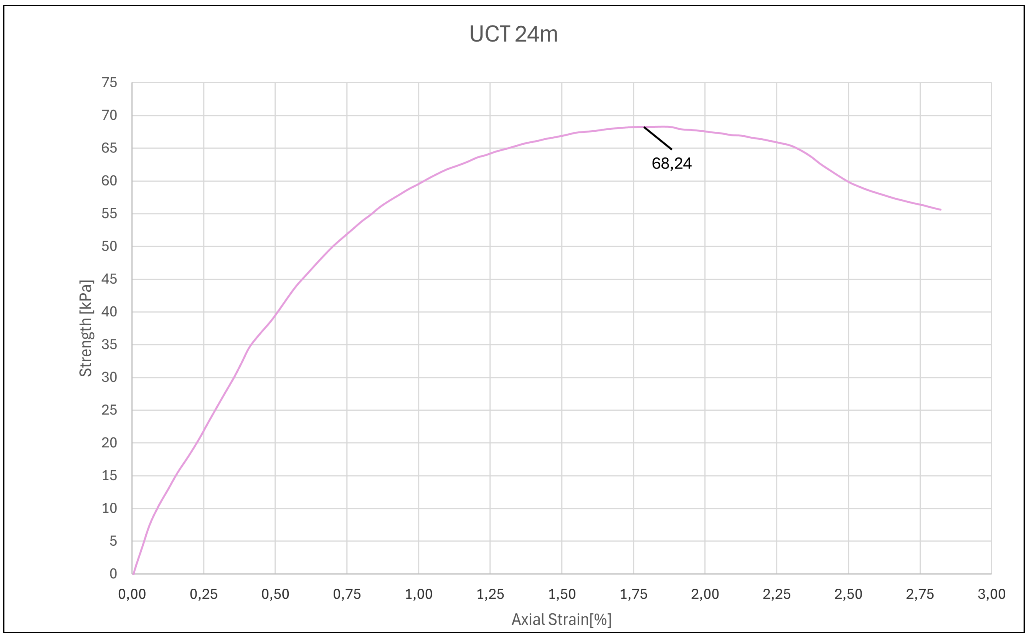

Figure B.10: UCT from 24m depth with the maximum strength marked

(a) Shear plane

(b) shear plane

Figure B.11: Specimen after UCT from 24 m at two different sides showing the shear plane

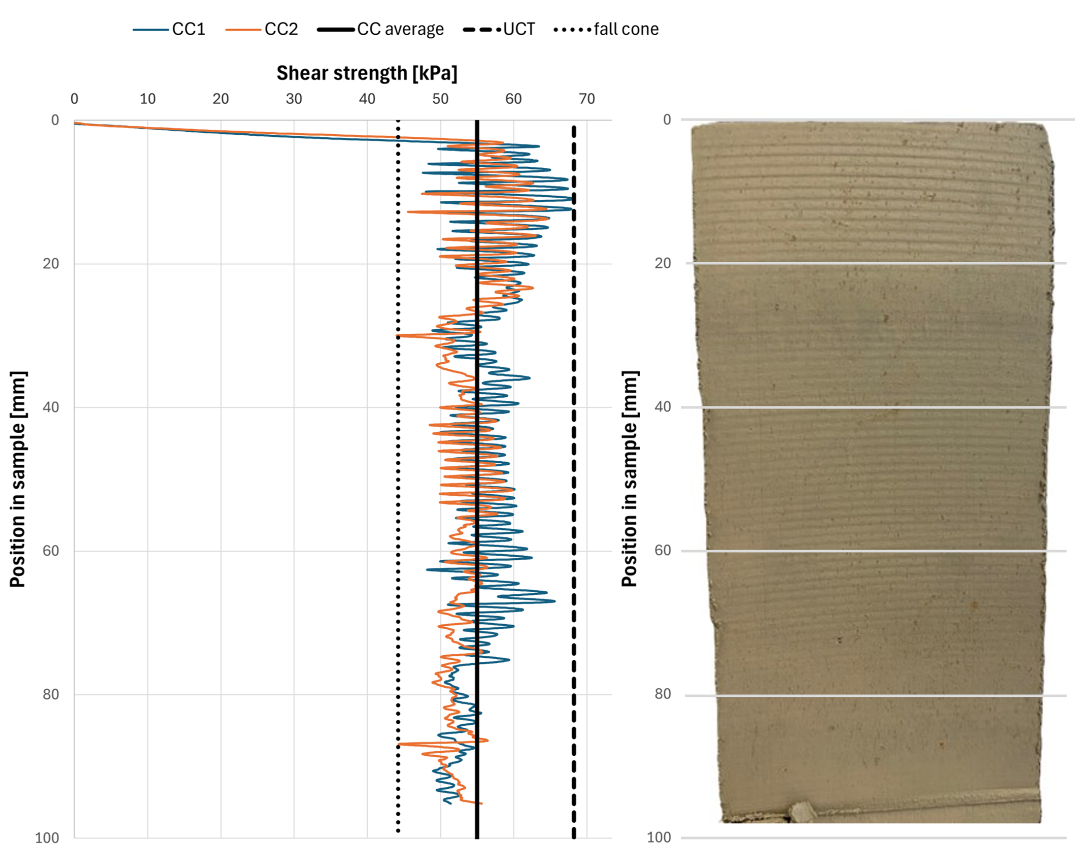

Figure B.12: CC Test and cross section from depth 24 m of Site B

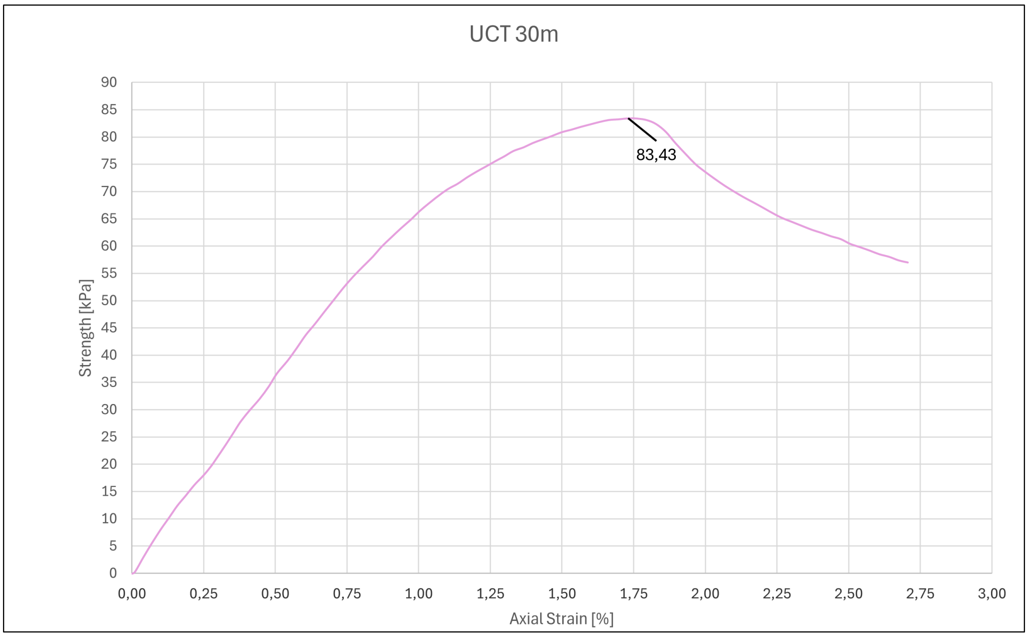

Figure B.13: UCT from 30m depth with the maximum strength marked

(a) Shear plane

(b) shear plane

Figure B.14: Specimen after UCT from 30 m at two different sides showing the shear plane

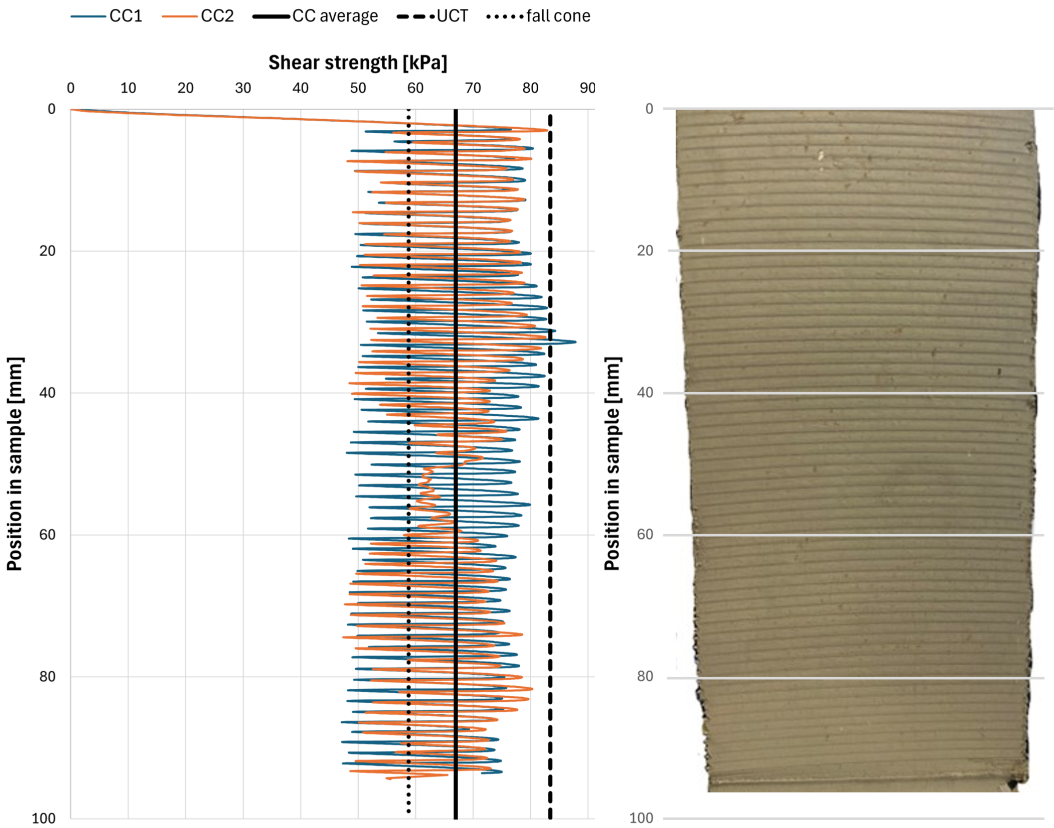

Figure B.15: CC Test and cross section from depth 30 m of Site B

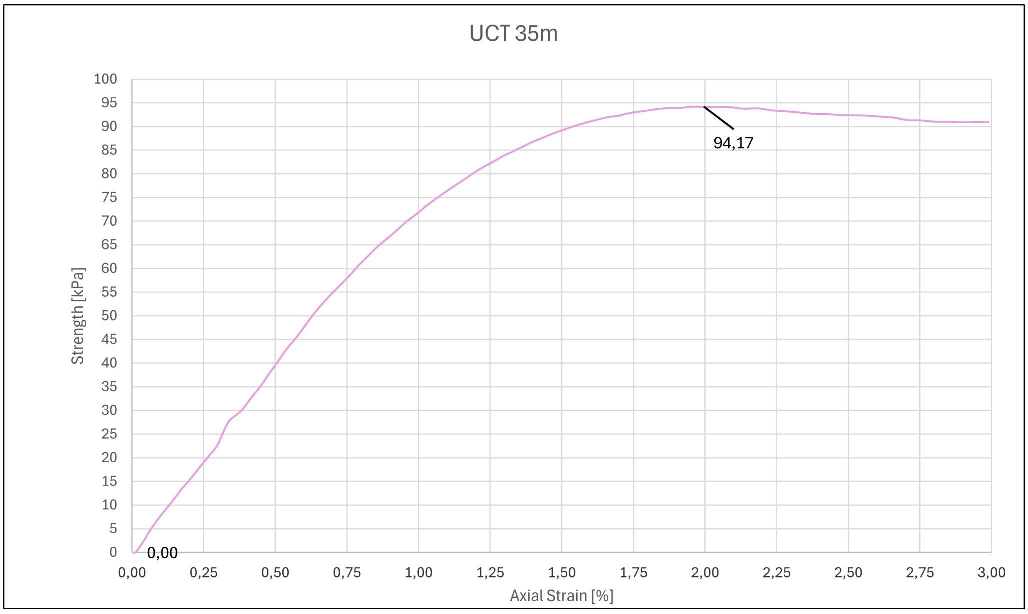

Figure B.16: UCT from 35m depth with the maximum strength marked

Figure B.17: Specimen after UCT from 35 m showing no clear shear plane. Instead a more slanted shape where the right side on the image has been more compressed than the left.

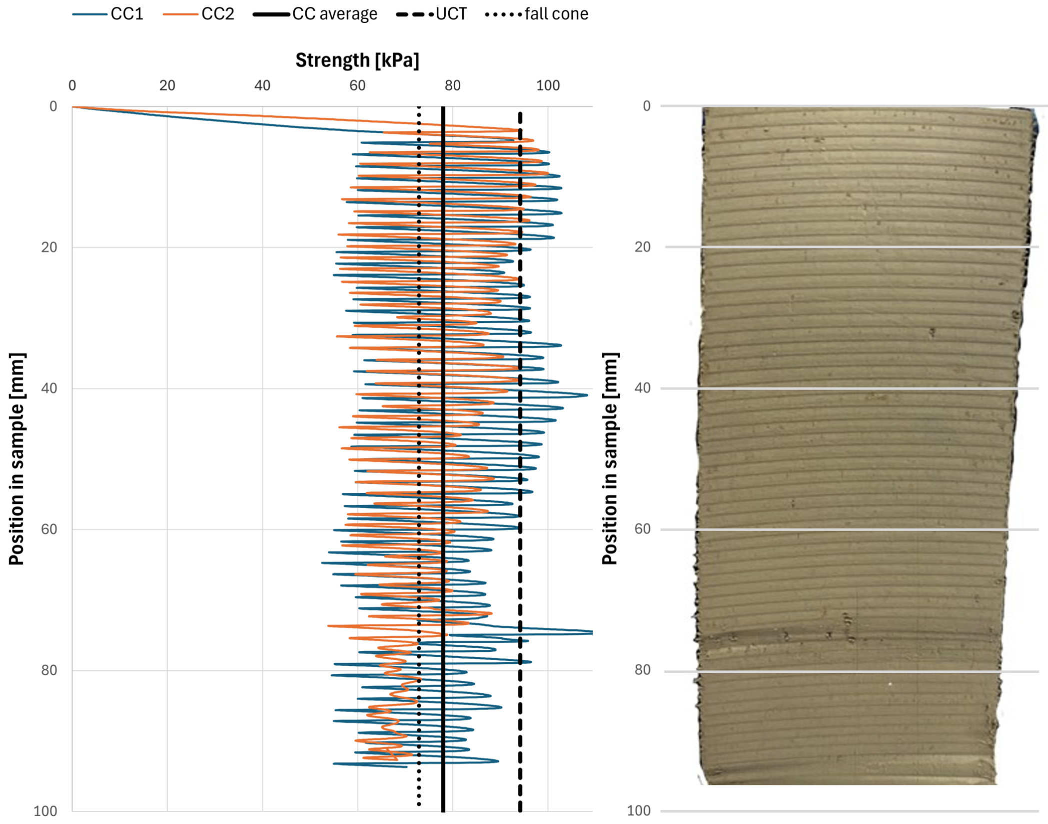

Figure B.18: CC Test and cross section from depth 35 m of Site B

Depth [m]	**Level**	**Cone impression [mm]**	**Cone impression [mm]**	**Water content [%]**
10	top	14	13	64.3
	middle	12.5	13.5	63.3
	bottom	12.5	13.5	63.6
12	top	11	11.5	60
	middle	11.5	11	59.5
	bottom	10.5	11.5	58.7
20	top	9.2	9.2	61.5
	middle	9.8	9.5	64.4
	bottom	9.6	9.7	62.7
24	top	9.5	9	61.2
	middle	10	19.5	62.3
	bottom	9	9.5	60.2
30	top	7	8	61.7
	middle	8.5	8.5	60.9
	bottom	8.5	8.5	63.6
35	top	7.5	7	60.5
	middle	7	7.5	60.4
	bottom	7.5	7.5	57.3

Table B.1: Data from fall cone test for Site B

C

Appendix 3

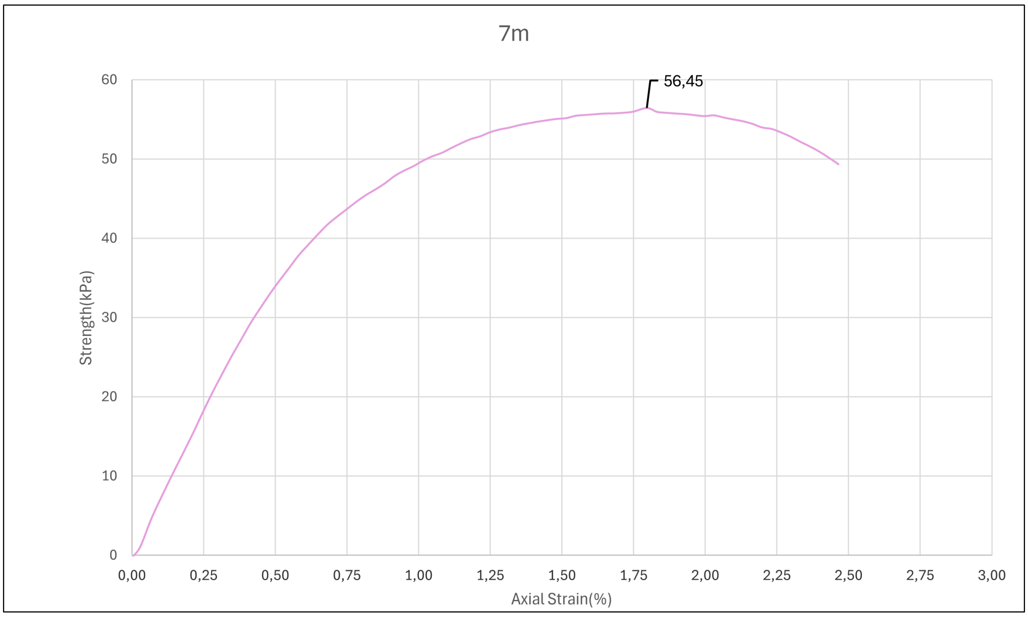

Figure C.1: UCT from 7m depth with the maximum strength marked

(a) Before UCT

(b) After UCT

Figure C.2: Specimen before and after UCT from depth of 7 m from Site C

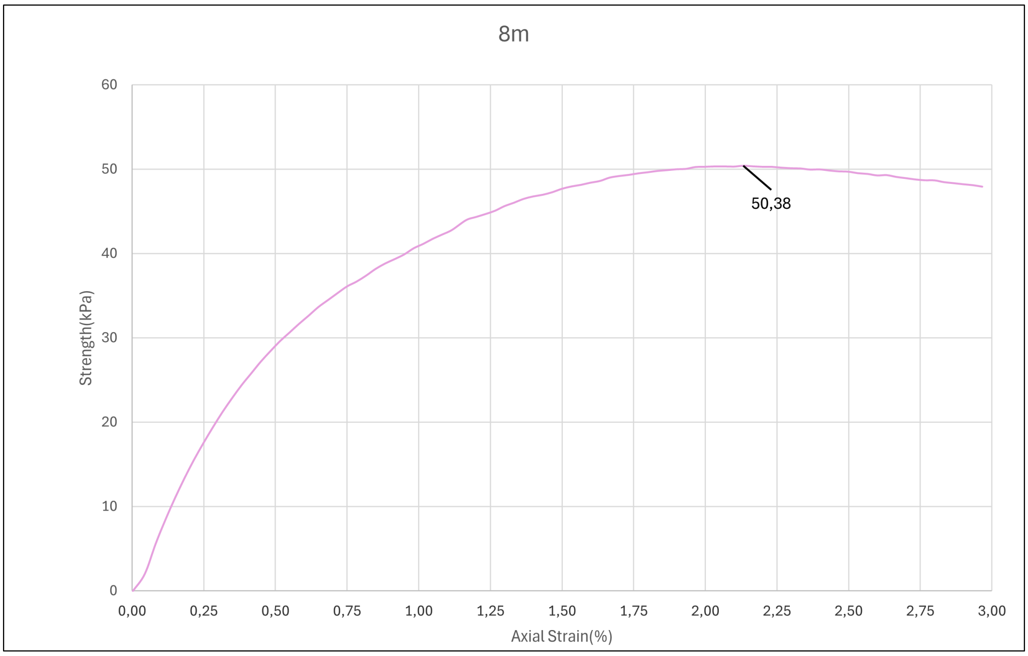

Figure C.3: UCT from 8m depth with the maximum strength marked

(a) Before UCT

(b) After UCT

Figure C.4: Specimen before and after UCT from depth of 8 m from Site C

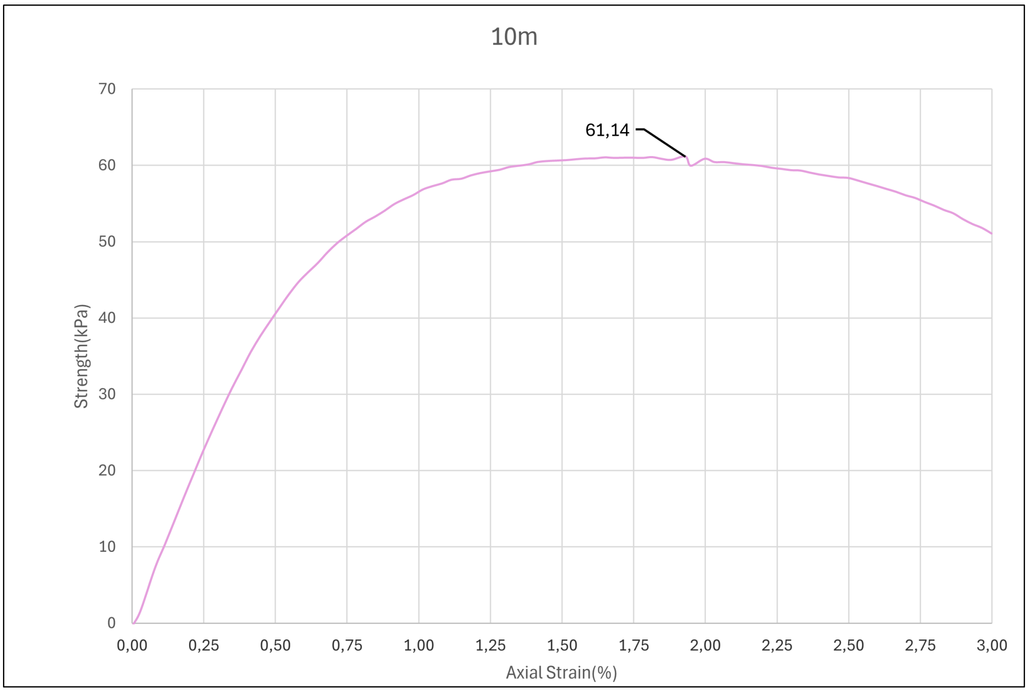

Figure C.5: UCT from 10m depth with the maximum strength marked

(a) Before UCT

(b) After UCT

Figure C.6: Specimen before and after UCT from depth of 10 m from Site C

Depth [m]	**Level**	**Cone impression [mm]**	**Cone impression [mm]**	**Water content [%]**
7	top	8.5	9	51.4
	middle	9.5	10	53.4
	bottom	9.5	9.5	51.1
7	reference	-	-	51.8
8	top	9	8.5	35.1
	middle	10	9.5	33.3
	bottom	10	10	36.7
8	reference	-	-	36.5
10	top	9.5	9.5	70.1
	middle	9	9	73.6
	bottom	8.5	8.5	63.1
10	reference	-	-	69.6

Table C.1: Data from fall cone test for Site C

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