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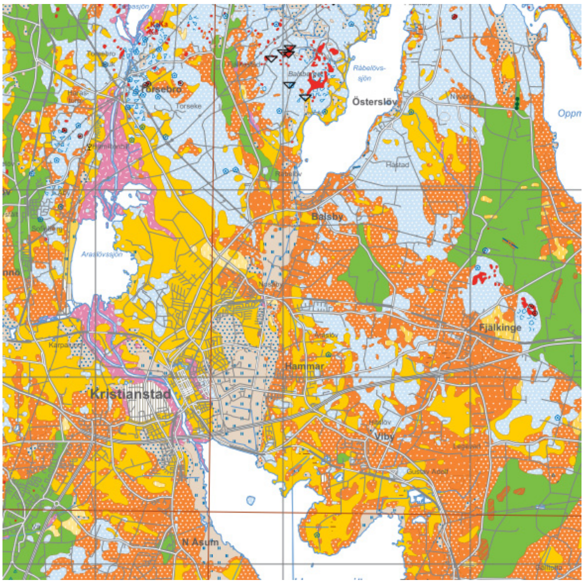

EVALUATING UNDRAINED SHEAR STRENGTH OF CLAY WITH DPSH-A

A down to earth assessment of the applicability in Scania, Sweden

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Geotechnical Engineering

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MASTER'S DISSERTATION

EVALUATING UNDRAINED SHEAR STRENGTH OF CLAY WITH DPSH-A

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Abstract

The undrained shear strength of cohesive soils is an important parameter to assess in geotechnical investigations. It is normally evaluated with the cone penetration test (CPT) but problems with penetrating coarse, heterogeneous materials that is also hard, such as clay till, have pointed the interest towards other possible in-situ methods. This has led to studies assessing the possibility to evaluate undrained shear strength from dynamic probing. This thesis evaluates a suggested relation between the dynamic probing and the undrained shear strength of soft clays in Scania, Sweden. The undrained shear strength was evaluated by dynamic probing, CPT, field vane and piston sampling to obtained undisturbed samples for triaxial tests in the laboratory. The undrained shear strength evaluated with DPSH-A, according to the studied relation, is consistently lower than the same parameter evaluated from triaxial testing, indicating an inaccuracy of the relation. However, with only two triaxial tests conducted it is difficult to draw conclusions. The comparison between DPSH-A and CPT did not indicate any reliable correlation with the studied relation, albeit the scatter is large. The data set obtained by field vane was largely scattered and no indication of the accuracy of the proposed relation could be given. An intention was made to find better suited relations for Swedish soft clay, however R^2-values of around 0,25 - 0,30 indicated that the data was largely scattered and no new relations could be suggested. Additionally, suspicions were raised that the data from DPSH-A might be unreliable because of the fact that the DPSH-A was conducted in such a soft clay that the recommended minimum number of blows per penetrated unit was not achieved. This indicates a possible source of error, probably affecting also the final results and analysis.

In conclusion, it is possible that the studied relation between the calculated dynamic tip pressure, q_d, from DPSH-A and the evaluated undrained shear strength of soft clay could be accurate in soft Swedish clays, but further research is needed.

Sammanfattning

Den odränerade skjuvhållfastheten för kohesionjordar är en viktig parameter att utvärdera i geotekniska undersökningar och utvärderas normalt med CPT (cone penetration test). Dock har problem med att kunna föra ned sonden i grova, mycket heterogena jordar, så som lermorän, riktat intresset mot andra möjliga in-situ-metoder. Det har lett till att flera studier som undersöker möjligheten att utvärdera odränerad skjuvhållfasthet med dynamisk sondering har genomförts. Detta examensarbete söker därför utvärdera en av de föreslagna relationerna mellan dynamisk sondering och den odränerade skjuvhållfastheten hos skånska lösa leror för att ge en indikation på om den är tillämplig även i Skåne. Den odränerade skjuvhållfastheten för leran i nordöstra Skåne utvärderades med dynamisk sondering (närmare bestämt DPSH-A), CPT, vingförsök och med kolvprovtagning för att ta upp ostörda prover för att senare kunna genomföra triaxialförsök i laboratorium.

Den odränerade skjuvhållfasthet som utvärderats med DPSH-A, enligt den föreslagna ekvationen, ger konsekvent lägre värden än samma parameter som utvärderats med triaxialförsök, vilket indikerar att ekvationen inte skulle vara giltig i Sverige. Det är dock svårt att, med endast två utförda försök, dra några slutsatser. Korrelationen mellan odränerad skjuvhållfasthet utvärderad med CPT och DPSH-A, enligt den föreslagna ekvationen, är svag och datan har en relativt stor spridning. Den data som samlades in från vingförsöken visade sig ha en mycket stor spridning och denna studie kan inte ge några indikationer på tillämpligheten hos ekvationen. Vidare väcktes misstankar om att datan från den dynamiska sonderingen kunde vara otillförlitlig på grund av att sonderingen utfördes i en lera som var så pass lös att det rekommenderade antal slag per penetrerat djup till största delen inte uppnåddes. Detta indikerar ett möjligt fel i den insamlade datan, vilket troligtvis även påverkar det slutliga resultatet och analysen.

Sammanfattningsvis kan sägas att det är möjligt att den föreslagna relationen mellan det beräknade dynamiska spetstrycket, q_d, för DPSH-A och den utvärderade odränerade skjuvhållfastheten i lös lera, kan vara tillämplig även i lös svensk lera, men fler studier på området är nödvändiga.

Acknowledgements

This master thesis concludes five years of studies at the Faculty of Engineering at Lund University (LU). The project was developed in association with the Division of Geotechnical Engineering at Lund University and WSP Civils in Malmö.

I would like to thank my supervisors Erika Tudisco, LU, Dennis Overgaard, WSP, and Mohamad Fakhro, WSP, for valuable support, guidance and feedback during this project. Additionally, I would like to thank the people in the geotechnical engineering group at WSP with whom I spent a lot of time and who listened to my speculations and offered support throughout the process of this thesis.

Last but not least, a thank you to Marie Sundström for her dreary jokes on soil.

Lund, May 2018

Julia Knutsson

Notations and symbols

Latin letters

A - cross section area of probe tip
c - cohesion
c' - effective cohesion
c_u - undrained shear strength
CD - consolidated-drained triaxial test
CPT - cone penetration test
CU - consolidated-undrained triaxial test
DPL - dynamic probing light
DPM - dynamic probing medium
DPH - dynamic probing heavy
$DPSH-A/DPSH-B$ - dynamic probing super heavy
e - average number of blows per penetration unit
f_t - mantle friction
g - gravitational acceleration
h - fall height of fall weight
HfA - Swedish ram-sounding
K_0 - coefficient for lateral earth stress at rest
k_e - the ratio q_d/q_c
$k'e$ - the ratio N_{DP}/q_c
K_{0NC} - coefficient for lateral earth pressure at rest when normally consolidated
I_c - consistency index
I_L - liquidity index
I_p - plasticity index
m - mass of fall weight
m' - total mass of equipment
M_{max} - maximum torque
N_{20} - number of blows per 20 cm
$N_{20,netto}$ - corrected number of blows per 20 cm
N_{60} - number of blows per penetration unit when corrected for efficiency
N_{DP} - number of blows per penetration unit
N_{DPL} - number of blows per penetration unit for DPL
N_{field} - uncorrected number of blows per penetration unit
OCR - over consolidation ratio

p', s' - effective mean stress
q - deviatory stress
q_c - cone resistance
q_d - dynamic tip pressure
q_T - total tip pressure
r_d - driving resistance
s - penetrated interval
S_t - sensitivity
t - shear stress
T - torque
u - pore pressure
u_0 - current pore pressure
UU - unconsolidated-undrained triaxial test
w_L - liquid limit
w_p - plasticity limit
z - depth

Greek letters

α - coefficient relating q_c and q_d
β - coefficient relating q_c and N_{20}
ε_a - axial strain
ε_{vol} - volumetric strain
γ - specific weight
ϕ - friction angle
ϕ' - effective friction angle
σ_0 - total vertical stress
σ_1, σ_3 - principal stresses
σ_a, σ_r - axial and radial stresses
σ_c - pre-consolidation pressure
σ_{v0} - vertical in-situ stresses
σ'_{v0} - effective vertical in-situ stresses
τ_f - shear strength
τ_v - corrected shear strength

Contents

Chapter 1

Introduction

The first chapter presents the background to this master thesis and also states objectives and limitations as well as describes the approach and methods used.

1.1 Background

The soil of the southernmost region of Sweden, Scania, consists mainly of clay till (Larsson, 2000). This type of soil material can be troublesome for one of the most commonly used methods for evaluating shear strength: the cone penetration test (CPT) (Robertson and Cabal, 2015). The CPT is a pressure probe and hence sensitive to heterogeneities present in materials such as clay till. Furthermore, it has also limitations as to maximum driving force, which often impede deep enough investigations in the hard Scanian clay till. Today, the CPT is used mainly in clay and soft clay tills with a large content of fine grains. Characteristics of the soil material are then evaluated with well established correlations between, e.g., the tip pressure of the CPT and the shear strength (Larsson, 2007). Nevertheless, in stiff to very stiff soils, the CPT probe cannot penetrate deep enough, or at all (Larsson, 2007), consequently the Swedish ram sounding method HfA-A, a type of dynamic probing, internationally called DPSH-A, is widely used in these cases. However, this type of sounding functions mainly for stratification purposes and whether or not it is suitable for evaluating further parameters is under discussion. Even though some studies and relations between dynamic probing, to which DPSH-A belongs, and undrained shear strength have been published (e.g., Butcher et al., 1996; Dalvi dos Santos and Bicalho, 2017; Gadeikis et al., 2010), the ability to evaluate undrained shear strength from DPSH-A is still largely in its bud and lacks the validation necessary for a wider implementation in Sweden.

To be able to use the DPSH-A for evaluating soil parameters in clay till, specifically shear strength, more research and studies are necessary. This master thesis examines the possibility to evaluate the undrained shear strength of clay by DPSH-A and functions as a preliminary study for further research on clay till.

1.2 Purpose and limitations

The objective of this master thesis is to evaluate the shear strength of clay from DPSH-A by calibrating it against triaxial tests, CPT and field vane tests. This study focuses on the clay in the north east of Scania, where the clays are predominantly categorised as soft clays and the analysis is therefore limited to the type of clay found on the site. Moreover, the results are area specific and are not necessarily comparable to soft clays from other regions. Due to time and resource restrictions, a limited number of triaxial tests is conducted, making this a preliminary study with further research needed.

1.2.1 Research questions

1. Can correlations between parameters from DPSH-A and the undrained shear strength of clay, suggested by Butcher et al., be used in soft clay in Scania?

2. If not, can any other statistically validated correlations be proposed?

1.3 Methods

1.3.1 Literature study

A literature study is conducted in order to provide an understanding of basic geological and geotechnical concepts, to describe the equipment used for sampling and testing and to provide a compilation of previous studies on the

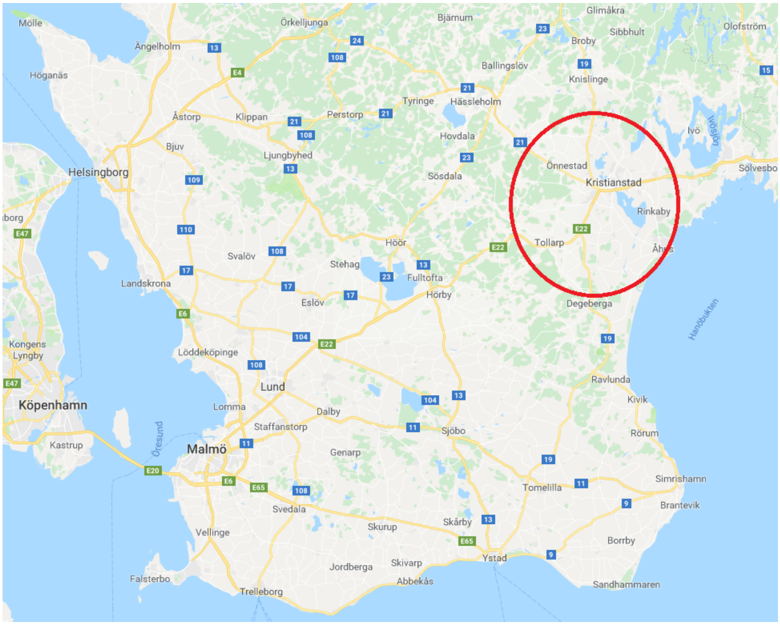


Figure 1.1: Investigation site. (Google Maps, 2018)

subject. The relations used to evaluate shear strength from DPSH-A found in literature are then analysed and assessed in this thesis.

1.3.2 Field and laboratory study

A field study is carried out, where data is collected from geotechnical field investigations with driving rig in the north east of Scania, see Figure 1.1 where WSP has an ongoing project regarding the construction of a bridge over an existing road. Field tests and sampling are necessary in order to examine the material underlaying the foundation to the bridge. CPT, field vane and DPSH-A was conducted, together with piston sampling, in order to obtain samples to examine in triaxial tests. The piston sampling and the DPSH-A was conducted as geographically close as possible, according to standards.

1.3.3 Laboratory study

A laboratory study is performed, where triaxial testing on specimen obtained from the assumed layer of clay is analysed. Routine tests, as well as triaxial testing, is conducted in laboratories in Halmstad, Gothenburg and Lund.

1.3.4 Method of analysis

Relations between the output parameters from DPSH-A and the undrained shear strength of clay published in international studies are analysed and evaluated in order to determine the validity in the soft clay in Scania. Furthermore, the derived results from the DPSH-A are also analysed in the light of CPT and field vane results.

1.4 Outline of report

The outline of the report is as follows:

Chapter 2: Theory

The theoretical chapter discusses relevant geological settings, divided into national, regional and local geology. It also treats basic geotechnical concepts, such as friction and cohesion soils, in-situ stresses, consolidation and shear strength.

Chapter 3: Methodology

In the methodology chapter, the field investigation and laboratory methods used in this master thesis, together with evaluation methods of undrained shear strength, are presented.

Chapter 4: Previous studies

Previous studies on relevant subjects will be presented in order to give a scientific base to this master thesis.

Chapter 5: Field study

Specifications and details regarding the field investigations conducted are treated, as well as conditions and execution on the laboratory tests.

Chapter 6: Results

Results from field investigations and laboratory tests are presented.

Chapter 7: Analysis

The results obtained from the previous chapter are analysed and discussed.

Chapter 8: Conclusion

In this chapter, a conclusive discussion on the results and analysis will be presented together with possible sources of error.

Chapter 9: Further research

Future research based on this thesis will be suggested in this last chapter.

Chapter 2

Theory

In this section, geology, basic geotechnical concepts and the theory behind the shear strength of soils are presented.

2.1 Geology

In this subsection the geology of Sweden and, specifically, the bedrock and quaternary deposits of northeastern Scania are presented.

2.1.1 Sweden

The geology in Sweden differs significantly between the north and the south. In the south, the sedimentary rock is predominant, while in the north, the older granite bedrock is the most common rock. The Swedish bedrock was created approximately 400 to 500 million years ago and consists of three different formations: the basement rock, the Kaledonidians and the sedimentary bedrock (Fredén, 2002). Tectonic movements have given rise to the Scandinavian mountain range on the border with Norway, while glacial periods have formed the landscape with visible ridges and valleys (Lundqvist et al., 2011).

The geology in Sweden is strongly affected by numerous glacial periods, effects which can be seen both in the landscape at large and, on a smaller scale, in the glaciofluvial deposits that appear abundantly all over the country. The last glacial period ended around 12 000 years ago and left the country to a land rise that is still ongoing (Fredén, 2002).

2.1.2 Scania

Scania is the southernmost region of Sweden and in many ways more similar geology-wise to continental Europe than to the rest of the country, except the typical Scanian clay till. The region is divided from northwest to southeast by the Tornqvist zone, a zone with a width of 75 km created by movements in the continental plates. The zone is visible in the Scanian landscape in faults and horsts (Fredén, 2002). South of this diagonal the geology is similar to the rest of Europe with thick sedimentary deposits, while north of the zone, the older Swedish bedrock lies close to the surface and is sometimes visible in the landscape (Lundqvist et al., 2011). Due to the violent history of glacial ices advancing and regressing, different types of till, predominantly clay till, are the most common soils in the region (Larsson, 2000).

2.1.3 North eastern Scania

The north east of Scania has had a less tumultuous past considering glacial ice movements than the southeastern part of Scania and hence the quaternary deposits in the area consist of less till and presents soil of less stiffness than the rest of the region, since the smaller clay and silt particles were given time and possibility to sediment (Larsson, 2000). The field investigation site, chosen for this thesis, is part of the southern Swedish moraine area, where the bedrock is dominated by granite and gneiss. However, in the vicinity of the construction site, the bedrock belongs to the Fanerozoic bedrock, consisting of mainly sedimentary deposits such as limestone, sandstone and clay (Fredén, 2002). This owes back to the so called Kristianstad basin, an area of cretaceous deposits east of Hässleholm and around Kristianstad, defined by a clearly marked fault in the south and slowly disappearing deposits to the north (Lundqvist et al., 2011). Till is the most commonly present soil type in the southern Swedish moraine area, albeit in the south eastern part, glaciofluvial deposits and peat exist abundantly and, on the Kristianstad plain, the soil consists mainly of varved clay and sand (Fredén, 2002).

2.2 Geotechnical concepts

Basic concepts of soil, the difference between frictional and cohesive soils together with the definition of clay, in-situ stresses and shear strength are presented.

2.2.1 Soil

Soil is a material consisting of three phases: solid, gas and liquid phases. The solid phase consists of particles of minerals and organic content, while the gas and liquid phases are represented by pore gas and pore water respectively. The composition of the soil, i.e., the relation between the three phases, depends on the geological formation and its history, and are crucial parameters when considering geotechnical soil characteristics. The relation can be described by a number of parameters, e.g., density, water content and porosity (Larsson, 2008).

2.2.2 Friction and cohesion soils

Soil is divided in two categories, separated by the type of binding that attracts the particles, resulting in the strength of the material. Friction soil builds up its strength from friction between the particles and consists mainly of coarse soils, such as sand. The friction angle is decisive for the strength of the material and when this is exceeded, failure occurs in the soil. For cohesive soils, not only frictional forces determine the strength, but cohesion, i.e., a force of attraction between molecules, is an important factor. Nonetheless, likewise as for frictional soils, failing occurs in cohesive soils when the strength in the material is exceeded. Clay, and to some extent silt, is considered cohesive soils (*Jords hållfasthet* 2018). This master thesis focuses on cohesive soils, i.e., clay.

Particles with a diameter of less than 2 micrometer are denominated clay particles and can appear as primary particles, i.e., single particles, or as secondary particles, i.e., units of particles. The latter is dominant in Swedish clays. The requisite for a soil to be denominated clay is that at least 40 % of its weight consists of fine material (i.e. particle size $\leq$ 0,006 mm), where at least 40 % of the fine material is made up of clay particles (Larsson, 2008). The liquid limit, w_L, and the plasticity limit, w_p, are important parameters when nominating and classifying clay and are defined as the transition limit between liquid and plastic behaviour and between plastic and half-solid behaviour of the soil. The clay is often categorised according to its plasticity index, $I_p = w_L - w_p$, or its consistency index, $I_c = (w_L - w)/I_p$. Regarding consistency index, the soil is classified as soft if it demonstrates an index below 0,50, and as stiff clay if above. For very soft clays, it is sometimes more accurate to classify the soil according to its liquidity index, $I_L = (w - w_p)/I_p$. The definition and classification of soft and stiff soils are of importance to this thesis, since the clay in the investigation area is expected to belong to soft clays. The classification is preferably conducted in a laboratory where liquid limit and plasticity limit can be evaluated (Larsson, 2008), albeit experience and previous investigations in the area interesting to this thesis are concordant of the type of clay found in this region, i.e., soft clay.

2.2.3 In-situ stresses

The soil on a specific depth is affected by the overlaying soil, creating vertical in-situ stresses that can be calculated from the mass of that soil and the depth.

$$\sigma_{v0} = \int \gamma dz \tag{2.1}$$

where:
σ_{v0} = Vertical in-situ stress, kPa γ = unit weight, kN/m^3
z = depth, m

The pore pressure also affects the soil behaviour and the effective vertical stress is derived as follows.

$$\sigma'_{v0} = \sigma_{v0} - u_0 \tag{2.2}$$

Where:
u_0 = current pore pressure, kPa

In cohesive soils, the effective stress can be larger than the total stress because of the pore pressure being negative above the ground water table. This owes back to the capillar nature of clay, where pore water can rise over the ground water table due to capillary forces in the cavities between the particles. The soil is also affected by horizontal stresses, normally derived in relation to the vertical stress with the lateral earth stress coefficient, K_0, based on empirical relations.

When conducting triaxial tests, in-situ stresses are an important parameter to assess in order to reconstruct conditions from where the specimen was acquired.

2.2.4 Consolidation

Consolidation occurs when the soil is exposed to increased stresses or groundwater pumping, causing the volume of fully saturated soil to decrease because of a drainage of pore water and compaction of the soil. The pre-consolidation pressure, σ'_c, represents the highest stress the soil has previously experienced. The soil is considered normally consolidated if current stress equals the pre-consolidation pressure, and any further applied stresses cause plastic deformations. Correspondingly, the soil is considered over-consolidated if the stress history indicates previously higher stresses than today and deformations due to increased stresses are elastic until reaching the pre-consolidation pressure of the soil. Under-consolidated soils also exist, albeit, not very commonly (Larsson, 2008). The pre-consolidation of a soil has large impact on the shear strength of the material, especially when the largest principal stress is the vertical stress, and it is therefore an important parameter to assess when evaluating the shear strength (Larsson, 2008). The over consolidation ratio, OCR, is the ratio between the preconsolidation pressure and the current pressure and the soil can be categorised, according to recommendations from the Swedish Geotechnical Society (SGF), as presented in Table 2.1 (Larsson, 2008). Normally the Swedish clays are slightly over consolidated, with an OCR of 1,3 on the west coast and 1,2 on the east coast (Larsson, 2008).

2.2.5 Shear strength

The parameter most commonly used to describe the strength of soil is shear strength, (τ_f), which is affected by stratification and stress history. This parameter is not characteristic for a specific type of soil but varies depending on internal and external conditions. For example, confining pressure, the

Table 2.1: Categorisation considering the over-consolidation ratio (OCR) (Larsson, 2008).

OCR	
Normally or slightly overconsolidated	1-1,5
Over consolidated	1,5-10
Strongly over consolidated	>10

OCR and the velocity of deformation during shearing all affect the resulting shear strength. The most common failure criteria for evaluating shear strength in drained conditions is

$$\tau_f = c' + \sigma' \tan\phi \tag{2.3}$$

In the equation, c' is the effective cohesion, ϕ is the friction angle and σ' is the normal effective stress.

In undrained conditions, the friction angle is of very little importance and the strength of the soil is determined, almost solely, by the cohesion. Therefore, Equation 2.3 changes to

$$\tau_f = c_u \tag{2.4}$$

These equations are based on principles developed by Mohr and Coloumb. In a confined soil element affected by principle stresses, σ_1 and σ_3, as illustrated in Figure 2.1, failure occurs when the shear stress in any plane reaches the shear strength, τ_f, as illustrated in Figure 2.2. However, the undrained shear strength of clay is best represented by Figure 2.3, as all of the strength consists of cohesion, as mentioned before. Since this thesis treats clay, it is Figure 2.3 that best illustrates the shear strength in question.

It is also common to present the stress path for a confined element, as can be seen in Figure 2.4. The stress path illustrates the changing stresses during laboratory a test, while a Mohr circle only illustrates the stresses in a specific moment, making the stress path a more illustrative manner of presenting the results and allowing easier interpretation and evaluation. Nevertheless, it has to be decided for each project individually the best way to present its results (SGF, 2012).

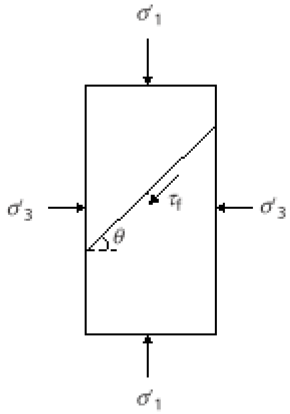


Figure 2.1: Normal, shear and principle stresses on a confined soil element. Inspired by Craig and Knappett, 2012.

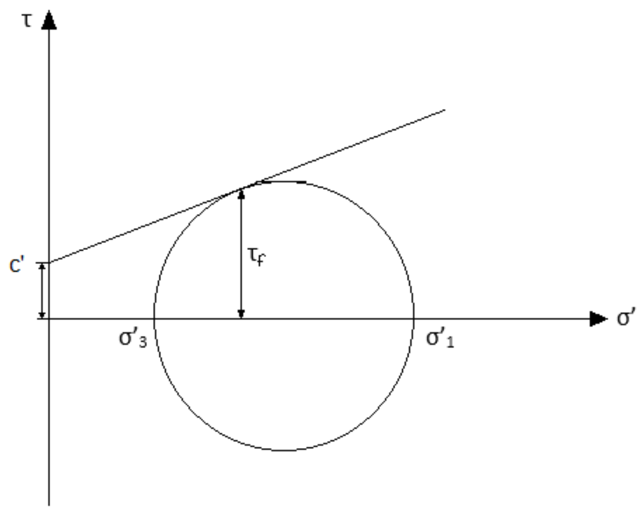


Figure 2.2: The Mohr-Coulomb failure criterion.

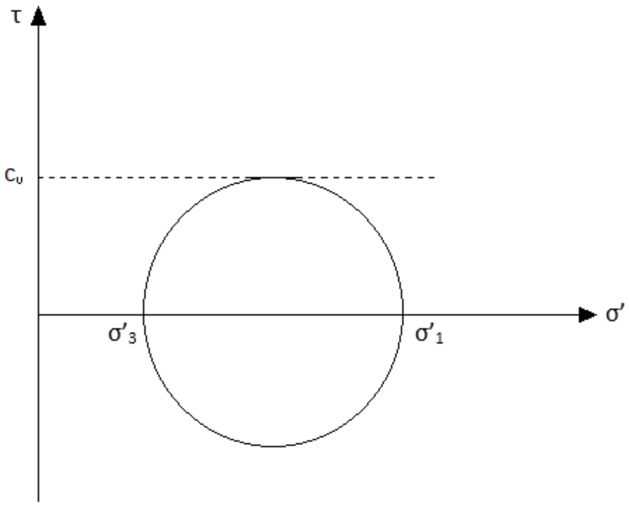


Figure 2.3: The Mohr-Coulomb failure criterion for cohesive soils.

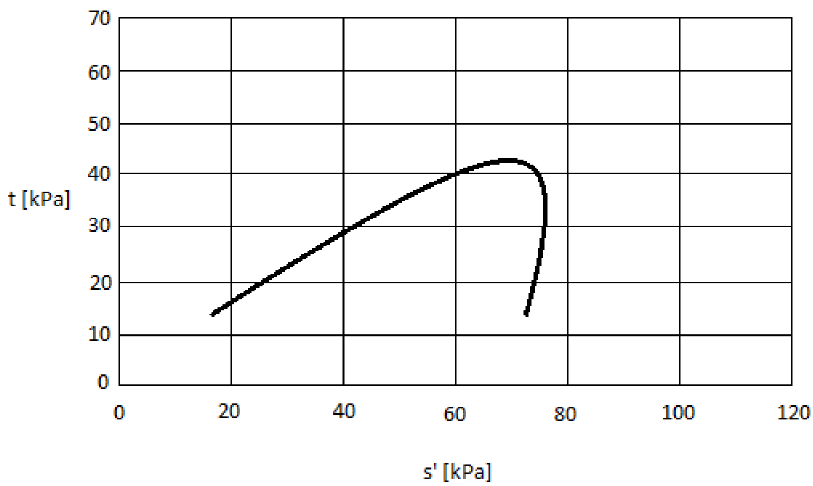


Figure 2.4: An example of a stress path typical for cohesive soils. Inspired by SGF, 2012.

Chapter 3

Methodology

In this section, the geotechnical methods relevant to this thesis, i.e., DPSH-A, piston sampler, triaxial tests, CPT and field vane, are explained. For each method, standards and recommendations are referred to. Furthermore, evaluation methods of shear strength are presented.

3.1 Geotechnical methods

3.1.1 Dynamic probing

A number of dynamic probing methods exist, but the principle of them all is that of a hammer falling from a certain height, hitting an anvil and thereby driving a rod downwards in the soil. The number of blows from the hammer necessary to penetrate the soil a certain distance is the output parameter that allows for an interpretation of the soil resistance and possibly the soil type beneath the surface (Bergdahl, 1984).

The International Society of Soil Mechanics and Foundation Engineering (ISSMFE) has standardised four types of dynamic probing, found in SS-EN ISO 22476-2:2005, where also the procedure itself is standardised. The differences consist mainly of the weight of the hammer, the fall height and the diameter of the cone tip. The four types are denominated, from lighter to heavier, DPL (dynamic probing light), DPM (dynamic probing medium), DPH (dynamic probing heavy) and DPSH (dynamic probing super heavy) (CEN, 2008). It was not until 1989 that the same organization published a so called *Recommended Test Procedures*, RTP, where weights and geometry of the equipment were specified, as well as blow ranges for each of the four types (Butcher et al., 1996). The type commonly used in Sweden is the DPSH-A, where nationally named the HfA-A method, superseding the older HfA-B method (comparable to DPSH-B) (Bergdahl and Dahlberg, 1973), with a blow range, as presented in blows/20 cm, of 5-100 for DPSH. As a comparison, the lighter methods have an accepted blow range of 3-50 blows. This range is stated as a recommendation, but should, nevertheless, only deviate in special cases and the procedure should be stopped if the number of blows deviate for more than one continuous meter, or if blows double the recommendation are required. Moreover, due to the importance of the rod to be driven vertically, standard specifications state the maximum inclination to 2 %, with 5 % accepted in especially difficult conditions. The standard further specifies the number of blows/minute to 15-30 and the rod rotation to 1,5 turns every meter, with an exception for heavy driving to 1,5 turns every 50 blows (CEN, 2008).

The method of super heavy dynamic probing (DPSH) has been used in Sweden since the 1940's, when the need for penetrating deeper into firm soils appeared. During this decade, the aspiration for building greater structures and hence the necessity for deeper foundations led to that the earlier method of weight sounding was considered insufficient. In 1971, *the Swedish Geotechnical Institution (SGI)*, introduced a new standard where the dynamic probing was divided into the older DPSH-B and the more modern DPSH-A. The DPSH-B was lacking in reliability and precision, which is why improvements developed for the DPSH-A method included, for instance, a fixed fall height and a standardised rotation of the probe (Bergdahl and Dahlberg, 1973). The DPSH-A method is now part of the Swedish, and European, standard SS-EN ISO 22476-2:2005 and standardised cones for probing are illustrated in Figure 3.1 (CEN, 2008).

Today the DPSH-A is mostly used for stratification purposes in predominantly coarse soils or clay till, soils where the CPT is not suitable (Gadeikis et al., 2010). According to Butcher et al. (1996) the most precise results are obtained by using the least heavy type possible, since the DPL gives a better resolution than the DPSH. Nevertheless, the lighter types (DPL, DPM and partially DPH) have limitations in their ability to penetrate firm soils, especially considering the presence of blocks, and thus DPSH has to be used (Butcher et al., 1996).

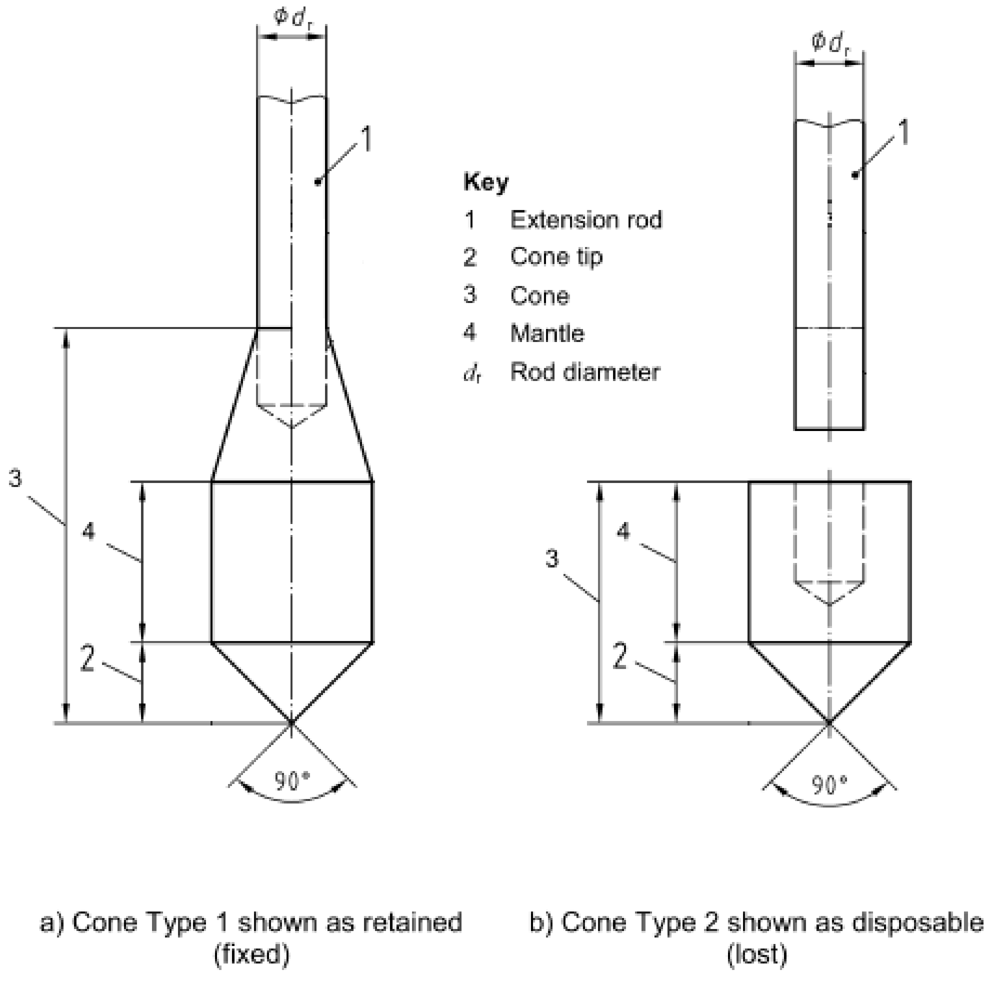


Figure 3.1: Dynamic probing cone alternatives according to ISO standards. Inspired by: CEN, 2008

Apart from the DP methods, another dynamic method is the standard penetration test (SPT), which is probably the in-situ test most commonly used in the world (Lingwanda et al., 2015). The SPT is based on the same principles as the DP methods, i.e., a hammer dropped from a certain height generating a number of blows necessary to penetrate the soil a certain distance. Nevertheless, the SPT has one advantage over the DP methods, consisting of the possibility to extract soil samples for further analysis (*Report of the ISSMFE Technincal Committee on Penetration Testing of Soils - TC16 with Reference Test Procedures CPT-SPT-DP-WST* 1989). This is possible thanks to the fact that the tip of the SPT is a cylinder instead of a cone as for the CPT. Due to the SPT being a world-wide method, many studies on correlations between CPT parameters as well as characteristics of the soil has been conducted and the SPT is widely used for evaluation purposes (Dalvi dos Santos and Bicalho, 2017; Lingwanda et al., 2015). However, the focus of this thesis is the dynamic probing, and therefore, SPT will not be discussed any further.

Evaluating the results

Results from dynamic probing are mainly presented in graphs with the number of blows per penetration unit plotted against the depth. The standard penetration units are 0,1 m and 0,2 m respectively for lighter types (DPL, DPM and DPH) and heavier types (DPSH) (Larsson, 2000). It is also common to present the resistance values r_d, the driving force when penetrating the ground, and q_d, the dynamic tip pressure when inertia of the equipment is considered (Bergdahl and Dahlberg, 1973; Butcher et al., 1996; Larsson, 2000; *Report of the ISSMFE Technincal Committee on Penetration Testing of Soils - TC16 with Reference Test Procedures CPT-SPT-DP-WST* 1989). These can be calculated as

$$r_d = \frac{mgh}{Ae}, Pa \tag{3.1}$$

$$q_d = \frac{m}{m + m'} r_d, Pa \tag{3.2}$$

where:
m = mass of fall weight, kg
g = gravitational acceleration, m/s^2
h = fall height of fall weight, m

A = cross section area of probe tip, m^2
e = average number of blows per penetration unit, m
m' = total mass of equipment, kg

Before using the registered number of blows, N_{20}, for further evaluation, the raw data should be corrected for torque friction according to Swedish practice. This is commonly done with an equation found in Bergdahl and Dahlberg (1973).

$$N_{20,netto} = N_{20} - 0{,}04T \tag{3.3}$$

Where:
$N_{20,netto}$ = Number of blows used for further calculations
T = measured torque, kNm

Furthermore, in order to calculate r_d, and later q_d, specific information regarding the equipment is needed. The information regarding the equipment used in this study was gathered from current standard and confirmed by the manufacturer (Carlson, 2018) of the equipment used (Table 3.1).

Table 3.1: Weights and measures of the DPSH-A equipment according to Swedish and European standards.

Weights and measures of DPSH-A	
m	63,5 kg
g	9,81 m/s²
h	500 mm
A	1600 mm²
m'	18 + 6 kg / 2m rod

3.1.2 Piston sampling

The piston sampling is in this project used to obtain undisturbed samples for testing with triaxial equipment and for routine tests in the laboratory.

The piston sampler procedure consists of a piston, containing tubes, continuously being pushed down into the soil to desired depth, where the piston locking mechanism makes it possible to open the piston and obtain soil samples in the inner tubes. From this method, undisturbed samples can be obtained. The purpose of the sampling is to acquire specimens with a quality high enough for use in laboratory testing, where commonly used methods are triaxial tests, direct shear tests and oedometer tests. In order for the laboratory tests to be feasible, the samples gathered need to have well preserved in-situ conditions, e.g., composition and water content (SGF, 2009b).

The standard piston sampler used in Sweden is based on a recommendation by the Swedish Geotechnical Society and is therefore not a standard, since the equipment is not standardized in detail but rather described, e.g. regarding geometry and function, in SGF (2009b). The standard piston sampler was developed in the early 1960's and, although further investigations and alternate versions have been assessed, it has been found that the standard piston sampler gives satisfying samples for most clays and clayey soils in Sweden. The piston sampler used today has very small alterations from the original standard sampler, though some details of the equipment and the procedure have been further specified. Two types of the standard piston sampler are in use today and have been named StI and StII. They are both subject to the recommendations by the Swedish Geotechnical Society and give the same quality of the specimen, nevertheless, mechanically they function partly differently. The main difference between these two is the mechanism for locking and releasing the piston when in operation (SGF, 2009b).

According to the mentioned recommendations, the standard piston sampler should have an inner diameter of 50 mm and a length of the piston of 700 mm. The outer piston contains three inner tubes of 170 mm each and an additional two outermost tubes half the length, which are not used due to the presumed disturbance in the soil in those areas. The piston is driven into the soil with a maximum velocity of 100 mm/s down to 1,5 meter above the predetermined depth for the sampling where the velocity is reduced to maximum 20 mm/s. The cutting edge of the piston is set to have a cutting angle of 5 degrees and is replaceable. This is especially convenient in firm or gravelly soils where the edge is repeatedly worn out (SGF, 2009b). A principle sketch of the Swedish standard piston sampler is illustrated in Figure 3.2.

The samples from the tubes should be pushed out carefully in a horizontal

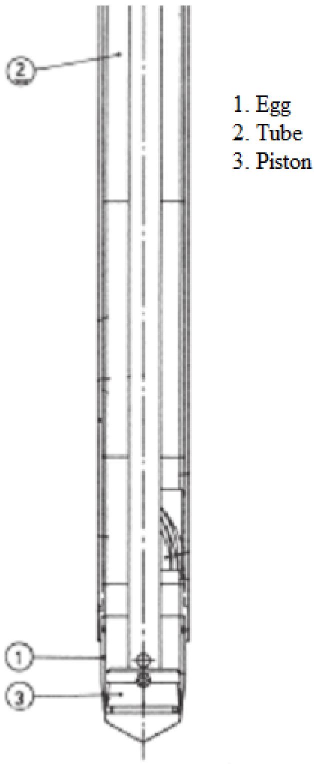


Figure 3.2: Principle sketch of the Swedish standard piston sampler. Inspired by SGF, 2009b.

position and placed in a cradle usually installed on the bore rig. Depending on the piston sampler used, the sample can be extracted with, e.g., a spring construction, but other methods are also common. It is important to follow instructions and recommendations when handling this part of the process since torque and shear stresses must be avoided. Furthermore, the soil samples should be cut off with a wire cutter in between the tubes in order to be able to seal them for conservation and transport to the laboratory (SGF, 2009b). The samples should be transported in specially designed sample boxes that decrease the risk for exposing the samples to heating, freezing or vibrations (Bergdahl, 1984; SGF, 2009b).

3.1.3 Triaxial test

Triaxial testing is an advanced laboratory method which can supply detailed and reliable results regarding strength and deformation parameters of a specimen. The test is based on the idea of increasing the load on a specimen axially and/or radially until failure occurs in the material. The vertical pressure is applied with a piston, while the horizontal pressure is controlled with a pressurized fluid. By recreating in-situ conditions such as stress and pore pressure, this method allows for a theoretically controlled basis when evaluating parameters, compared to other methods based on empirical relations (SGF, 2012). The procedure is usually conducted according to Swedish standard SIS-CEN ISO/TS 17892-8:2005 (CEN, 2005), but depending on the type of test other standards are also used.

The specimen

Specimen meant for triaxial testing are usually obtained by piston sampling. The longer the time between the extraction of the specimen and the testing with the triaxial equipment, the larger is the risk for the original in-situ conditions to have been altered. The triaxial test is preferably done within a week from the sampling for best results and no longer than one month afterwards for reliable results (SGF, 2017). According to European and Swedish standards on geotechnical investigation and sampling (*EN 1997-2:2007 Geotechnical design Part 2: Ground investigation and testing* 2007; *ISO 22475-1:2006 Geotechnical investigation and testing – Sampling and groundwater measurements. Part 1: Technical principles for execution* 2006), the specimen should be of quality class 1, the highest of 5 classes, and testing cat-

Table 3.2: Soil characteristics possible to evaluate related to testing category and quality class. Inspired by SGF, 2007.

<table>
<tr><th></th><th colspan="5">Quality class</th></tr>
<tr><th>Characteristics preserved</th><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td>particle size</td><td>x</td><td>x</td><td>x</td><td>x</td><td></td></tr>
<tr><td>water content</td><td>x</td><td>x</td><td>x</td><td></td><td></td></tr>
<tr><td>density, porosity, permeability</td><td>x</td><td>x</td><td></td><td></td><td></td></tr>
<tr><td>shear strength</td><td>x</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="3">Category according to ISO/FDIS 22475-1</td><td colspan="5">A</td></tr>
<tr><td colspan="2"></td><td colspan="3">B</td></tr>
<tr><td colspan="4"></td><td>C</td></tr>
</table>

egory A (from A to C) when conducting triaxial tests (SGF, 2007, 2012). The higher the class or the category, the better preserved is the specimen, where most laboratory testings require as little disturbance as possible (SGF, 2007). The classification refers to an overall level of preservation regarding, e.g., chemical composition, water content, porosity and stratification (SGF, 2012) and depending on the category and the quality class, different parameters can be evaluated (SGF, 2007), as displayed in Table 3.2. According to Swedish practice, all specimen in categories A and B should be denominated and subjected to a routine test, which includes, e.g., determination of bulk density, water ratio and liquid limit (SGF, 2007).

The specimen used in the laboratory should, according to European standards, have a geometry where the height is twice the diameter. Nonetheless, it is worth mentioning that other geometries sometimes are used, e.g., specimens where the height and the diameter have the same dimensions also exist (Larsson, 2000).

Test types

The specimen is confined in a triaxial cell, shown in Figure 3.3, with a thin membrane covering its walls in order for the pressurized fluid to act as a total stress and not a pore pressure. The upper and lower ends are in contact with porous stone lids that are connected to the drainage, which is kept open or closed depending on the desired test procedure (SGF, 2017). According to the European standard procedure, back-pressure is applied in order to assure the saturation of the soil volume (Larsson, 2000) and should represent the pore pressure in-situ (SGF, 2017). Assuring the complete saturation

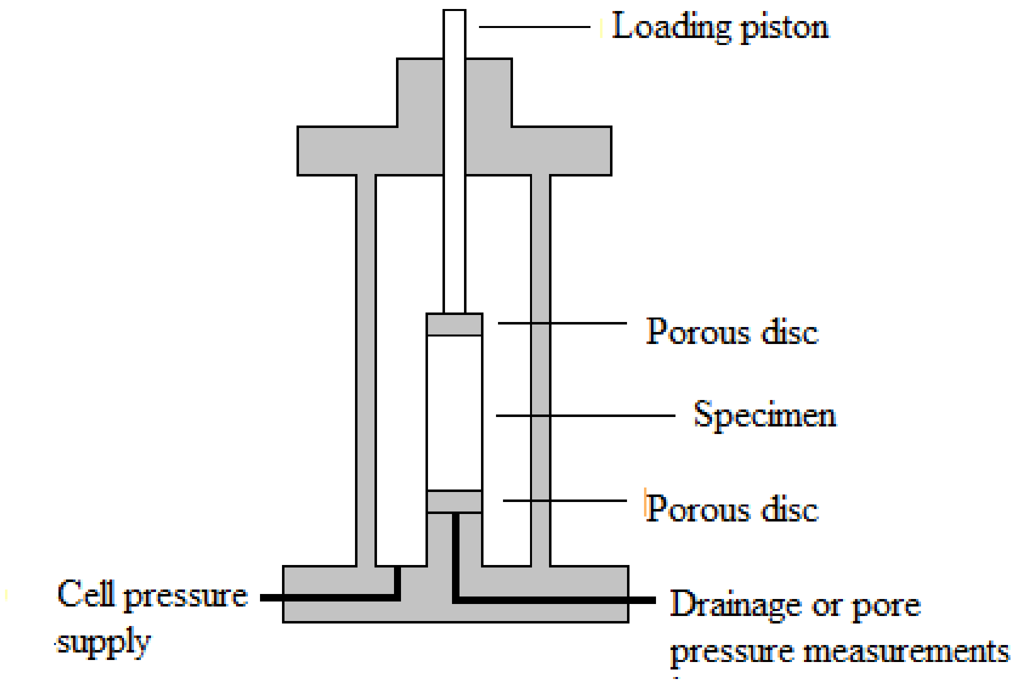


Figure 3.3: Illustration of a triaxial cell. Inspired by Craig and Knappett, 2012.

of the specimen can be important in order to be able to correctly measure the parameters in question (Larsson, 2000). Furthermore, the back-pressure prevents the registration of a so called false cohesion (Larsson, 2000), which results in an increase in strength due to the air in the pores of unsaturated soil. (*Jords hållfasthet* 2018).

The triaxial test can be conducted under drained or undrained conditions. The specimen in the triaxial cell is, as mentioned earlier, connected to a drainage that can be opened or closed, depending on the chosen situation. When testing under drained conditions, the pore water can freely leave the cell, giving a decreased volume but no change in pore pressure. If the drainage is kept closed, as for undrained conditions, no water can leave the cell and consequently, the pore pressure will increase (SGF, 2012).

The test itself is usually conducted as one of three principle variants:

Unconsolidated-Undrained (UU), *Consolidated-Undrained (CU)* and *Consolidated-Drained (CD)* (Craig and Knappett, 2012). During an *Unconsolidated-Undrained* (UU) triaxial test, the specimen is mounted in the triaxial cell and applied with a confining pressure assessed from in-situ conditions and then immediately loaded with vertical stresses. This allows for no drainage or consolidation to take place (Craig and Knappett, 2012), as this is considered unnecessary since the clay is assumed to preserve its characteristics regarding consolidation. A UU test can simulate short term conditions where rapid loading occurs and is common in clayey soils (*D2850 - 95; Standard test method unconsolidated-undrained triaxial compression test on cohesive soils* 1999). If the test is conducted as a *Consolidated-Undrained (CU)* test, the specimen is subjected to a confining pressure according to in-situ conditions during the initial consolidation phase, where the sample is allowed to consolidate. This is then followed by the shear phase where the external vertical stresses are applied. Not during any part of the test is the specimen allowed to drain (SGF, 2012). The *Consolidated-Drained (CD)* test is very similar to the CU, but with the difference that the specimen is allowed to drain, during both consolidation and shear phase (Craig and Knappett, 2012).

Undrained shear strength is the most common parameter used to describe the shear strength of cohesion soils. The parameter is evaluated as the highest pressure obtained before failure, something that usually occurs with a vertical compression of 1-5 %. (SGF, 2017). As mentioned before, the drainage of the cell is kept closed during an undrained test, resulting in no volume deformation but in an increased pore pressure (SGF, 2017). Parameters evaluated when conducting a drained triaxial test are the effective cohesion, c', and the effective friction angle, ϕ' (SGF, 2017).

Furthermore, triaxial testing can be done as active or passive tests where active is the most common test for clay. When conducting an active test, the axial pressure is increased while the radial pressure is kept constant. For a passive test, normally, the radial pressure is kept constant while the axial loading is decreased (SGF, 2012). Whether an active or a passive test is most appropriate depends on the stress situation in-situ as explained in Section 3.2 and illustrated in Figure 3.6. Active tests usually reveal a stronger undrained shear strength than passive ones and are the most commonly used alternative, especially since the area affected by active earth pressure is generally larger than the passive area (Larsson et al., 2007). The axial pressure is applied with a constant velocity, the Swedish recommendations suggest for clay a deformation of 0,7 % per hour in undrained tests, compared to 0,07 % in drained. Failure load usually occurs at a vertical compression of 1-2 %, with a higher percentage indicating a lower quality of the specimen, i.e., more or less disturbed (SGF, 2017). A higher percentage can also indicate a higher degree of heterogeneity, especially common in clay till where large particles can leave cavities in the walls of the specimen (Larsson, 2000). Passive tests are conducted with an increased horizontal pressure rather than with a vertical. The specimen normally reaches its ultimate load at an axial strain of around 3-5 %, a higher percentage than for active tests, resulting in a lower undrained shear strength (SGF, 2017).

Results from triaxial tests

The measurements made during a triaxial test can include applied stress, confining pressure, pore pressure and displacement. From these parameters can then be calculated axial, σ_a, and radial stress, σ_r, axial strain, ε_a and a change in volume, ε_{vol}. Whether there is a change in pore pressure or in volume is connected to whether the test has been conducted as an undrained or a drained test, since during a drained test the specimen is allowed to change its volume whereas the pore pressure is unchanged. The opposite is valid for an undrained test, where the volume is unchanged due to a closed drainage but the pore pressure varies.

When presenting the results from a triaxial test, the stresses and strains are usually displayed graphically as deviatory stress, q, shear stress, t, axial strain, ε, and effective mean stress p' or s' (SGF, 2012, 2017)

$$q = \sigma'_a - \sigma'_r \tag{3.4}$$

$$t = (\sigma'_a - \sigma'_r)/2 \tag{3.5}$$

$$p' = (\sigma'_a + 2\sigma'_r)/3 \tag{3.6}$$

$$s' = (\sigma'_a + \sigma'_r)/2 \tag{3.7}$$

For both drained and undrained tests, the stress path, i.e., q or t against effective mean stress, p' or s', and q or t against axial strain ε, are presented in graphs. Additionally, for an undrained test, the varying pore pressure is presented against the axial strain, whereas for a drained test it is the change in volume that is plotted against axial strain. It is also common to display stresses at failure load with Mohr's circle, albeit these only illustrate the stress in a specific moment and not the changing stress throughout the test. Therefore, it is sometimes recommended to display the results as stress paths (SGF, 2012). In geotechnical projects, the most common way of displaying the results are in a graph plotting the shear stress, t, against mean stress, s', while the $q - p'$-graph is more common in advanced material modelling (SGF, 2012).

During the test, the axial and the radial stresses are interpreted as major and minor principal stresses as it is assumed that no shear stresses affect the walls of the specimen. During an active test, the axial stress acts as the major stress and the radial as the minor, whereas during a passive test the relation is inverted. The undrained shear strength is evaluated as the highest shear strength reached during the shear phase if the failure exhibits a distinguishable peak and as a certain percentage of the highest value if no peak is detected (SGF, 2012).

3.1.4 Cone penetration test

The cone penetration test (CPT) is a widely used method within geotechnical surveys with well-established relations to evaluate shear strength and friction angles as well as the stratification of the soil (Robertson and Cabal, 2015). The procedure consists of a probe penetrating the soil while the tip pressure is being continuously registered and it is a method with good repeatability that also offers rapid execution and economic viability (Lingwanda et al., 2015). Apart from the original CPT, there are other versions that can include measurements of pore pressure (CPTu) as well as seismic measurements (SCPT) in the procedure. According to a recommended standard from The Swedish Geotechnical Society, the test should be conducted as a CPTu (Larsson, 2007) and according to current standard SS-EN ISO 22476-1:2012 (CEN, 2015). The method can be used in a variety of soils, although, the usage in coarser soil than sand, as well as in urban areas, can be limited (SGF, 2009a). The sounding is conducted with a probe with a cross section area de-

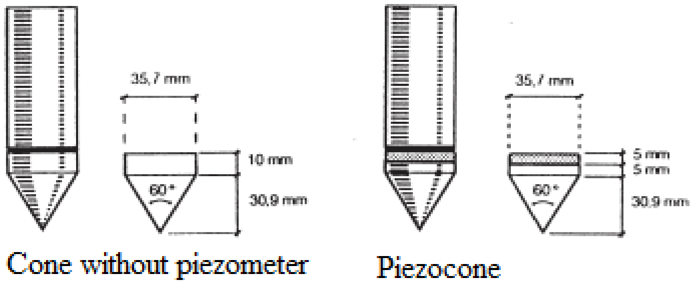


Figure 3.4: Illustration of a CPT cone (left) and a CPTu (right). Inspired by Larsson, 2007.

pending on intended depth and soil type (Robertson and Cabal, 2015). Most common sizes are the 1000 mm^2 and the 1500 mm^2 probes (Robertson and Cabal, 2015), with the 1000 mm^2 probe being preferred in Swedish standards (Larsson, 2007).

The standard in use in Sweden further specifies the cutting angle to 60 degrees and a penetration velocity of 20 mm/s ± 10 % (a slightly larger variation is accepted if the measurements are done without a pore pressure registration). In saturated soils, the pore pressure, u, is measured together with mantle friction, f_t, and the cone resistance, q_c. The latter is later transformed to a total tip pressure, q_T, where the cone resistance is corrected for pore water pressure, in order to evaluate the undrained shear strength. The measurements are taken electrically and should be conducted with such frequency that a continuous readout can be obtained. In order to evaluate any characteristics of the soil it is also necessary to have knowledge regarding existing pore pressure, u_0, and in-situ vertical stresses, σ_{v0}. Recommendations for clay and organic soils further stipulate the evaluation of the liquid limit, w_L, for a more detailed assessment (SGF, 1992). The depth of penetration depends largely on the stiffness of the soil and the presence of boulders or layers of coarse material (SGF, 1992, 2009a). Greater depths can be reached by decreasing rod friction with, for example, drilling mud or an expanded coupling (SGF, 1992).

The shear strength of clay is, in Sweden, often evaluated according to

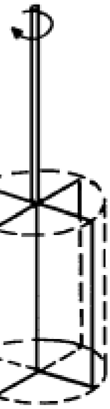


Figure 3.5: Principal sketch of a field vane and the assumed surface of failure in the soil. Inspired by SGF, 1993.

TK GEO which is based on the measured total tip pressure, q_T, the total vertical stress, σ_0, the liquid limit, w_L, and the over consolidation ratio, OCR, (Karlsson and Moritz, 2016a) as

$$c_u = \frac{q_T - \sigma_0}{13{,}4 + 6{,}65 w_L} \left(\frac{OCR}{1{,}3}\right)^{-0{,}20} \tag{3.8}$$

3.1.5 Field vane shear test

Shear strength of cohesive soils can be evaluated in-situ with a field vane shear test where the equipment consists of a vane with four perpendicular arms connected to a rod, as illustrated in Figure 3.5. The field vane is pushed downwards until reaching desired depth after which measurements of the shear strength can be initiated by rotating the vane (SGF, 1993).

The rod with the connected vane should be operated without blows, rotation or vibration during the phase where the rod is being pushed down. The velocity should not exceed 1m/60sec in order to assure vertical alignment and before starting the measurements by rotating the vane, a 2-4 minute waiting period is prescribed (SGF, 1993). The relation between the height and the diameter of the vane should be 2,0 with a maximum size of the vane of 100×200 mm and a minimum size of 40×80 mm. If the top layer of the soil profile consists of a dry crust or made ground, pre-drilling should be conducted in order to avoid any damage of the vane (SGF, 1993).

By measuring the torque required for failure to occur in the soil, undrained shear strength, remoulded shear strength and sensitivity can be evaluated on specified depths. The equation normally used in Sweden for evaluating shear strength from a field vane test is (SGF, 1993)

$$\tau_v = \frac{6M_{max}}{7\pi D^3} \tag{3.9}$$

Where M_{max} is the failure momentum and D is the diameter of the circle created by the vanes when rotating.

Corrections should be done both for rod friction and for the liquid limit, w_L, of the soil. The former is applied directly on the measured torque, while the evaluated shear strength is corrected for the liquid limit by a correction factor applied afterwards. In case of a heavily over-consolidated soil, the result should be corrected also for the OCR (Larsson et al., 2007) as

$$c_u = \tau_v \left(\frac{0{,}43}{w_L}\right)^{0{,}45} \left(\frac{OCR}{1{,}3}\right)^{-0{,}15} \tag{3.10}$$

With the requisit:

$$\left(\frac{0{,}43}{w_L}\right)^{0{,}45} \geq 0{,}5 \tag{3.11}$$

3.2 Evaluation of undrained shear strength

Traditionally, the shear strength of clay in Sweden has been determined by field vane and laboratory fall cone tests, today also supplemented with CPT. The shear strength used for further evaluation and designing is evaluated as a chosen mean value, where the relevance of each method has been weighed in and an appropriate factor, depending on the number of methods used, has been applied. In order to evaluate the plausibility, the obtained mean value should be compared to expected shear strength based on experience. If the investigated material is anisotropic, the shear strength is affected and consequently, an assessment of the relevance of the obtained mean value is necessary for further use of the evaluation. If the anisotropic nature of the soil is considered influential on the shear strength, the variation can be taken into account by confirming laboratory tests or by choosing values in the lower spectra. Anisotropy of the soil has shown to have a greater impact on the shear strength when the degree of plasticity is high (Larsson et al., 2007).

Both undrained and drained shear strength can be of importance and the designing parameter depends on the stress situation (Larsson, 2000; Larsson et al., 2007). Usually, normally and slightly over-consolidated cohesion soils require to be evaluated with undrained shear strength, while the drained case has more relevance when considering long-time effects or when the soil is strongly over-consolidated or consists of friction material (Larsson, 2000, 2008).

Furthermore, the shear strength of the soil depends on the stress situation, where active, passive and direct shearing has to be considered (Larsson, 2008). The stress situation varies depending on the position of the soil element in question, for instance its position relative to the failure surface, as is shown in Figure 3.6 (SGF, 2012). The active loading case is usually representative when evaluating the strength of steep slopes (SGF, 2007). Passive tests with triaxial equipment are mostly relevant in highly over-consolidated cohesion soils but can also be designing in soils with a high water pressure (Larsson et al., 2007). According to the Swedish Geotechnical Society SGF (2017), the passive shear strength is also important to consider when evaluating the strength of the soil regarding sliding surfaces in a passive zone.

Swedish clays are normally assumed to be slightly over-consolidated. The strength of the soil depends largely on the geological formation and on the loading history, which explains the characters of Swedish clays, with stiff appearances and high values of shear strength on glacial clay and softer appearances of post-glacial clay (Fredén, 2002). Clay can be categorised according to its evaluated shear strength, where a value under 40 kPa indicates a low shear strength while a value above 150 is considered as very high (Larsson et al., 2007).

Because of the formation process of the Swedish geology, with the weight of the overlaying ice creating an enormous pressure on the land, the empirical relations for evaluating soil characteristics are highly specific for the

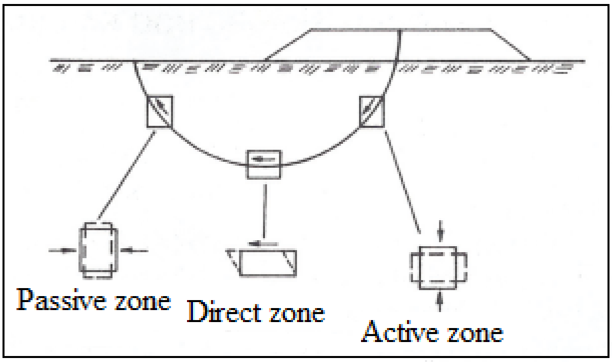


Figure 3.6: Type of triaxial test according to stress situation. Inspired by SGF, 2012.

country. Empirical formulas used in Sweden are mainly based on national experience, even though experience from other Nordic countries have been considered accurate. Nevertheless, site or project specific empirical evaluations often give the best parameters for local conditions. When evaluating the shear strength of cohesive soils, a parameter important to assess is the pre-consolidation pressure, which is normally evaluated in oedometer tests, CRS-tests or can be roughly estimated from CPT measurements. Evaluation methods based on empirical research need to be corrected, most commonly for the liquid limit, w_L, but also for the over consolidation ratio, OCR, when strongly over-consolidated (Larsson et al., 2007).

The guiding document *TR GEO*, published by the *Swedish department of traffic (Trafikverket)*, declares the possibility to evaluate the shear strength on an empirical basis, based on the pre-consolidation and the liquid limit (Karlsson and Moritz, 2016b).

$$c_u = a\sigma'_c \tag{3.12}$$

Where a is a constant, dependent on soil type and, for clay, also dependent on the stress situation, specified as active shearing ($a = 0{,}33$), direct shearing ($a = 0{,}13 + 0{,}17w_L$) and passive shearing ($a = 0{,}06 + 0{,}23w_L$).

If the soil is over-consolidated, the OCR can be taken into account by dividing the previously mentioned equation (3.12) with the OCR value to the power of a constant b, empirically decided to 0,8, as shown in Equation 3.13 (Karlsson and Moritz, 2016b).

$$c_u = a\left(\frac{\sigma'_c}{OCR}\right)^{1-b} \tag{3.13}$$

Chapter 4

Previous studies

In this chapter, a brief overview of previous studies is presented. Most of them treat the correlation CPT-DPL and are conducted on sandy soils. Nevertheless, these studies serve as a basis of knowledge and understanding and lead up to the last study presented, which is similar to what is studied in this thesis and will be used in the analysis of the obtained results.

Empirical relations between geotechnical methods are widely discussed in literature, albeit comparisons including dynamic probing methods are somewhat scarce. Within dynamic probing, the DPL seems to be the most commonly discussed method, probably since it is one of the most preferred alternatives internationally. A range of literature claim a strong correlation between DPL and CPT, in many different types of soil, considered even more reliable than the much more analysed correlation CPT-SPT (Dalvi dos Santos and Bicalho, 2017; Lingwanda et al., 2015).

The number of blows per penetration depth, N_{DP}, can be correlated directly with the cone resistance, q_c, registered by the CPT, as mentioned by Dalvi dos Santos and Bicalho (2017) when referring to earlier studies. According to those studies, the ratio $k'_e = N_{DPL}/q_c$, has been suggested to, for example, $k'_e = 0{,}1$ (Martins and Miranda, 2003) and $k'_e = 0{,}46$ in clayey sands(Lingwanda et al., 2015). Dalvi dos Santos and Bicalho (2017) further point out that although a comparison between registered blows, N_{DP}, with the cone resistance, q_c, from CPT, might be of interest, a transformation from N_{DP} to the dynamic tip pressure, q_d, might be an even more relevant parameter to use for comparison. This transformation from N_{DP} takes into account the fact that DP is a dynamic method which is dependent of the fall height and the hammer mass. Dalvi dos Santos and Bicalho (2017) further refer to Waschkowski (1983) who suggests that the ratio k_e, defined as q_d/q_c, being 1, specifying that the derived dynamic cone pressure from the DP is equal to the cone pressure measured by the CPT (Dalvi dos Santos and Bicalho, 2017). The same ratio has been suggested by Butcher et al. (1996), although Swedish experience has shown uncertainties regarding the validity of these ratios in Swedish clays (Larsson, 2000). Dalvi dos Santos and Bicalho (2017) also refer to studies conducted in sandy soils, where the ratio, k_e, was found to be 1,3-1,5 for DPSH depending on the relative density of the soil (Gadeikis et al., 2010) and a mean k_e' of 1,15 for DPH (Czado and Pietras, 2012). The study published by Dalvi dos Santos and Bicalho (2017) analyses sandy soils in Brazil and proposes the ratio between q_c and N_{DPL} to 0,23, with a high statistic security, and a k_e-ratio of 2,25, albeit with less statistical security. Nevertheless, a k_e of 2,25 is similar to those ratios found in other studies and the authors therefore argue that it should be considered reasonable (Dalvi dos Santos and Bicalho, 2017). Even though those numbers are higher than the k_e suggested by Waschkowski (1983) and Butcher et al. (1996), another study claims that the ratio is also related to the homogeneity of the soil, with a higher number indicating a more heterogeneous material (Viana da Fonseca, 1996). A study in clayey sands in Tanzania, shows a strong CPT-DPL correlation when analysing the k_e' ratio. The correlation was found even more accurate when considering the friction component for CPT (Lingwanda et al., 2015).

The article *SPT capability to estimate undrained shear strength of fine-grained soils of Tehran, Iran* investigates the correlation between N_{SPT} and undrained shear strength in fine-grained soils in Iran. The authors conclude that there is a correlation, even though the initial statistical R^2-value is low, it increases when considering other factors such as water content, liquid limit and plasticity index (Nassaji and Kalantari, 2011).

The Swedish Geotechnical Society made a comparative study in 2009, where DPSH-A, CPT and JB-sounding were correlated in soils with the main fraction sand. In order to compare DPSH-A and CPT, a transformation from number of blows/0,2 meters to dynamic cone pressure, q_d, was made for the dynamic sounding. The chosen transformation is not stated in the report, which makes it difficult to make comparisons with the study. Nevertheless, the transformation can be assumed to follow recommendations in Swedish and international standards and is said to have given reliable results in the shallower parts of the soil (SGF, 2009a).

A common method to transform the dynamic probing parameter blows/ penetrated unit to a dynamic tip pressure, q_d, is to use the so called Dutch formula (Dalvi dos Santos and Bicalho, 2017; Kulhawy and Mayne, 1990)

$$q_d = \frac{mghN_{DPL}}{As}\frac{m}{m+m'} \tag{4.1}$$

Where:
m = hammer mass, kg
g = acceleration, m/s^2
h = fall height of fall weight, m
N_{DPL} = Number of blows per penetrated unit
A = cone tip area, m^2
s = penetrated interval, mm
m' = mass of equipment, kg

This is the method used in the previously presented studies in order to calculate the dynamic tip pressure, q_d, from the dynamic probing and the derivation of q_d has shown to give relatively similar results in various soils, with the reservation that lighter methods have a better resolution and thus more concordant results (Butcher et al., 1996). Nevertheless, other surveys have shown the necessity to interpret the results from cohesive soils and on great depths cautiously since the friction from the rod can be large in these situations (*Report of the ISSMFE Technincal Committee on Penetration Testing of Soils - TC16 with Reference Test Procedures CPT-SPT-DP-WST* 1989).

The data obtained by DPSH is frequently related to both CPT and SPT data since there are many well established relations between the characteristics of the soil and CPT and SPT respectively. Common practices to correlate these methods are, among others: to assume the methods give equal values, to correct the raw data from DPSH for friction or simply to add a correlation factor (Charles et al., 2010).

A comparative study in Lithuania was executed in predominantly sandy

Table 4.1: The parameter q_c, registered by CPT, correlated with the DPSH-A parameters N_{20} and q_d in sandy lithuanian soils. Inspired by Gadeikis et al., 2010.

Soil	Density	α	β
Medium coarse sand	Medium dense	2,3	1,1
	Dense	2,4	1
Silty sand	Dense	1,2	0,4
Fine sand	Loose	1,6	0,7
	Medium dense	2,1	1
	Dense	2,3	0,9
	Very dense	2,5	0,9

soils, with the purpose of establishing relations between CPT and DPSH parameters. The dynamic cone resistance was derived according to Equation 4.1 (Gadeikis et al., 2010).

By categorising the soil by density, correlations between the tip pressure registered by CPT, q_c, and DPSH-A parameters N_{20} and q_d, were established:

$$q_c = \alpha q_d \tag{4.2}$$

$$q_c = \beta N_{20} \tag{4.3}$$

Evaluated constants, α and β are presented in Table 4.1 (Gadeikis et al., 2010).

In 1995, Butcher et.al. (Butcher et al., 1996) conducted a study on dynamic probing in cohesive soils with the purpose to examine the dependency of the equipment (DPL, DPM, DPH or DPSH) and to correlate the obtained data with the shear strength of the soil. 10 sites, with well known soil characteristics were chosen, some of them in Great Britain and some of them in Norway. Five were denominated as soft clays and five as stiff clays. The study finds that the repeatability of the testing is very high and the dynamic point resistance (q_d), derived by using the Dutch formula presented in Equation 4.1), is to a high extent independent of the equipment used. Nonetheless, using a lighter equipment gives a higher resolution, easier interpretation and, consequently, better results. However, by correcting heavier methods for rod friction it is possible to improve the results. The shear strength of the clays was determined by small in-situ tests or by triaxial testing in order to correlate the derived dynamic tip pressure, q_d, from the DP methods with the shear strength. The results showed a clear distinction between the soft and the stiff clays, hence two different relations were proposed (Butcher et al., 1996).

$$c_u = \frac{q_d}{170} + 20, Pa \qquad \text{for soft clay} \tag{4.4}$$

$$c_u = \frac{q_d}{22}, Pa \qquad \text{for stiff clay} \tag{4.5}$$

During the correlation process it was suspected that the sensitivity of the clay impacted the results and thus a new relation was proposed, with the dynamic tip pressure, q_d, corrected for the sensitivity, S_t.

$$c_u = 0{,}045\frac{q_d}{S_t} + 10, Pa \tag{4.6}$$

This relation was found to correlate very well with the measured shear strength, independently of whether the clay was considered soft or stiff (Butcher et al., 1996).

These equations have begun to be implemented in Swedish practice, albeit, no studies have been conducted in Swedish soils and the relations are used with great caution (Larsson, 2000). For a wider implementation, these equations need further analysis and validation in Swedish clays. For this thesis, only the relation regarding soft clays is examined, since soft clay is what was found on the site of investigation. The sensitivity of the clay was not measured, and hence, no consideration regarding that parameter is taken.

Additionally, it is important to note that relations including DP are highly dependent on conditions in the area and therefore site specific (Dalvi dos Santos and Bicalho, 2017; Nassaji and Kalantari, 2011). A number of factors are relevant when evaluating material parameters and it can be difficult to consider, for example, general geology, strength, stiffness, normalization and treatment of data (Lingwanda et al., 2015). Furthermore, many geotechnical practitioners are concordant when arguing that the correlation between the methods are largely material specific and that special caution has to be taken when using the relations in cohesive soils (Charles et al., 2010).

Chapter 5

Field and laboratory study

In this section the field site area is described, as well as data on the field investigations together with data of the boring. Furthermore, the tests conducted with triaxial equipment at Lund University is comprehensively described.

The field investigation site is located in the northeastern part of the region Scania, see Figure 1.1. WSP has an ongoing project at the site where a bridge over an existing road is being designed. The investigations were conducted during February 2018 and the piston samples were then transported to the laboratory at Lund University for triaxial testing and to WSP laboratories in Halmstad and Gothenburg for routine tests. The triaxial testing was conducted by the author, while the testing realised in Halmstad and Gothenburg was conducted by laboratory personnel.

5.1 Field investigations

Since the construction site is situated on deposits of clay and the construction of the bridge requires embankments, a fairly extensive investigation program was initiated and a consideration regarding the need for stabilisation of the embankment due to settlements was also considered. The geotechnical investigations were conducted on each side of the existing road and extended outwards, following the alignment of the embankment. Used methods were DPSH-A, CPT, auger, field vane and piston sampling, in a total of eight bore holes, as shown in Table 5.1. CPT was conducted in seven of those, DPSH-A

Table 5.1: Field investigation plan.

	CPTu	DPSH-A	Auger	Field vane	Piston	GW-pipe
18W01	x		x	x		
18W02	x		x	x		
18W03		x				
18W04	x	x	x		x	x
18W05	x	x				
18W06	x	x	x		x	x
18W07	x		x	x		
18W08	x		x	x		

in four and field vane in another four. Furthermore, augers were decided to be used in six boreholes.

5.2 Local geology

The area mainly consists of clay, with an expected depth of 10-20 meters (*Jorddjupskarta 1:5000, SGU* 2018), but made ground is also expected in the top layer (Figure 5.1). The bedrock in the area consists of sedimentary rock, rich in carbonates (*Berggrundskarta, SGU* 2018).

The pre-gathered information was validated partly by the augers from the investigation site. Nevertheless, the clay shows less thickness than expected and was not found any deeper than 7 - 8 meters below ground surface. The stratification was assessed by both field personnel and in the laboratory and resulted in a conceptual model of the geology in the area, as illustrated in the Figure 5.2. The clay has been divided in two layers, with the top layer denominated as a dry crust, i.e. clay that has been dried out and subjected to wethering.

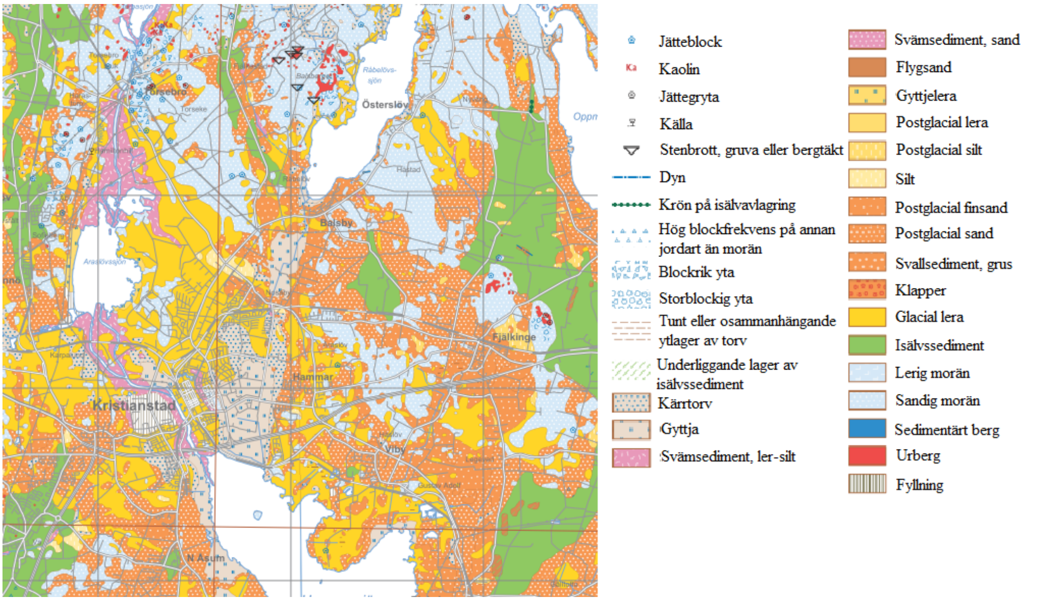


Figure 5.1: Soils in North Eastern Scania (*Jordartskarta 1:100 000, SGU* 2018).

5.3 Laboratory tests

Several augers were conducted as part of the geotechnical field investigation program, mainly for stratification purposes, but also to obtain disturbed samples from the site. These samples were then analysed with routine tests, conducted at the WSP laboratory in Halmstad as part of the bridge project. Furthermore, CRS and fall cone tests, together with routine tests, were conducted at undisturbed samples, in the WSP laboratory in Gothenburg. Unfortunately, no undrained shear strength could be evaluated for those samples due to difficult soil material. According to the laboratory technicians these specimen had a high content of silt, which obstructed the tests. Information obtained from the external laboratory tests that have been used in this thesis is presented in Section 6.1.

The specimen obtained by piston sampling were transported from the site of investigation to a pre-storage and then stored in a temperature controlled room for four weeks before initiating the triaxial tests.

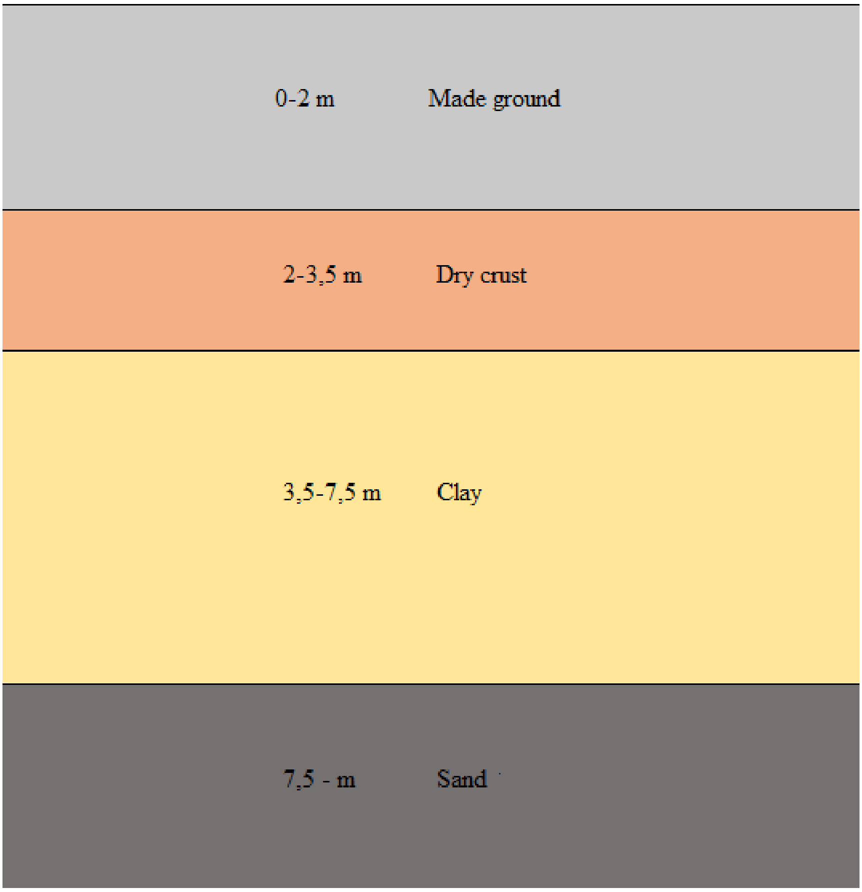


Figure 5.2: Conceptual geological model of the investigated area.

5.3.1 Triaxial tests

The selection of samples to be used for triaxial tests is mainly based on from which depths samples were extracted in the two boreholes 18W04 and 18W06, in order to conduct comparable tests from the same depths, and partly where middle tubes from the piston sampling is available. This led to two samples from each borehole to be chosen, on the depths five and six meters from borehole 18W04 and the depths four and six meters from borehole 18W06. This is assumed to be representative samples from a mid layer in the thickness of the clay, illustrated in Figure 5.3. However, due to complications in the laboratory, with the equipment needing adjustments and each test taking longer time than expected, the conducted number of triaxial tests are only two. Those two are conducted on the specimen from borehole 18W04. Even though fewer than initially planned for, the tests can give an indication of the accuracy of the studied relation. The triaxial tests are monitored by an in-house programme in LabVIEW, which provide a graphic interface to display technical experiments.

The tests were originally planed to be conducted as unconsolidated undrained tests (UU), which is common internationally when working with clay. This reduces the test to a shear phase only, since the consolidation phase is considered unnecessary because of the ability of the clay to preserve its in-situ stresses. However, after initiating the testing it was found that consolidated undrained tests (CU) was better suited for the extracted soil samples since they were unable to reach and stabilize to in-situ conditions. The test was therefore extended to both a consolidation and a shear phase. The tests are conducted as active tests, since this type of test is suitable when handling normally or slightly over-consolidated clay and considering the stress situation where vertical stresses are expected to be higher and active failure in the soil is more common.

Preparation

The density of the clay was determined in the WSP laboratory in Gothenburg in the two boreholes where the piston samples were taken. The analysis gave results between 1,69 and 1,86 t/m^3, but since these determinations were not conducted on the exact same depths as the triaxial testing, a simplified value

Figure 5.3: Specimen selected for triaxial testing marked with an X.

of 1,75 t/m^3 were chosen as representative for the theoretical mid-layer of clay and the density to be used for further evaluation.

According to evaluations based on the conducted CPT on the investigation site, the over consolidation ratio (OCR) seems to decrease with depth from a rather high ratio of 20-40 for the uppermost layer of clay to 2-3 for the deepest layers. The over consolidation ratio is interesting for the triaxial tests since a high OCR indicates horizontal stresses larger than the vertical stresses. An over consolidation ratio evaluated by CPT is generally considered as a rough estimation, but since for this thesis no other method of determination was possible, the CPT results were analysed and a representative OCR of 5, for the depths were the triaxial tests were conducted, was chosen based on mean and average values. The chosen value was then used in order to calculate the horizontal stresses in-situ, which also represents the confining pressure applied on the specimen in the triaxial cell. According to literature, horizontal stresses, when considering the over consolidation ratio, is calculated as

$$K_0 = K_{0NC} OCR^{0,5} \tag{5.1}$$

With:
K_0 = coefficient for lateral earth pressure at rest
K_{0NC} = coefficient for lateral earth pressure at rest when normally consolidated

K_{0NC} can be found in literature, varying depending on the type of soil in question. A value of K_{0NC} = 0,5 is suggested for varvey clay with traces of silt (Larsson et al., 2007), which is what was found on the site of investigation. With a chosen value of 5 for the OCR and a K_{0NC} of 0,5, the coefficient for lateral earth pressure at rest equals 1,12, indicating fairly isotropic conditions. With this type of condition, the confining pressure to be applied when executing the test is derived by multiplying the vertical in-situ stresses, calculated from density and thickness of overlaying soil, with the coefficient K_0.

Also of interest are the ground water levels. This is relevant since the specimen should be saturated when conducting undrained triaxial tests. Ground water pipes were installed in both borehole 18W04 and 18W06 and read three times, with the latest assumed to be the most reliable since the ground water level is expected to stabilize with time. The ground water levels were assessed to a depth of 1,6 meters in 18W04 and 2,2 meters in 18W06. This indicate that the selected specimen, with the shallowest from 4 meters below ground surface, should be fully saturated. The saturation of the specimen was then analysed, in the laboratory, with a B-test after mounting the specimen in the triaxial cell, illustrated in Figure 5.4. During a B-test, the confining pressure is increased, in this case 100 kPa was chosen as an appropriate increase, while the drainage is closed and the increase in pore pressure is thereafter examined. If the specimen is fully saturated, the increase in pore pressure should equal the augmented cell pressure. For this test, the specimen was considered fully saturated with a B-test indicating a ratio $\delta\sigma/\delta u$ over 98 %.

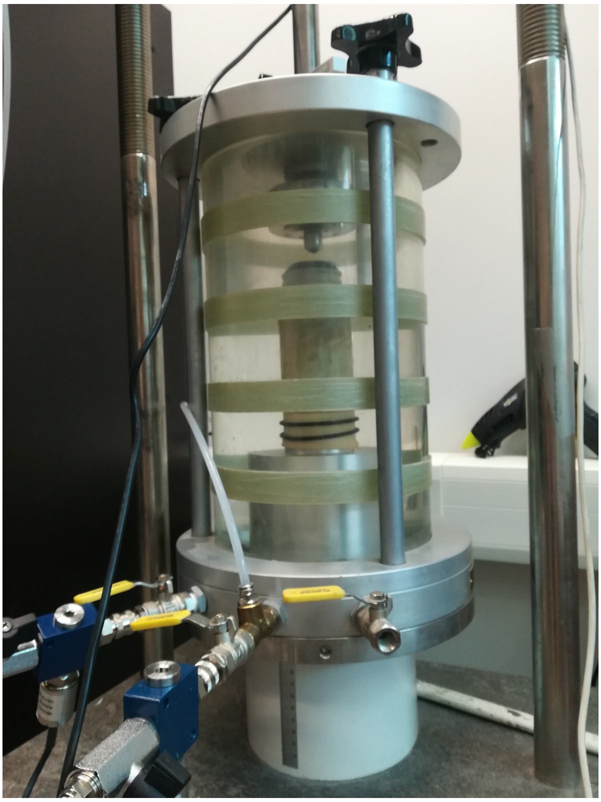


Figure 5.4: A specimen mounted in the triaxial cell at Lund University.

All of the specimen were deformed with the same velocity, originally planned to 0,01167 mm/min, causing a deformation of approximately 10,5 mm in 15 hours. The chosen velocity follows Swedish practice (SGF, 2017). Unfortunately, due to problems with the handling of the equipment, the shearing was conducted with the velocity 0,02 mm/hour and a deformation of 18 mm in 15 hours. Nevertheless, both tests were deformed with the same velocity and executed as strain controlled tests (*D2850 - 95; Standard test method unconsolidated-undrained triaxial compression test on cohesive soils* 1999).

In connection with the triaxial testing, diameter in three places and the height of the specimen were taken and the water content, i.e., the weight of the water related to the weight of the solid phase, of each specimen was also assessed and are presented in Chapter 6.

The input parameters for a presentation of triaxial tests, according to Swedish practice, are minor and major principle stresses, axial strain and change in pore pressure. While the latter was measured directly during the test and the horizontal stress was imposed, both axial strain and vertical stress were derived from indirect measurements. The applied confining pressure was evaluated as the minor principle stress, i.e., σ_3, while the major principle stress, σ_1, was derived by adding the confining pressure to the applied axial stress. The axial stress was derived from the registered force applied by the piston, divided by the area of the specimen at any given moment. The area of the sample was assumed to deform linearly based on the initial condition of constant shear velocity. With the initial height and area known, as well as the deformed height, the deformed area was calculated taking into account that the test was conducted as an undrained test and thereby presuming no change in volume and the preservation of a cylindrical shape.

Realization

GK2287

The clay felt saturated and held together nicely, apart from a layering 75 mm from the bottom. During the first phase of the test, the pore pressure did not stabilize and suspicions were raised that the membrane was broken. This led to the decision to put on a new membrane and thereafter continue the test. Furthermore, leakage was detected in two places and regulated before the pore pressure could stabilize. When the pore pressure was under control, several B-tests were conducted during several days, with the pore pressure given time to stabilize in between. Nonetheless, a satisfying level of saturation was unable to be reached. Therefore, it was decided to initiate the shear phase with a ratio $\delta\sigma/\delta u$ of 95 %.

KK1868

The second specimen felt saturated and relatively soft and the specimen held together nicely during extrusion and mounting. However, the pore pressure rose to very high levels, rising suspicions that the specimen was somehow disturbed and hence unable to reach its in-situ stresses on its own. A calculated effective total stress was then imposed on the specimen to consolidate it to in-situ stresses. When the pore pressure was stabilized, the b-test was run, indicating a saturation of around 95 % and, as for the first sample, this was considered sufficient and hence the shear phase was initiated. Even after several days of consolidation, the B-test did not indicate 100 % saturation for neither sample. Nevertheless, the samples were most probably fully saturated after such a time span and possible explanation to the lower values of the b-test is air in the system. Figure 6.10 illustrates specimen 2 before and after the shear phase. In Figure 6.11 - 6.13, the results are presented according to Swedish practice. Additionally, the results are displayed as Mohr circles in Figure 6.14.

Chapter 6

Results

The results from each method are interpreted and analysed and are presented below.

6.1 External laboratory results

Laboratory tests ran by WSP in Halmstad and Gothenburg that are of interest for the thesis are shown in the Tables 6.1-6.3.

Table 6.1: Density determined in laboratory.

Density		
Borehole	Depth [m]	Density [t/m^3]
18W04	4	1,86
	7	1,69
Chosen value: 1,75 t/m^3		

6.2 DPSH-A

The relation regarding soft clays, proposed by Butcher et.al. in their study on British and Norwegian clays from 1996 will be analysed in order to obtain an indication whether or not it can be validated or rejected as appropriate also to the studied clay in Scania. The results from DPSH-A are presented

Table 6.2: Average and median values of the OCR, as evaluated by CPT, as well as chosen value used for further evaluation.

Over consolidation ratio	
Average OCR	5,78
Median OCR	4,74
Chosen value	5

Table 6.3: Liquid limits, w_L, evaluated in laboratory.

Liquid limit, w_L								
	18W01	18W02	18W03	18W04	18W05	18W06	18W07	18W08
Lab Halmstad	50	52		0,5 (<4,2 m)		0,56 (<2,2 m)	0,54	0,56
				0,62 (>4,2 m)		0,58 (>2,2 m)		
Lab Gothenburg				0,49 (4 m)		0,62 (1,83 m)		
				0,76 (7 m)		0,52 (1,83 m)		

in N_{20}, r_d and q_d, as recommended in SS-EN ISO 22476-2:2005 (CEN, 2008), as well as in c_u as evaluated by

$$c_u = \frac{q_d}{170} + 20, Pa \tag{6.1}$$

Figure 6.1-6.4 below display the results, with the parameters mentioned above, from all four probings with DPSH-A. In Tables 6.4 and 6.5, only the depths where data is available from both DPSH-A and triaxial testing, are displayed.

Table 6.4: The parameters N_{20}, r_d and q_d from DPSH-A, on 5 and 6 meters depth in borehole 18W04.

18W04			
Depth [m]	N20 [nr]	r_d [kPa]	q_d [kPa]
5	2,8	2756	1759
6	3,5	3441	2196

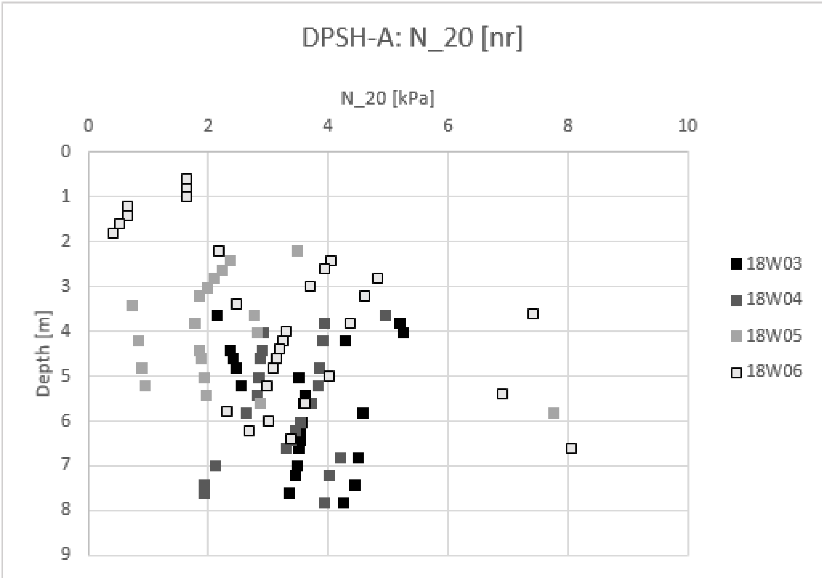


Figure 6.1: Number of blows per 20 cm penetration, N_{20}, from DPSH-A, for all depths in all boreholes.

Table 6.5: Undrained shear strength, c_u, evaluated from DPSH-A, on 5 and 6 meters depth in borehole 18W04.

18W04	
Depth [m]	c_u [kPa]
5	30,3
6	30,3

6.3 Triaxial tests

The results from the triaxial tests are presented below, for each specimen separately, in graphs illustrating shear strength (t) plotted against effective mean stress (s'), t against axial strain (ε_a), the change in pore pressure (Δu) against ε_a and as Mohr circles representing the initial, middle and end phase of the test. Furthermore, the depth, in-situ stresses, height, diameter and water content of the samples can be seen in Figures 6.6 and 6.8. Last, a perspicuous summary is given.

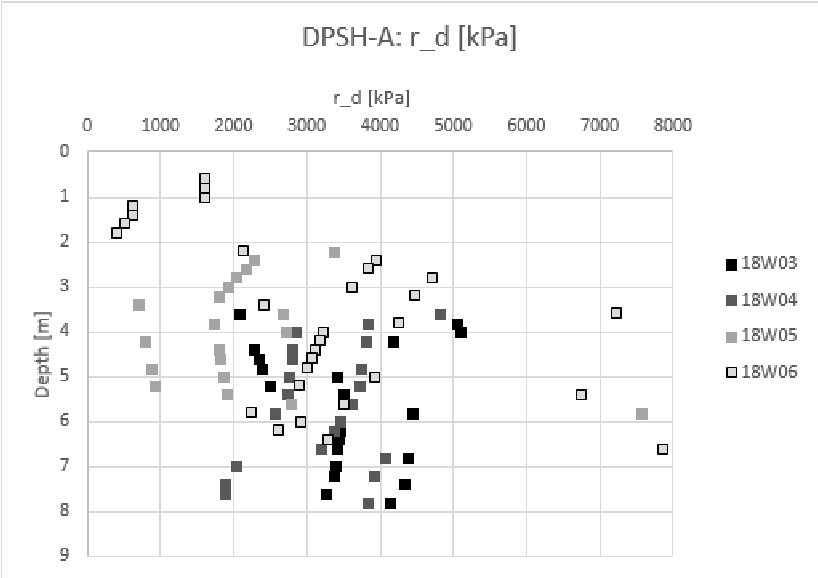


Figure 6.2: Tip resistance, r_d, from DPSH-A, for all depths in all boreholes.

Specimen GK2287

The first specimen was taken from borehole 18W04, from a depth of 5 meters with calculated in-situ stresses of 75,2 kPa vertically and 84,2 kPa horizontally. Measures and water content of the specimen are presented in Table 6.6.

Table 6.6: Height, diameter and water content of the sample GK2287.

Measures: GK2287	
Height [mm]	100,5
Diameter [mm]	50,4
Water content [%]	43

Figure 6.5 shows the sample before and after testing. In the photo after testing, the failure mode, consisting in a single shear band, is visible. In Figure 6.6 - 6.8, the results are presented according to Swedish practice. Additionally, the results are displayed as Mohr circles in Figure 6.9.

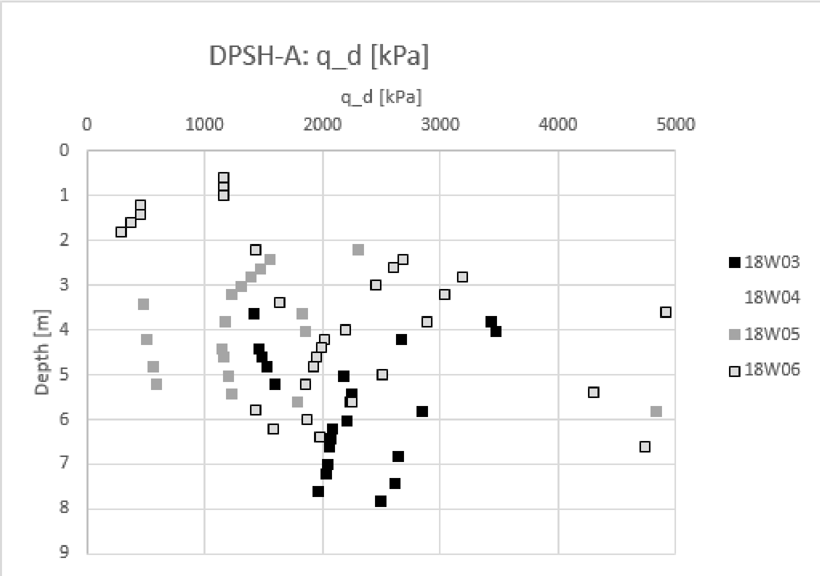


Figure 6.3: Dynamic tip pressure, q_d, from DPSH-A, for all depths in all boreholes.

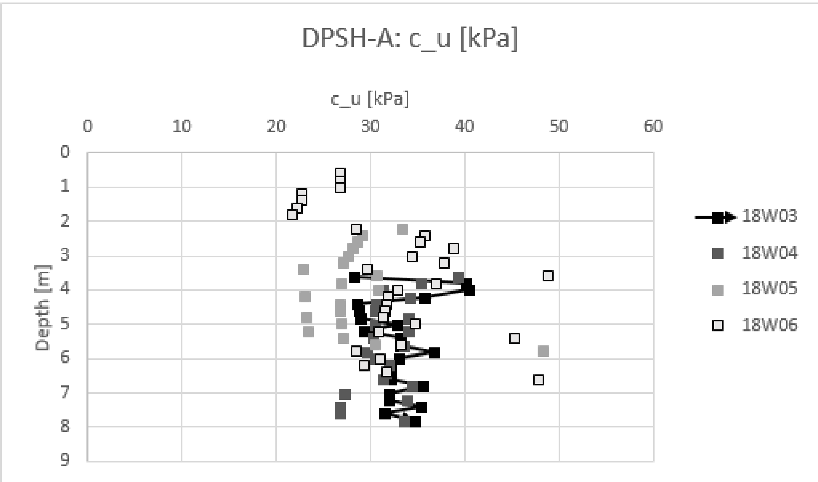


Figure 6.4: Evaluated undrained shear strength, c_u, from DPSH-A.

The undrained shear strength can be evaluated from the graphs as the highest shear strength reached during the test, since Figure 6.6 and Figure

(a) Before.

(b) After.

Figure 6.5: Specimen GK2287 before and after shearing.

6.7 both exhibit a distinct peak failure. The change in pore water pressure plotted against axial strain can be seen in Figure 6.8. In Figure 6.7 the evaluated undrained shear strength is presented together with axial strain at failure and the maximum change in pore pressure.

Table 6.7: Evaluated parameters from sample GK2287.

GK2287	
Undrained shear strength [kPa]	44
Axial strain [%]	3
Pore pressure change [kPa]	51

An evaluated undrained shear strength of 44 kPa is reasonable and an axial strain at failure of 3 % is considered normal and corresponds to a slightly disturbed specimen according to literature, where failure is predicted to occur at an axial strain of 1-3 % deformation for an active test. A change in pore

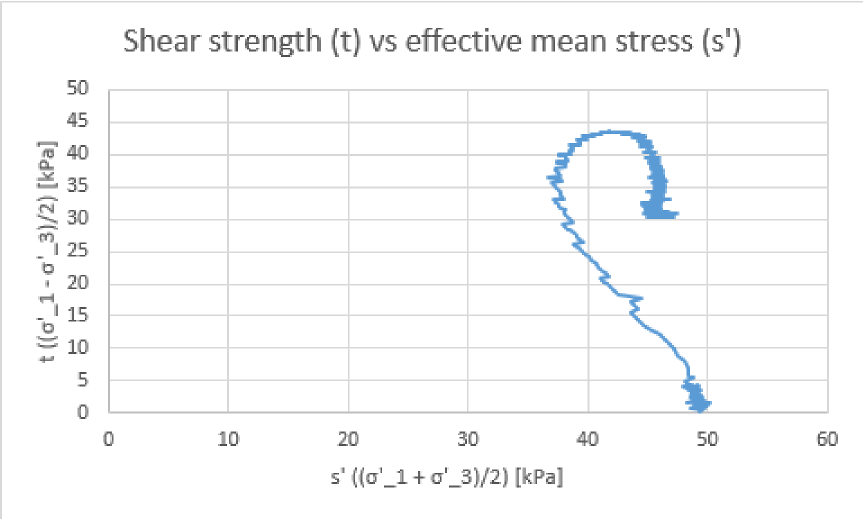


Figure 6.6: Shear strength plotted against effective mean stress for sample GK2287.

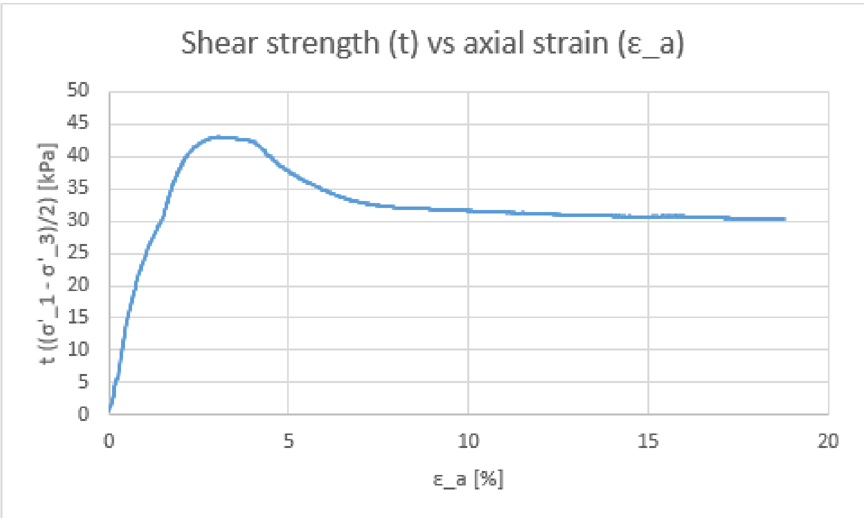


Figure 6.7: Shear strength plotted against axial strain for sample GK2287.

pressure that increases until failure occurs and then decreases, indicates that the clay is contracting until failure and then dilating.

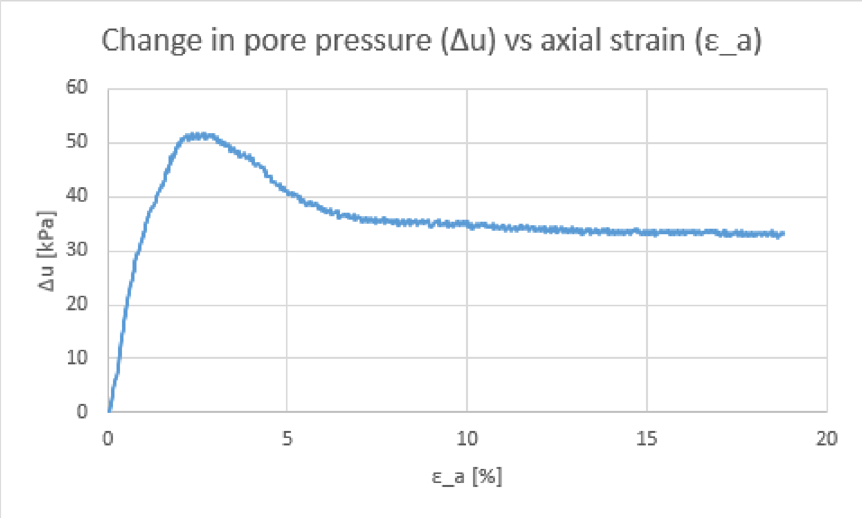


Figure 6.8: Change in pore pressure plotted against axial strain for sample GK2287.

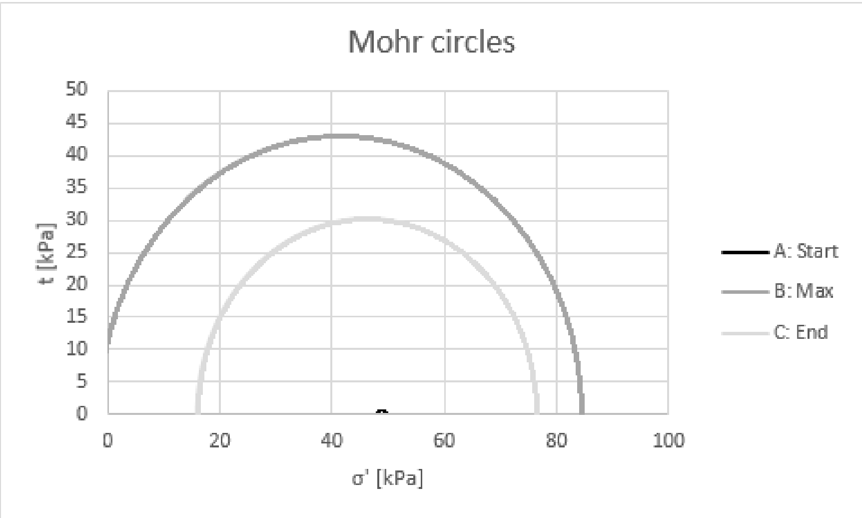


Figure 6.9: The shear strength of sample GK2287, displayed as Mohr circles, in its initial, maximum and end states.

Specimen KK1868

Specimen 2 was taken from borehole 18W04, from a depth of 6 meters with calculated in-situ stresses of 92,7 kPa vertically and 103,8 kPa horizontally.

Measures and water content of the specimen are presented in Table 6.8.

Table 6.8: Height, diameter and water content of sample KK1868.

Measures: KK1868	
Height [mm]	101
Diameter [mm]	50,9
Water content [%]	57

(a) Before.

(b) After.

Figure 6.10: Specimen K1868 before and after shearing.

Even though Figure 6.11 and Figure 6.12 differ slightly from the same graphs for specimen GK2287, they do exhibit a clear peak. Therefore, the undrained shear strength is evaluated as the highest shear strength reached during the test, i.e. 50 kPa. The change in pore water plotted against axial strain can be seen in Figure 6.13, while Table 6.9 displays the evaluated undrained shear strength together with axial strain at failure and the maximum

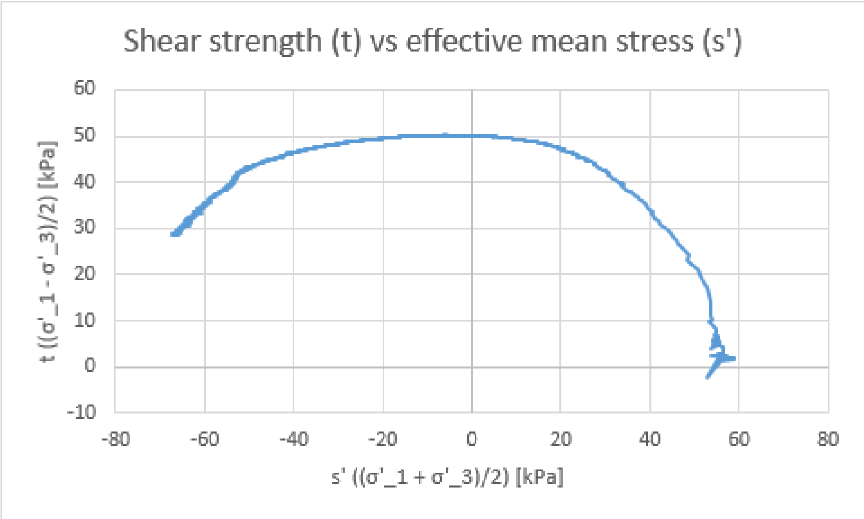


Figure 6.11: Shear strength plotted against effective mean stress for sample KK1868.

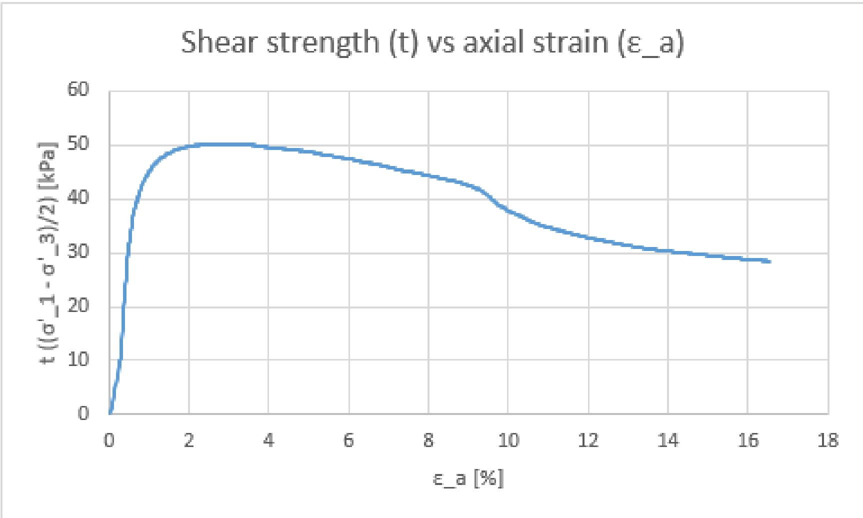


Figure 6.12: Shear strength plotted against axial strain for sample KK1868.

change in pore pressure.

The undrained shear strength is evaluated as peak shear strength. Nev-

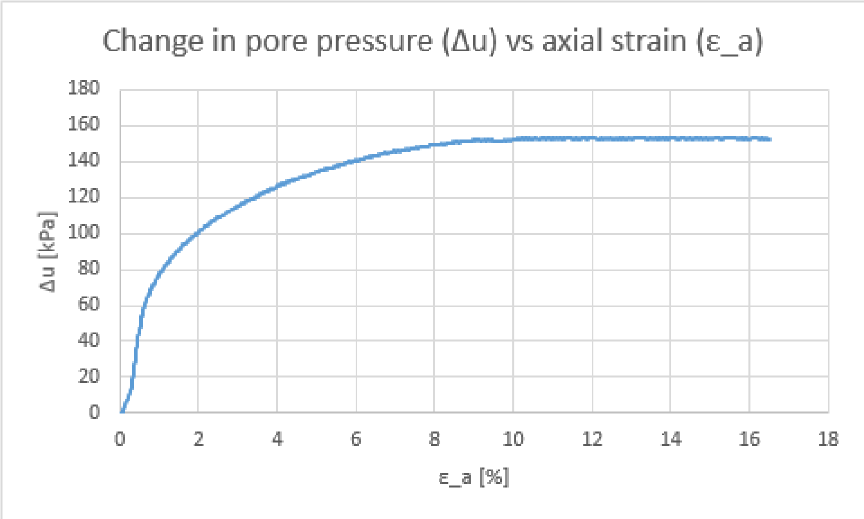


Figure 6.13: Change in pore pressure plotted against axial strain for sample KK1868.

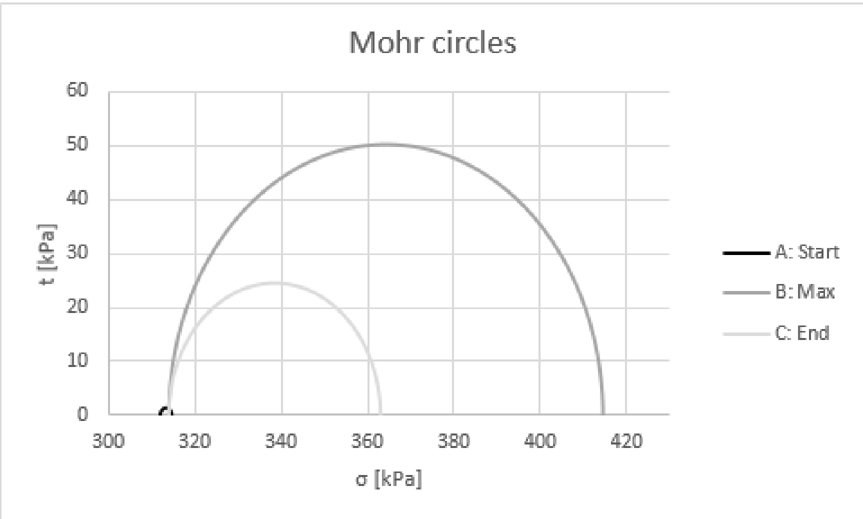


Figure 6.14: The shear strength of sample KK1868 displayed, as Mohr circles, in its initial, maximum and end states.

ertheless, the graph displays a somewhat different shape than for specimen GK2287, indicating a peak stress for less brittle failure for the second sample.

Table 6.9: Evaluated parameters from sample KK1868.

KK1868	
Undrained shear strength [kPa]	50
Axial strain [%]	3
Pore pressure change [kPa]	151

Figure 6.12 displays an axial strain of nearly 3 %, which indicates a somewhat disturbed sample, but not as disturbed as could have been expected, considering the time consuming consolidation phase. The pore pressure kept increasing during the test and reached a maximum value of 151 kPa, indicating a contracting behaviour. This differs from the pore pressure change during the triaxial test on specimen GK2287 and can possibly be explained by slightly different over consolidation ratios.

Compilation

The evaluated undrained shear strength from the two specimen are displayed graphically in Figure 6.15, as well as in Table 6.10.

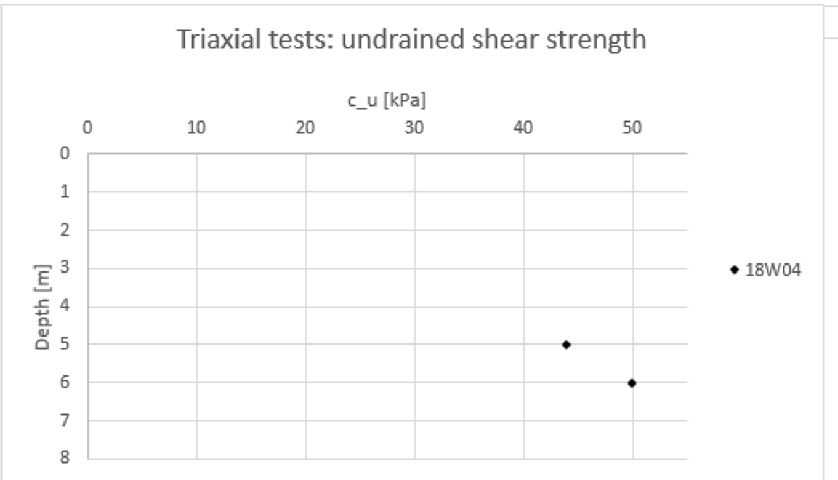


Figure 6.15: Evaluated undrained shear strength from the two triaxial tests from borehole 18W04.

Table 6.10: Evaluated undrained shear strength from the two triaxial tests from borehole 18W04.

Undrained shear strength [kPa]	
Depth [m]	18W04
5	44
6	50

6.4 CPT

The raw data from the CPT has been analysed with the programme Conrad, according to Swedish practice. Input parameters which improve the interpretations and evaluations are stratification and liquid limit, w_L, which have been obtained by field and laboratory studies and presented in previous sections. Total tip pressure and evaluated shear strength, from all seven boreholes, are presented in Figure 6.16 and 6.17 respectively.

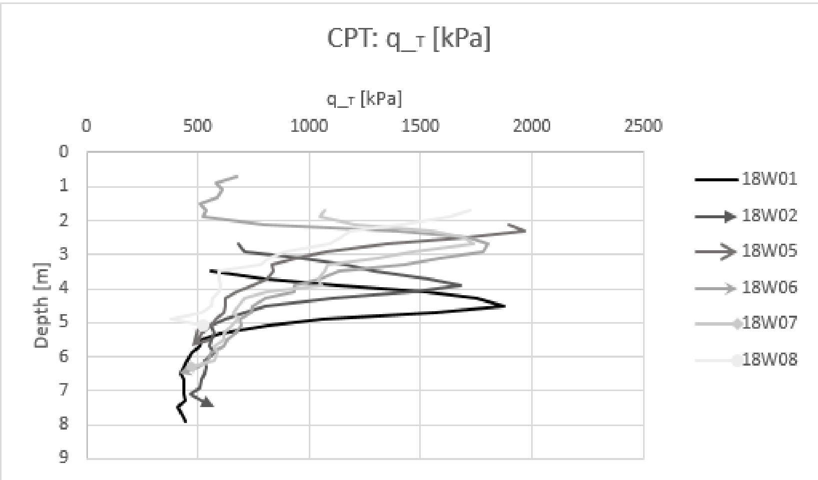


Figure 6.16: Total tip pressure, q_T, from CPT, for all depths in all boreholes.

CPT sounding was conducted only close to one of the two boreholes where piston samples were obtained, but due to difficulties with the triaxial testing, the CPT and the triaxial tests were not conducted in the same area. Nevertheless, since the clay has been interpreted as one geological formation with, more or less, the same characteristics throughout the layer,

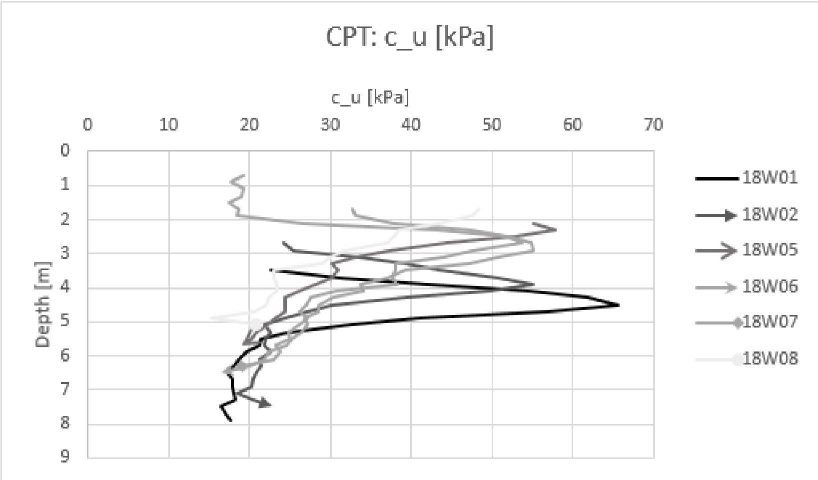


Figure 6.17: Evaluated shear strength, c_u, from CPT, for all depths in all boreholes.

a comparison is possible. Presented in Tables 6.11 and 6.12 are evaluated total tip pressure and evaluated undrained shear strength from all boreholes at the depths 5 and 6 meters, as well as an average, that corresponds to the depths where the triaxial testing was conducted.

Table 6.11: Tip pressure, q_T registered by CPT from all boreholes as well as an average, on the depths 5 and 6 meters.

Total tip pressure, q_T [kPa]							
Depth [m]	18W01	18W02	18W05	18W06	18W07	18W08	Average
5	940,0	591,0	569,1	691,2	669,5	452,3	652,2
6	461,0	553,7	-	554,1	579,5	-	537,1

Table 6.12: Undrained shear strength, c_u, evaluated by CPT from all boreholes as well as an average, on the depths 5 and 6 meters.

Undrained shear strength, c_u [kPa]							
Depth [m]	18W01	18W02	18W05	18W06	18W07	18W08	Average
5	36,9	23,1	22,8	27,1	26,7	18,1	25,8
6	19,2	21,9	-	22,2	23,5	-	21,7

6.5 Field vane

The sizes of the vanes used in the investigation in the area were 110x150 mm and 130x65 mm. The decision of when to use one size or the other was taken by the field operator, based on the satisfaction of the results displayed on the field computer. Corrected evaluated undrained shear strength is presented graphically in Figure 6.18, from all four boreholes where the method was conducted. Results from the depths 5 and 6 meters are also presented in Table 6.13, i.e., on the depths where also triaxial testing was conducted.

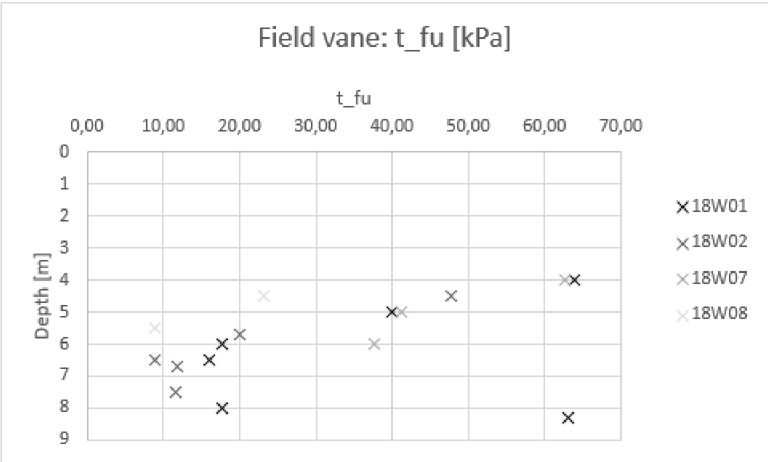


Figure 6.18: Corrected shear strength, τ_{fu}, from field vane tests, for specific depths in boreholes 18W01, 18W02, 18W07 and 18W08.

Table 6.13: Evaluated undrained shear strength from field vane tests, for the two boreholes where the test was conducted on the depths 5 and 6 meters.

Undrained shear strength [kPa]		
Depth [m]	18W01	18W07
5	39,9	41,2
6	17,7	37,6

Chapter 7

Analysis

All the information obtained on undrained shear strength of the clay is analysed and compared in this chapter.

7.1 DPSH-A - Triaxial tests

7.1.1 Undrained shear strength

As illustrated in Figure 7.1, the undrained shear strength evaluated by triaxial tests displays consistently higher values compared to DPSH-A, which can partly be explained by the fact that the triaxial tests were conducted as active tests. This type of test gives a value in the higher end of the range, while a passive one results in values in the lower end. The fact that the shear phase of the triaxial tests were performed with a slightly higher velocity than planned, might also have affected the evaluated undrained shear strength in a way that it was evaluated as higher than it should have.

7.1.2 The Butcher et al. relation

Figure 7.2 gives an indication of the accuracy of the relation suggested by Butcher et al. (1996), here displayed as the relation between the calculated dynamic tip pressure, q_d, from DPSH-A and the undrained shear strength evaluated from triaxial tests. As can be seen in Figure 7.2, the evaluated undrained shear strength in this thesis is consistently higher than the same parameter evaluated according to the relation by Butcher et al. (1996). Nevertheless, similarities, regarding gradient, can be detected, albeit best linear

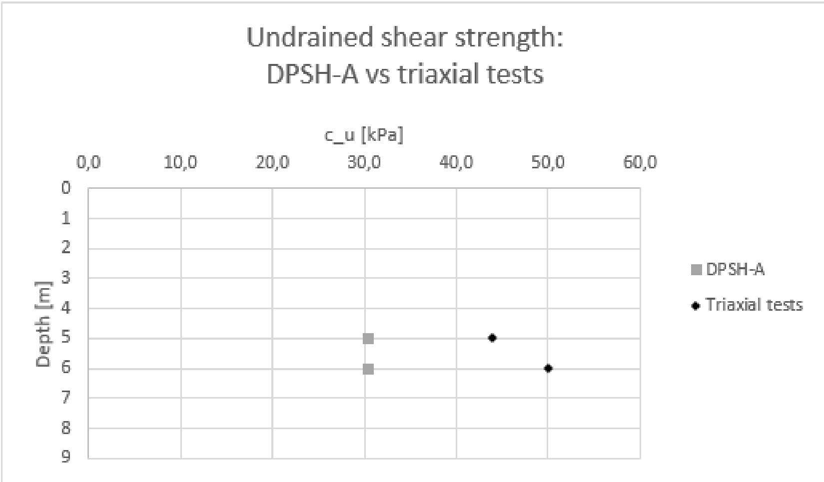


Figure 7.1: Evaluated undrained shear strength by DPSH-A and triaxial tests on the depths 5 and 6 meters.

fit is described by $c_u = \frac{q_d}{73} + 19{,}8$ is not the same as the $c_u = \frac{q_d}{170} + 20$ (Butcher et al., 1996). Even though best fitted line displays an R^2-value of 1, only two points of investigation are not, however, sufficient to represent a valid relation between to parameters with any certainty.

Previous studies have also seeked to develop a relation between the number of blows per penetrated unit, even though many studies have also pointed out that the parameter N_{20} might be inappropriate to use for further evaluation since the parameter does not consider the dynamic feature of the DPSH-A. Since this study only consists of two triaxial tests, it is not possible to make any meaningful intent.

7.2 DPSH-A - CPT

7.2.1 Undrained shear strength

Figure 7.3 displays the evaluated undrained shear strength, from all the boreholes where the two methods were conducted, i.e., also data from boreholes where only one of the methods was conducted are included. As has been

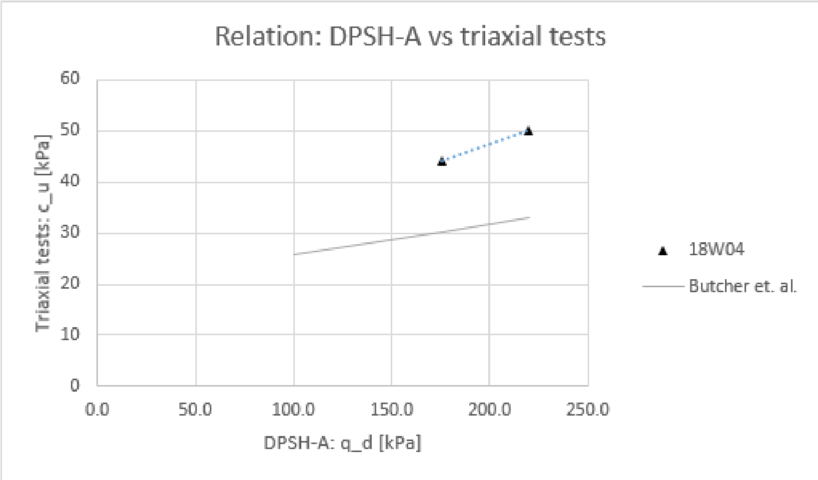


Figure 7.2: Comparing the relation suggested by Butcher et al. (1996) with the results from DPSH-A and triaxial tests for borehole 18W04 on 5 and 6 meters.

mentioned previously, only in boreholes 18W05 and 18W06 both CPT and DPSH-A were conducted, therefore, the undrained shear strength evaluated from these two boreholes are displayed in Figure 7.4. All graphs exhibit more or less the same trend, i.e., the undrained shear strength evaluated by DPSH-A is rather independent of depth while the undrained shear strength evaluated with CPT displays a clearly decreasing trend with depth. Nevertheless, they are displaying values in the same range. The lack of a decreasing trend in the DPSH-A results might owe to fact that the clay in the area is rather soft, resulting in the heavy equipment of DPSH-A being unable to register the number of blows correctly. As has been suggested in literature, the lighter the equipment of the dynamic probing, the better the resolution. It is possible that if DPL had been used, the two curves (CPT and DPL) would have concurred better.

7.2.2 The Butcher et al. relation

Figure 7.5 illustrates the relation between the dynamic tip pressure, q_d, from DPSH-A and the evaluated undrained shear strength by CPT in boreholes 18W05 and 18W06. Also displayed in the graph is the relation proposed by

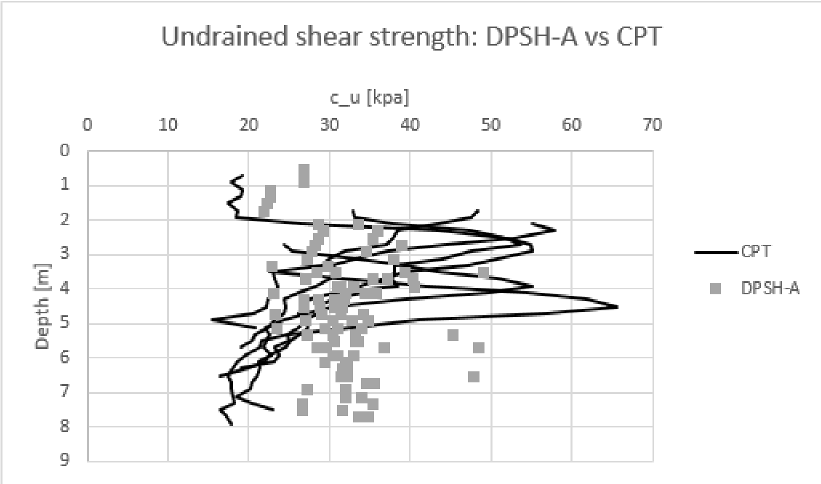
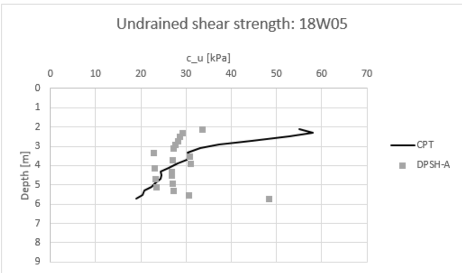
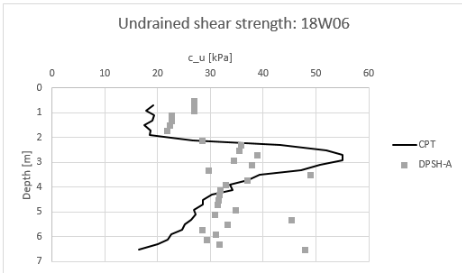


Figure 7.3: Evaluated undrained shear strength, c_u, from CPT and DPSH-A in all available boreholes.



(a) Borehole 18W05.



(b) Borehole 18W06.

Figure 7.4: Evaluated undrained shear strength, c_u, from CPT and DPSH-A in two boreholes.

Butcher et al. (1996). It can be observed that the data is largely scattered and fitted lines display low R^2-values of 0,25 and 0,39 for each borehole respectively. When removing data suspected to be outliers, the fitted lines change significantly, leading to an even less accuracy of the relation by Butcher et al. (1996), as is displayed in Figure 7.6. Best fitted lines were then evaluated as polynomial and exponential, with an R^2-value of 0,24 and 0,65 respectively, indicating a slightly less scattered set of data for borehole 18W06. Nonetheless, R^2-values in this range is considered very low and none of the relations can be said to be valid for a general case.

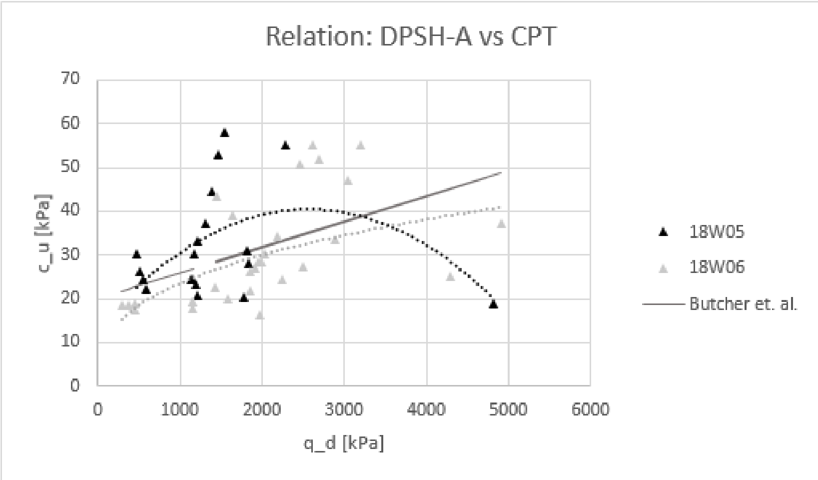


Figure 7.5: Comparing the relation suggested by Butcher et al. (1996) with q_d from DPSH-A and q_c from CPT for boreholes 18W05 and 18W06.

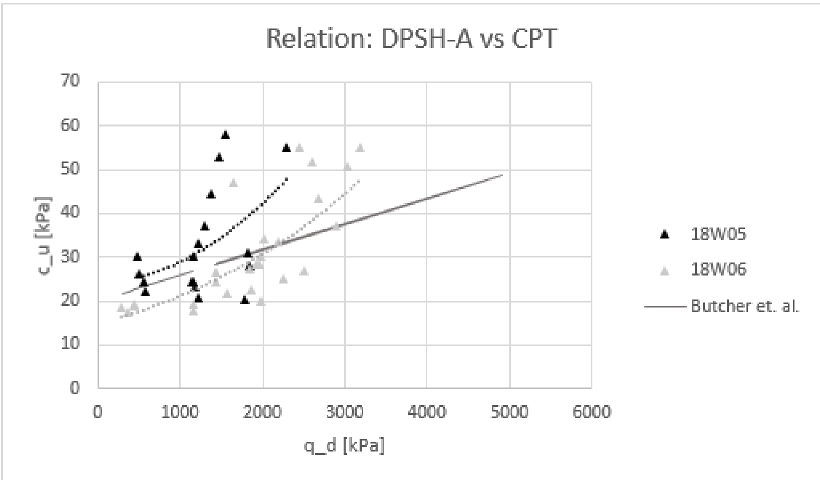


Figure 7.6: Comparing the relation suggested by Butcher et al. (1996) with the results from DPSH-A and CPT for boreholes 18W05 and 18W06, with outliers removed.

Comparing the results in this thesis with the results obtained by Butcher et al. (1996) a difference in scatter between the two studies can be observed. Even though Butcher et al. (1996) conducted more tests, the results exhibit less scatter than the graphs in this thesis.

7.2.3 The ratio k_e

The parameter k_e is defined as the relation between the tip pressure from CPT, q_c (i.e., uncorrected measured tip force/tip area), and the evaluated dynamic tip pressure from DPSH-A, q_d and is one way of relating the two methods to each other. In Figure 7.7, this ratio is plotted, separately for each borehole as well as an average value and as a chosen value. Studying the graph, it is obvious that the tip pressure varies significantly between the two methods and does not correlate easily. Furthermore, the increase with depth of the ratio k_e (Figure 7.7) follows the observation that the undrained shear strength interpreted by CPT exhibits a decreasing trend with depth, while the undrained shear strength interpreted by DPSH-A is independent of the same. Comparing the results with the studies described in Chapter 4, the values of k_e evaluated in this thesis, seems to be reasonable. It results slightly higher than proposed by Butcher et al. (1996) when analysing clay till and slightly lower than the k_e suggested in the studies considering sandy soils and using DPL.

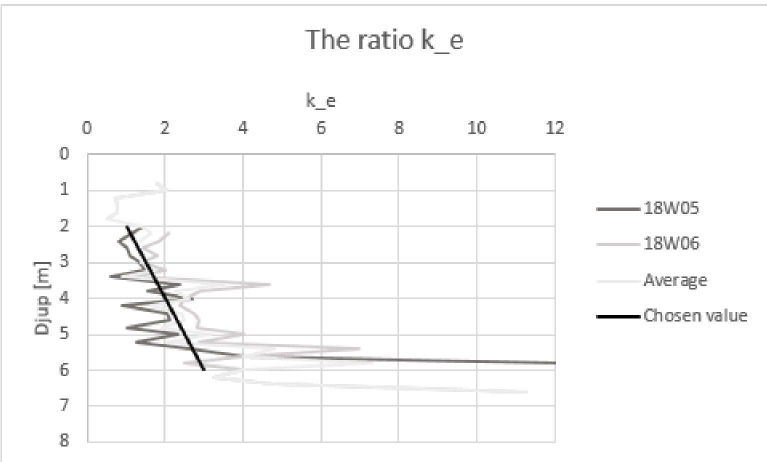


Figure 7.7: The ratio $k_e = q_d/q_c$ from boreholes 18W04 and 18W06.

The weight and geometry of the equipment might influence the results, especially considering that it has been argued before that lighter methods, such as DPL, has better resolution. Consequently, it is of interest to separately compare the k_e-ratio of this study with the study conducted in Lithuanian sandy soils, since both studies use DPSH. The Lithuanian study gave a k_e of 1,3-1,5 (Gadeikis et al., 2010), which is lower than other studies in sand, where DPL was used. This might indicate a lower ratio when using heavier DP methods. However, the k_e evaluated in this thesis, is higher than the parameter suggested for clay and clay till in other studies, which indicates the contrary. Furthermore, the increasing k_e-ratio is troublesome and could indicate an erroneous set of data.

7.2.4 The ratio k'_e

In other studies, the ratio k'_e, is indicating the relation between the tip pressure, q_c, from CPT and the number of blows, N_{DPSH}, registered by DPSH-A. In Figure 7.8 below, those two parameters are plotted as the ratio k'_e, separately for the two boreholes as well as an average and a chosen value. The graph exhibits many resemblances with the graph displaying the ratio k_e and the previous argumentation can be said to be valid for both ratios. Nevertheless, it is worth mentioning that the resemblance is not surprising since the dynamic tip pressure, q_d, is derived from blows per penetrated distance, N_{DPSH}, merely resulting in a change in scale when comparing the two to the same parameter q_c.

A k'_e of 0,001-0,005 as observed in this thesis seems rather small when comparing the results to previous studies. For example, Martins and Miranda (2003) suggest a k'_e of 0,1 in coarse soils, while Lingwanda et al. (2015) propose a k'_e of 0,46 in sandy soils. However, those studies were conducted in coarse soils, using DPL, something that possibly could explain the much lower ratio derived in this thesis. The heavy equipment of DPSH-A combined with the fine grained clay, result in a very low number of blows per penetration unit and hence a small k'_e. However, further studies in clay are needed in order to draw any conclusions.

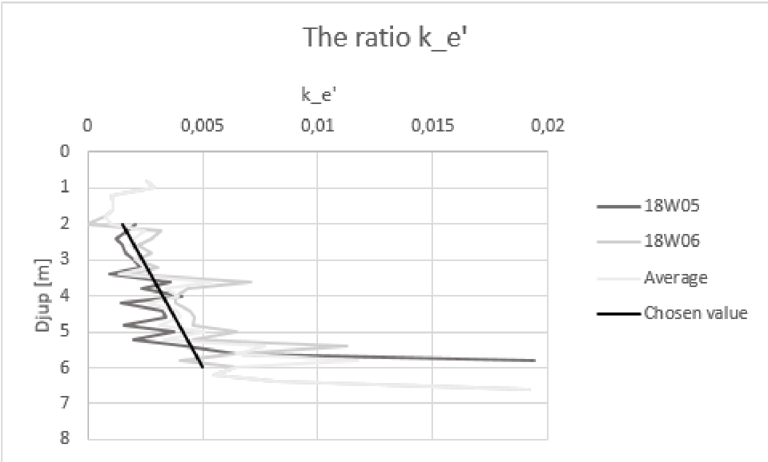


Figure 7.8: The ratio $k_e' = N_{DPSH}/q_c$ from boreholes 18W04 and 18W06.

7.3 DPSH - Field vane

Field vane and DPSH-A were not conducted in the same boreholes, albeit the clay can be interpreted as being part of the same geological formation and therefore have more or less the same characteristics within the investigated area.

7.3.1 Undrained shear strength

Illustrated below are the undrained shear strength evaluated from DPSH-A, according to the relation suggested by Butcher et al. (1996), and the undrained shear strength evaluated from field vane.

The undrained shear strength evaluated from field vane exhibits a largely scattered result and it is difficult to observe any trend. When observing the results in Figure 7.9, the data sets show little to no resemblances.

Even when looking at specific numbers from comparable depths, as in Table 7.1, it is difficult, if not impossible, to find any correlation. However, it can be said that in the upper layer of the clay, field vane seems to indicate

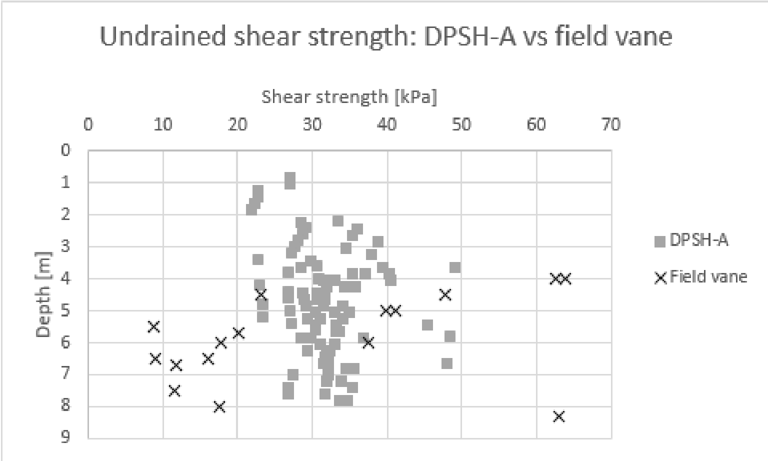


Figure 7.9: Undrained shear strength evaluated from DPSH-A and field vane.

Table 7.1: Evaluated undrained shear strength from the two boreholes where the test was conducted on the depths 5 and 6 meters.

Undrained shear strength [kPa]								
	Field vane				DPSH-A			
	18W01	18W02	18W07	18W08	18W03	18W04	18W05	18W06
4	63,8		62,7		40,4	31,3	30,9	32,9
4,5		47,7		23,2	28,6	30,5	26,7	31,7
5	39,9		41,2		32,8	30,3	27	34,7
5,5				8,9	33,2	30,2	27,2	45,3
5,7		20,1			33,1	33,6	30,5	33,2
6	17,7		37,6		33	32,9		30,9
6,5	16	9			32,1	31,6		31,6
6,7		11,8			32,1	31,3		47,8
7,5		11,5			35,3	26,6		
8	17,6							
8,3	63,1							

a higher undrained shear strength than DPSH-A, whilst in the lower layer, the relation is reversed. Once again, this might be related to the weight of the DPSH-A equipment, as it can be observed in the figures above that the DPSH-A results are more or less independent of depth while the field vane results follow the general trend with decreasing undrained shear strength with depth.

7.3.2 The Butcher et al. relation

Figure 7.10 relates the dynamic tip pressure, q_d, from DPSH-A with the evaluated undrained shear strength from field vane and compares this to the relation proposed by Butcher et al. (1996). In the analysis, one data point that was considered an outlier was removed. Due to a small set of data, it is very difficult to draw any extensive conclusions on the results, however, the relation by Butcher et al. (1996) possibly gives an indication of an average value, albeit neglecting the scatter and variation in the data set.

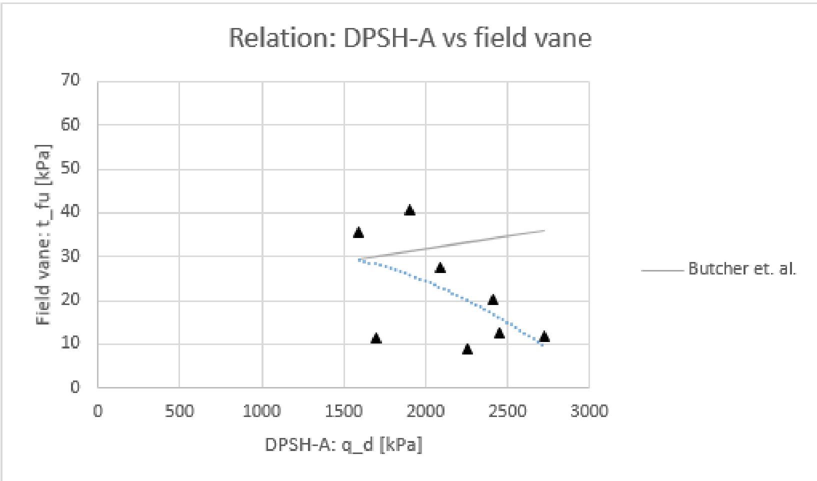


Figure 7.10: Comparing the relation suggested by Butcher et al. (1996) with the results from DPSH-A and field vane for all boreholes.

Chapter 8

Conclusions

At large, this thesis concludes that the relation regarding evaluation of undrained shear strength from dynamic probing, suggested by Butcher et al. in a study from 1996, might be valid also in the soft clays in Scania, but due to results with large scatter it is impossible to neither validate nor reject it.

The data set obtained from the triaxial testing is very small and no certain conclusions can be drawn. In order to further validate or reject the relation suggested by Butcher et al. (1996), more research is necessary. Comparing the evaluated undrained shear strength from CPT with the calculated dynamic tip pressure, q_d, from DPSH-A, the relation by Butcher et al. (1996) does not exhibit sufficient accuracy and the scatter, which seems to increase with higher number of blows per penetrated unit (i.e. q_d), is significant. The field vane results were, in themselves, noticeably scattered and the data set obtained was too small to support neither a validation nor a rejection of the relation by Butcher et al. (1996).

The results from the CPT were also evaluated with respect to the ratios k_e and k'_e, found in literature. Regarding the k_e ratio, difficulties to interpret the values were found, since the result exhibited little analogy with previous studies. The ratio k'_e, was, in this thesis, evaluated as remarkably low compared to other studies, possibly explained by the fact that comparable studies were conducted in coarse soils and with DPL. In conclusion, the CPT ratios analysed could not be completely explained, with the implementation of such ratios not yet recommended in geotechnical projects.

Another interesting annotation can be done regarding the variation of undrained shear strength with depth. Triaxial tests, CPT and field vane all exhibit a decreasing trend with depth for the undrained shear strength in the studied layer of clay. This is something that is absent in the results from DPSH-A, where the evaluated undrained shear strength is more or less independent of depth. As has been discussed in the previous chapter, this could be an error depending on the soft characteristica of the studied clay combined with the heavy weight of the equipment, together impeding the DPSH-A from conducting a realistic number of blows per penetration unit. As of the obtained results, the variation in the soil was probably ignored, which could explain the registered number of blows, N_{20}, being very similar independently of depth and consequently, resulting in an evaluated undrained shear strength with no variation with depth. This shows the importance of assuring that the right kind of equipment is used.

Overall, more research is needed to evaluate the suggested relation for DPSH-A in order to assure the accuracy and in order to be implemented in geotechnical projects. The relation proposed by Butcher et. al was, in this thesis, not validated nor rejected for use in the soft clays in Scania, however, this study is too small to draw any extensive conclusions.

8.1 Research questions

Below, the research questions, initially stated in Chapter 1, will be answered.

1. Can correlations between parameters from DPSH-A and the shear strength of soils, suggested by Butcher et al. (1996), be used in the soft clays in Scania?

2. If not, can any other statistically validated correlations be proposed?

The analysed relation was not rejected as inaccurate to use in soft clays from Scania, neither was validated. The collected data set is small and largely scattered, and in order for the suggested relation to be implemented in geotechnical projects in Sweden, further research is necessary.

Furthermore, it is difficult, if not impossible, to propose new relations with a limited data set, such as is available in this thesis, and with such scattered data. In order to suggest more accurate relations regarding undrained shear strength evaluated by DPSH-A in Swedish clays, further research, where more data and more parameters are considered, is necessary.

8.2 Sources of error

The number of triaxial tests that has been possible to conduct within the time span of this thesis is limited and it is therefore difficult to obtain reliable results. This thesis does not aspire to give a complete answer to the question whether DPSH-A can give reliable evaluations of undrained shear strength when used in clay, but rather serve as a pilot study. Two triaxial tests have been conducted, from one borehole and on the depths 5 and 6 meters. In order to give more reliable results and recommendations, far more tests would have to be conducted, with a greater variation in depth, location, and type of clay than what has been possible in this thesis.

An error, that could have had a large impact on the results and on the possibility to analyse them is the fact that the DPSH-A was operating, to a large extent, outside of the recommended interval, i.e., the required minimum number of blows were not achieved, probably due to the clay being very soft. This ignored the variation in the soil to a great extent and may have produced secondary errors in the evaluation of undrained shear strength.

Recommendations say that advanced laboratory tests, such as triaxial tests, should be done, preferably, within a week from the sampling, but no longer than a month afterwards. The triaxial tests conducted in this thesis have, due to logistic issues, been conducted five to six weeks after the samples were collected. This might have influenced the quality of the samples and, consequently, possibly also the results. Nevertheless, the samples have been stored in a confined box in a temperature controlled room from the time of sampling until the time of testing, and the specimens have therefore probably preserved their main characteristics.

A lot of the input parameters used in assumptions, calculations and analyses have been generalised and chosen in order to make it possible to further evaluate the research questions. These generalisations, such as the density and the over consolidation ratio of the clay have probably impacted the results and the analysis of them.

Another possible source of error is the human factor. Both when operating the machines on the site of investigation as well as in transportation and handling of the samples. Furthermore, triaxial testing require skilled lab technicians in order to get fully reliable results. The triaxial tests in this thesis have been preformed by the author herself, who holds little to no previous knowledge of the equipment.

Chapter 9

Further research

Further research on this matter is necessary in order to confirm any conclusions in this thesis. The scope and time span of this work have been limited and hence, sampling, testing and analysing a larger number of samples is recommended.

It is also of interest to take into account more parameters than what have been considered in this thesis. For instance, Butcher et al. (1996) proposed that considering the sensitivity of the clay produces more reliable results with less deviation. It would also be valuable to consider other parameters, such as liquid limit, water content or the content of fine grains.

Moreover, different types of clay should be investigated. This thesis has only considered soft clays, since that was the type of clay available at the site of investigation. The relation for stiff clay, also suggested by Butcher et al. (1996), is still to be evaluated. Furthermore, clays from different locations should be studied.

One problem with conducting DPSH-A in the clay in this thesis was the fact that the recommended minimum blows per penetrated unit were, for the most part, not achieved. This adds uncertainty to the results. Further research could either focus on finding a soft clay where the number of blows are still meeting the recommendations, or evaluating whether a lighter DP method is more appropriate for clays as soft as the clay analysed in this work.

The use of DPSH-A is of interest mainly when CPT can not be conducted due to excessive resistance in the soil e.g. clay till. Therefore, it is crucial to perform similar research on clay till.

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