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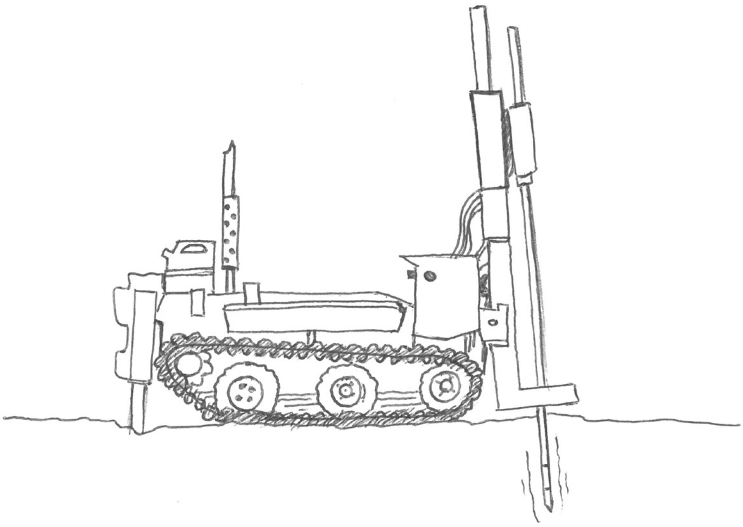

CONE PENETRATION TESTING IN THE CLAY TILLS OF SKÅNE

An investigation of the cone factor

JOHAN LINDSTRÖM

Geotechnical Engineering

Master's Dissertation

DEPARTMENT OF CONSTRUCTION SCIENCES
GEOTECHNICAL ENGINEERING

ISRN LUTVDG/TVGT--17/5061--SE (1-72) | ISSN 0349-4977
MASTER'S DISSERTATION

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Printed by Media-Tryck LU, Lund, Sweden, July 2017 *(Pl).*

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Abstract

The undrained shear strength is a key parameter in the design process of geotechnical structures. One of the most frequently used methods to investigate this parameter is the cone penetration test (CPT). The CPT does not measure the undrained shear strength directly, but instead the resistance that a conic probe encounters as it is pushed into the ground. A calibration factor, the cone factor (N_{kt}), relating the two quantities is, therefore, needed. The Swedish Geotechnical Institute suggests to use a cone factor of 11 when using CPT in the clay tills of Skåne. However, in Denmark, a value of 10 is recommended for similar soils. The magnitude of the calibration factor is critical when determining the undrained shear strength and, consequently, for further design.

To investigate the cone factor, measurements of the undrained shear strength and the net cone resistance were compared for five types of clay till found in Skåne at a total of three different sites. Depending on availability, other soil parameters were considered in the analysis in order to explain variations of the calibration factor N_{kt}.

The comparison between the undrained shear strength measurements, from field vane tests, and the net cone resistance gave a mean value of N_{kt} in the range of 9.3 to 10.2. However, a significant standard deviation, ranging from 0.7 to 1.7, is connected to the evaluated cone factors.

Due to the significant degree of scatter in the data, partially caused by the inhomogeneous nature of the soil and partially influenced by primarily using field vane tests to evaluate the undrained shear strength, it is hard to draw exhaustive conclusions. However, for the different soils studied in this project, it seems reasonable to use a cone factor lower than 11. It also appears that the cone factor is closely linked to the overconsolidation ratio, though more tests at various locations are needed to verify this hypothesis.

Preface

This project concludes five years of inspiring and developing studies at the faculty of engineering, LTH, in Lund.

The project was conducted in association with the Division of Geotechinical Engineering at LTH and the geotechnical group at Sweco Civil AB in Malmö. First and foremost, I am very grateful to have had the opportunity to gain a broad insight into the field of geotechnics. It would not have been possible without this project, and also my supervisors Erika Tudisco, LTH, and Håkan Lindgren, Sweco Civil AB, whom I would like to thank for valuable insights, feedback and guidance. Furthermore, I would like to thank the other personnel at the geotechnical group at Sweco in Malmö for providing helpful thoughts and ideas, as well as a general interest.

Finally, I would like to thank Ann Dueck, Rolf Larsson and Håkan Garin for conducting previous investigations of the clay tills of Skåne, thus making available a considerable amount of data regarding the materials properties. This project would not have been possible without it.

Lund, june 2017

Johan Lindström

Notations and Symbols

m - bulk mass
V - bulk volume
w - natural water content
m_w - water mass
m_s - solid mass
ρ - bulk density
ρ_s - solid density
S - degree of saturation
n - porosity
e_0 - void ratio
l_c - clay content
w_L - liquid limit
w_P - plastic limit
σ_{v0} - total overburden stress
u - pore pressure
σ'_{v0} - effective overburden stress
σ'_c - preconsolidation stress
OCR - overconsolidation ratio
σ'_h - horizontal stress
K_0 - coefficient of earth pressure at rest
K_{0nc} - coefficient of earth pressure at rest for a normally consolidated soil
c_u - undrained shear strength
q_t - total cone resistance
f_t - total sleeve friction
R_f - friction ratio
N_{kt} - cone factor
N_{ct} - cone factor, preconsolidation stress
m - mean value, CPT-smoothing
s - standard deviation, CPT-smoothing
f_l - constant, CPT-smoothing
q_s - smoothed cone resistance
c_v - undrained vane strength
P - applied force, field vane test
g - gravity acceleration

a - lever, field vane test
$M,\ M_1,\ M_2,\ M_3$ - static moment, field vane test
$h,\ d,\ r,\ t$ - vane geometry
c_k - undrained cone strength
K - constant based on the apex angle of the cone
m_{cone} - cone mass
i - cone penetration
$\sigma_a,\ \sigma_r$ - axial and radial stress, triaxial test
$\varepsilon_a,\ \varepsilon_{vol}$ alt. $\varepsilon_1,\ \varepsilon_V$ - axial strain and change in volume, triaxial test
$\sigma_1,\ \sigma_3$ - principal stresses
p - mean stress
q - deviator stress
$\alpha,\ \Lambda$ - constants, Ladd- and Foott's formula
ϕ' - effective friction angle

Contents

1 Introduction

1.1 Background

Determining the strength parameters of a soil is not an easy task, and the procedure is even more difficult if the soil is non homogeneous and highly overconsolidated. The main problem is to obtain values that are representative for large areas based on a series of point investigations. However, in civil engineering it is of great importance to be able to describe the properties of the ground in order to enable calculations of load bearing capacity and settlements.

As a result of this, many different test methods have been developed over the years to measure important soil parameters. These methods vary a considerable amount in complexity, cost and utility. However it is possible to divide the different methods into two categories; in-situ tests and laboratory tests. In modern, commercial, geotechnical investigations there is a strong desire to derive the soil properties from in-situ methods as these tests are conducted on soils in their natural state. Furthermore, studies with advanced laboratory methods also tend to be costly and very sensitive to the quality on gathered soil samples, and also the ability to recreate the in-situ stress condition.

The Cone Penetration Test (CPT) is one of the geotechnical in-situ methods most frequently used in investigations of the clay till that is characteristic for both Skåne and Denmark. A lot of the early experience in the field was established in conjunction with the investigations conducted for the construction of the Storebæltsbroen in Denmark [18]. In the investigations it was found that the CPT-probe normally penetrates clay till fairly well. However, it was also noted that CPT occasionally failed to penetrate firm variants of clay till and clayey till. Problems were also detected in clay tills containing firm layers of friction soil and/or stones and boulders [18]. In Skåne this relates to the Baltic clay tills being relatively easy to investigate due to their high content of fine soil material and relatively small amounts of coarser soil particles. On the other hand, the Northeast clay till is less suitable for CPT with a Swedish standard setup as it is firmer and in general contains more coarse soil particles relative to its clay- silt- and fine sand content [16][23].

As the undrained shear strength is not directly measured with CPT, a relation is needed to translate the cone resistance in the sounding to the desired strength parameter. The relation itself is simple, and is based on dividing the cone resistance by a cone factor, N_{kt} in order to receive the undrained shear strength [42]:

$$c_u = \frac{(q_t - \sigma_{v0})}{N_{kt}} \qquad (kPa) \tag{1.1}$$

The cone factor is to be seen as a calibration parameter that describes how the cone resistance measured with CPT-soundings corresponds to the undrained shear strength measured with different geotechnical methods. The in-situ method that is commonly used to acquire a referential value of the strength is the field vane test. If it is possible to obtain undisturbed samples of the material, advanced lab tests, like the triaxial compression test can be conducted.

The value of the cone factor is traditionally related to the liquid limit and overconsolidation ratio of the soil material when the CPT is used in clay soils in Sweden [21]. However, a cone factor $N_{kt} = 11$ is suggested by the Swedish Geotechnical Institue (SGI) [23], though it is possible to use different values if better knowledge of the geotechnical properties at an arbitrary site exists. Previous studies in Denmark, that have been conducted on similar clay till, also show that a cone factor, of about 10 is a possibility [3][26][27][28].

Furthermore, different deposits of clay till present varying degree of preconsolidation. The clay content, void ratio and grain size distribution, among other soil properties, also differ between sites. This variation might complicate the idea of having just one constant value of the cone factor for all of Skåne.

1.2 Purpose and Delimitations

The main purpose of this project has been to investigate values of the cone factor, N_{kt}, that might be relevant for the clay tills of Skåne. The aim has also been to study variations between different field sites, and also different layers of clay till and analyze what the causes for possible variation could be. Furthermore, by obtaining a deeper understanding of the material behaviour and the parameters affecting it, enable improvements when evaluating the soils' bearing capacity. In turn, this will lead to an improved foundation design and a better utilization of economic resources.

The focus in the project is on the cone penetration test, and finding suggestions regarding the value of the cone factor. The complimentary geotechnical methods, used to obtain a variety of soil parameters that are relevant for the study, have not been investigated further than what has been deemed necessary.

Furthermore, as the quaternary deposits are varying throughout Skåne the results obtained in this project can only be considered valid in the close surroundings of the conducted tests.

Lastly, no effort have gone into the possibility of evaluating drained parameters and/or the stress-strain moduli from CPT.

1.3 Method

The approach chosen was to calibrate the CPT against field vane- and triaxial tests as this matches the traditional investigation methodology well. Furthermore, it generates data that can be compared to previous investigations. A piston sampler was used to obtain undisturbed samples for tests with a triaxial apparatus, and a non-continuous flight auger was used in conjunction with the field vane tests to obtain disturbed samples for routine tests.

The possibility to establish another site than Tornhill was investigated based upon the extents of various glacial advancements described by the quaternary maps of the southwestern part of Skåne. The stratigraphy at potential locations was determined based on previous drillings in the vicinity of the location.

To enable the project, a thorough literature study on the clay till of Skåne and its properties was conducted. The study handled the different geotechnical investigation methods that have been used previously to investigate clay till and clayey till in Sweden and Denmark. The study serves as a basis for the investigations conducted in the project but also as mean of gathering necessary data for evaluation.

2 Geology

2.1 Regional Geology

The geology of the southwestern part of Skåne features an abundance of very stiff soils, generally superimposing sedimentary bedrock [8]. The various soil types in the area are rich in clay compared to the vast majority of soil in Sweden. The generous clay content combined with the overall high stiffness of the soil makes the quaternary deposits in Skåne appear more similar to the soils frequently encountered in Denmark than in the rest of Sweden. The explanation behind this is primarily based on the type of bedrock in Skåne and in its vicinity, and also the influences of the glacial periods in the northern part of Europe.

2.1.1 Bedrock

The border between the southwestern part och Skåne and the rest of Sweden is defined by the Sorgefrei-Tornquist zone. This zone is a crustal boundary between the hard crystalline bedrock of the Baltic shield to the northeast and the Phanerozoic bedrock to the southwest [25].

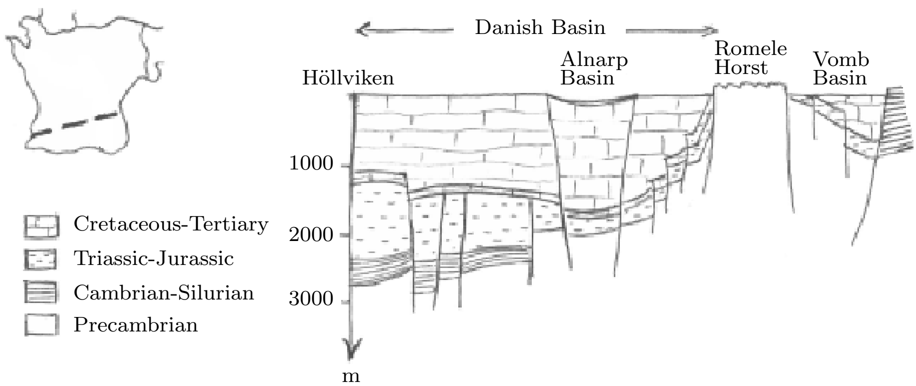


Figure 2.1: Rock formations in the southwest of Skåne. Inspired by [8]

The Phanerozoic formations located to the west of the Romele horst are more than two thousand meters thick, superimposing the crystalline rock formations otherwise found at the rock surface to the northeast of the Sorgenfrei-Tornquist zone. The Phanerozoic rock formations found in Skåne are also encountered in the entirety of Denmark. The formations dating back to the Phanerozoic period essentially consists of multiple layers of claystone, limestone and sandstone. Further down in the Jurassic and Triassic layers coal is also encountered [8]. A simplified version of the rock formations in the southwest of Skåne is shown in Figure 2.1.

2.1.2 Quaternary Deposits

The quaternary deposits in the southwest of Skåne consist, to a great extent, of glacial soil types. These deposits, in turn, commonly consist of different variants of clay till and clayey till along with various glaciofluvial soils [8]. The clay till is an unsorted material that was deposited by the glacial ice, either at the bottom of the ice as lodgement till or melting out from top of the ice mass as ablation till, seen in Figure 2.2. This deposition primarily happened as the ice slowly melted and therefore retreated backwards [25]. However, deposition of lodgement till also occurred at times when the glacial ice could not overcome the friction against the ground underneath or when it was advancing uphill. As the lower levels of the glacial ice contained most of the soil material the relation between lodgement- and ablation till heavily favours lodgement till [25].

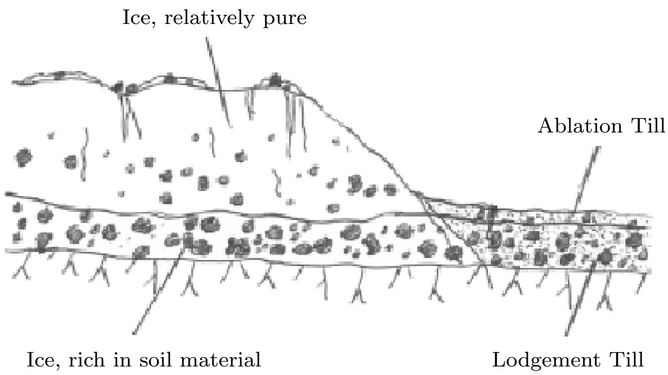


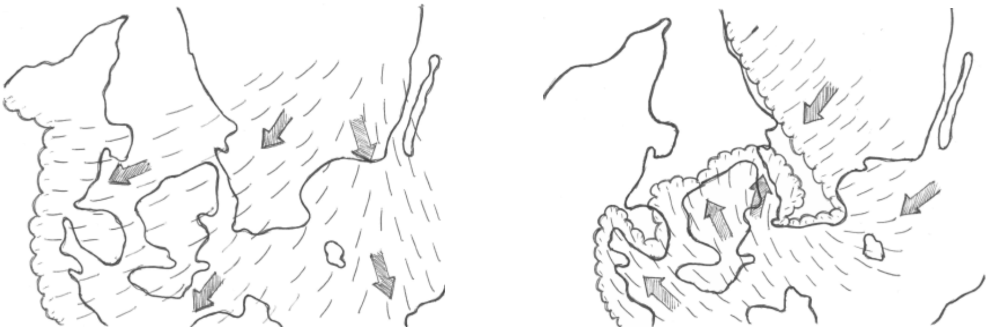
Figure 2.2: Depostiton of till

The unsorted nature is lent to the material by this formation process. Soil particles varying greatly in shape and size where deposited simultaneously [18]. The soil material that later came to be deposited as till was gathered by the glacial ice as its front crept outwards during the expansion phases, eroding older soil deposits and bare rock surfaces [8]. The material was then transported within the ice by slow moving currents. During this process of transportation the soil particles were ground against one another resulting in further erosion [18]. A direct result of this action is a very well graded soil, as some coarser particles managed to pass through the glacial movement intact while other got reduced into finer fractions [18]. One way to describe the soil is as matrix-supported, which corresponds to bigger aggregates; gravels, stones and boulders, that ”float” in a matrix of fine material [25].

Onwards, the deglaciation of Skåne that occurred during the final stages of the Weichel glacial period was complex. It was comprised of both relatively warm periods, when the melting ice crept backwards, and cold periods when the ice front advanced yet again [25]. During the warm periods till was deposited and ice lakes were formed where water from the melting glacier could not drain away. In these lakes fine soil particles had the possibility to settle down [25]. Then, when the ice advanced again, parts of the previously deposited till and fine sediments were yet again incorporated in the ice and served as material for the next layer of till to be deposited. This process has resulted in stratigraphies where interglacial soil layers are embedded within the till. The properties of the till also vary greatly over the stratigraphy [18]. These variations within the till layers are influenced a great deal by what glacial movement deposited them. This in turn reflects in which direction the glacial front advanced and also from where it gathered the material that later comes to make up the till. Essentially, mapping the material composition of the till gives a fairly good indication of the bedrock in the area, albeit slightly offset in the direction the ice front advanced in [18].

Denotation of the different till deposits are based on these phenomena, resulting in two distinguished types of clay till and clayey till. The first of these being the Northeast clay till, which was deposited by the glacial front that advanced from the northeast, as the name suggests. To the Northeast, the glacier encountered hard crystalline bedrock and old soil deposits, rich in coarse soil particles. This resulted in a clay till with a relatively low clay content and thereby higher content of coarser grains [18]. The Northeast clay till does not always meet the required clay content to be classified as clay till and is then classified as clayey till [48]. In contrary to the Northeast till, the

Baltic clay tills are often rich in clay, whereas the coarser fractions appear to a lesser extent [18]. This is mainly due to the fact that the glacier incorporated soil material from soft sedimentary bedrock and the sedimentary soil deposits that had built up in the Baltic ice lake. As parts of the sedimentary rock formations were made up of limestone, the Baltic clay tills are often rich in lime [8].



(a) The Northeast advancement (b) The Baltic advancement

Figure 2.3: Notorious periods of the Weichsel glaciation. Inspired by [41]

The extents of the Northeast clay till covers about the whole area of south-western Skåne in accordance to the boundary of the ice in Figure 2.3a. The Baltic clay till on the other hand, is located in the south-southwestern part of the region, its extents to the east limited by the Romele horst and its extents in the north reaching just past Svalöv, Figure 2.3b [47][48]. Furthermore, whether the clay till was deposited as ablation- or lodgement till greatly influences its geotechnical properties. The soil that was formed at the bottom of the ice experienced a tremendous pressure from the remaining superimposed ice mass. This resulted in the consolidation of the soil, where pore water was slowly squeezed out of the material while the soil particles were packed together tightly. The clay till was left with a higher density and stiffness [20]. Even though most of the soil material was located in the lower levels of the glacial ice, a considerable portion was deposited from the upper part of the glacier as it melted. This clay till was not originally consolidated to the same extent, and thus it is softer. However, later advancements of the ice front have, in some cases, covered previously superficially deposited till and effectively changed its properties to match those of clay till or clayey till that has been deposited on the the bottom of the ice mass [18].

2.2 Field Site Geology

The various field sites presented, and utilized, within this report display the large variations of the clay till deposits in Skåne. The sites presented includes Tornhill, just to the north of Lund, Lergöken, in the southern part of Malmö and Torreberga located in the vicinty of the Alnarp basin.

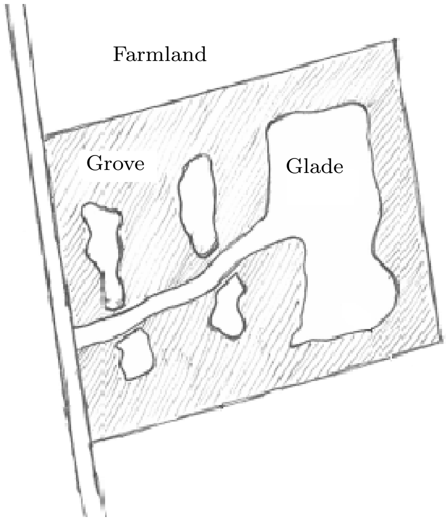
2.2.1 Site I - Lund - Tornhill

The field site Tornhill is located just to the north of Lund, within the bounds of the estate Vallkärratorn 1:18. This is an already established site for geotechnical testing of clay till, and has been used since 1995 for this purpose. It is featured in a couple of studies, for instance by Dueck [3][4] and Larsson [19]. Basic soil parameters that previously have been evaluated at the site are presented in Table 2.1.

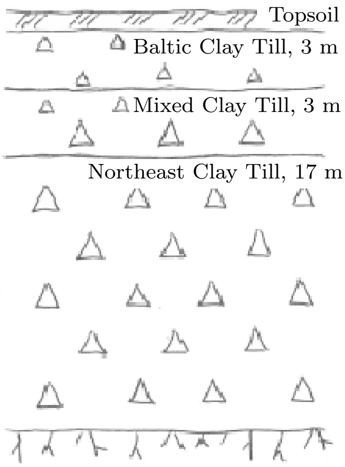
Table 2.1: Values of different soil properties for the test site Tornhill [3]

Property	Baltic clay till 0-3 m	Mixed clay till 3-6 m	Northeast clay till 6-10 m
Bulk density, t/m^3	2.11	2.17	2.27
Particle density, t/m^3	2.72	2.72	2.72
Natural water content , %	17.0	15.8	12.4
Liquid limit, %	32.0	26.9	23.4
Plastic limit, %	16.3	16.3	13.3
Index of plasticity, %	11.4	12.2	9.7
Degree of saturation, %	81.0	84.0	82.5
Clay content, %	35.0	25.1	21.1
Cone resistance, MPa	4	2	7
Vane strength, kPa	350	290	450

The estate Vallkärratorn 1:18 is occupied in its entirety by a square grove that is about 86 $\times$ 105 m^2. Harneskgränden passes by to the west of the grove, while the remaining three sides are surrounded by farmland. A large glade, where the previous tests have taken place, is located in the eastern part of the grove. The ground is relatively flat within the area and it is located about 62 meters above sea level [3]. A map of the site and the evaluated soil profile is shown in Figure 2.4.



(a) Map



(b) Stratigraphy

Figure 2.4: Site information Tornhill

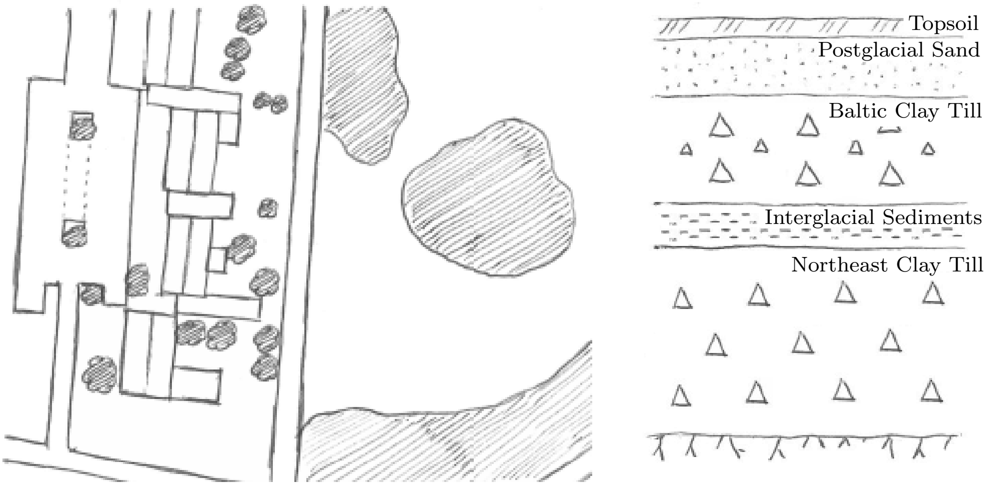
As the site has been throughoutly investigated, the stratigraphy is well known. The top layer, just beneath a small amount of topsoil, consists of Baltic clay till. This layer is defined as 3 meters thick with little variance within the bounds of the site. The Baltic clay till at the site is denoted as fine clay till, as the clay content exceeds the 25 % limit by far. Furthermore, it has a relatively low content of coarse soil particles [3][47].

Below the Baltic clay till is another layer of clay till that is denoted as mixed. This layer contains elements from both the Baltic- and the Northeast clay tills. The clay content is on average above 25 %, though it decreases with depth [3]. The Norhteast-material that has been blended into the soil also induce a higher content of coarse soil than in the superimposing layer [18].

The bottom layer consists of Northeast clay till and has a thickness of about 17 meters [3]. Based on investigation of the top 4 meters of the layer, it has been classified as coarse clay till. The clay content, the determining factor, is evaluated to about 20 %. This layer is rich in coarse particles compared to the soil above it [18][41]. Furthermore, this layer is considerably firmer than the other two. The Northeast clay till is resting on sedimentary rock that is made up of clay shale [18].

2.2.2 Site II - Malmö - Lergöken

The field site Lergöken is located within the southern border of Malmö, just to the east of the estates Lergöken 5 and 6 in Almvik. The area at the site consists of a grass field that slopes slightly to the south, together with a couple of small groves. The elevation of the site is about 33 meters above sea level. A map of the site is presented in Figure 2.5a.



(a) Map (b) Stratigraphy

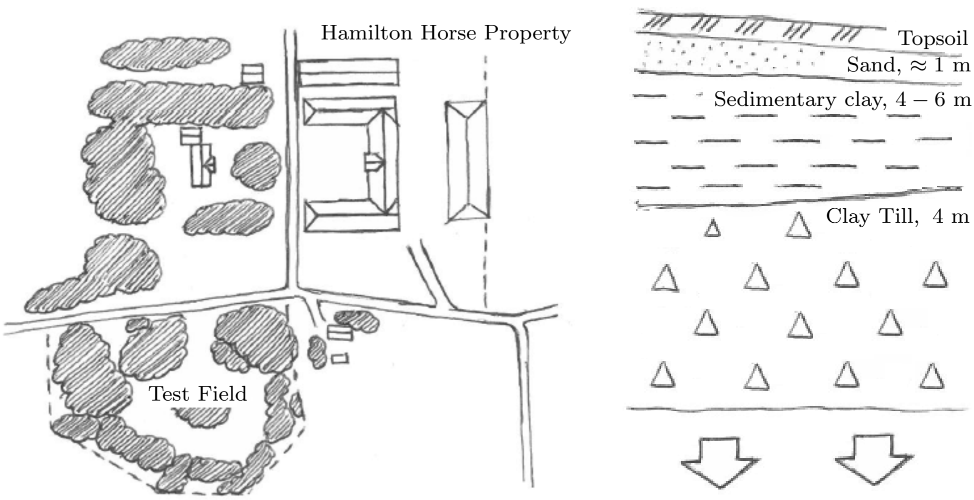
Figure 2.5: Site information, Lergöken

The soil deposits in the vicinity of the site consists mainly of clay till, however, a topmost layer of sand, and also interglacial layers, are common [39][40]. The thickness of these deposits vary greatly, in a relatively small area the depth to the underlying rock formation ranges from 5 to 25 meters [46]. From the information available a plausible stratigraphy is presented in figure 2.5b. Essentially a thin layer of topsoil superimposes a layer of postglacial sand, which in turn rests on the upper bed of clay till. Between the upper and lower beds of clay till there is a significant chance to encounter interglacial sediments. The lower bed of clay till is assumed to rest directly on the Danian limestone formation. At the investigation point the limestone should be encountered at a depth of about 20 m below the ground surface [46]. It is reasonable to assume that the quaternary deposit at the site features layers of both Northeast- and Baltic clay till as both ice advancements have passed over the area. The clay till found in the area to the south of Malmö is to a great extent coarse and the clay content is in general below 30 % [39].

2.2.3 Site III - Staffanstorp - Torreberga

The field site Torreberga is located to the southeast of Staffanstorp, directly east of Grevie. This site lies within the bounds of the Alnarp basin, presenting vast soil deposits [46]. The site has previously been used by Dueck and Garin [6] for testing the usability of a pressuremeter in firm clay and also clay till. The pressuremeter tests were complimented by field vane tests, CPT-soundings, and lab tests conducted on acquired piston samples. No further tests were conducted at the site during this project.

The mean elevation of the site is about 15 meters, however the elevation in the vicinity varies between 10 and 20 meters. The tests were conducted to the south of the gravel road, leading to the west, from the Hamilton Horse property. A map of the site is presented in Figure 2.6a.



(a) Map (b) Stratigraphy

Figure 2.6: Site information, Torreberga

The upper part of the soil deposit, resting on top of the vast Alnarp sediments consists of clay, fine friction soil and different clay tills, in accordance to Figure 2.6b [6].

3 Test Methods

3.1 Field Tests

3.1.1 Cone Penetration Test

The basic execution of the cone penetration test is that a probe with a conical tip, attached to a series of drill rods, is pushed into the ground at a constant rate. The exterior of the conical probe is completed by a pore pressure filter and a friction sleeve according to Figure 3.1. While the probe is forced into the ground continuous measurements of the cone resistance, sleeve friction and pore pressure buildup are made by the transducers connected to the different parts [42].

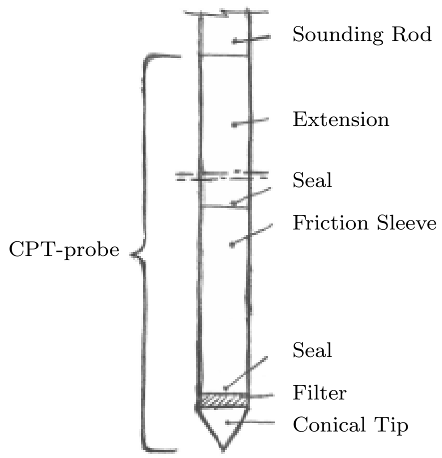


Figure 3.1: Geometrical design of a CPT-probe. Inspired by [21]

The bottom part of the Swedish standard probe consists of a cone with a 60°apex angle and a base diameter of 35.7 mm , which results in a cross sectional area of 1000 mm^2 [10]. The filter, connected to the transducer used for pore pressure measurement, is normally placed just behind the cone, although there are other possible placements in various locations on the probe [10][42]. Following the cone and the filter is a metal sleeve that is used to measure the friction the probe experiences on its surrounding surface area as it is forced through the soil [10]. Moreover, there are a couple of different sizes of probes. The most commonly used are those with a cross sectional area of either 1000 mm^2 or 1500 mm^2. However, there are also finer probes with a cross sectional area of 200 mm^2 that are used to obtain better investigation resolution when probing soft clays. Lastly there are more robust probes with a cross sectional area in the range of 4000 mm^2, which are effectively used for soundings in coarser frictions soils, and very stiff soils, essentially soils in which a standard probe can not penetrate sufficiently [42]. This robust probe gives comparatively poor resolution when used in softer soil [42].

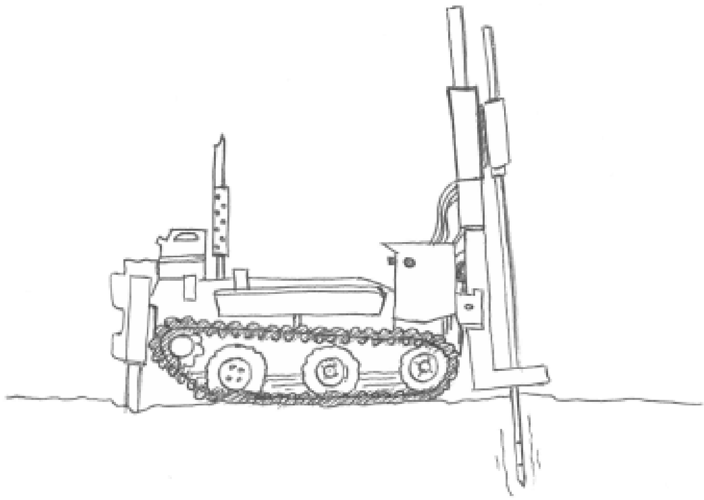


Figure 3.2: Drill-rig mounted 5 ton CPT unit. Inspired by [21]

The CPT-probe is driven by a machine consisting of an extendable rod system, a thrust mechanism and a reaction frame. The capacity of the thrust mechanism used naturally defines the dimensions of the reaction frame. For example a CPT unit with a thrust capacity of 25 ton normally requires to be mounted on a large truck whereas the 5 ton setup, frequently used in Sweden, can be mounted on a standard drill-rig as seen in Figure 3.2 [21][42].

The execution of CPT-soundings in Sweden is carried out according to European Standard, SS EN ISO 22476-1:2012. The standardization involves acceptable equipment deviations and the test procedure itself [21]. Previous to the implementation of the European standard, soundings were done according a recommended standard developed by the Swedish Geotechnical Society (SGF) [10]. It is worth noting that the two standards differ very little.

The parameter that is mainly calculated based on the CPT-data is the undrained shear strength, however as previously mentioned in Section 1.1 this is done with the help of a calibration factor N_{kt}. The essential relation is, for example, defined by P. K. Robertsson [42] as:

$$c_u = \frac{(q_t - \sigma_{v0})}{N_{kt}} \qquad (kPa) \tag{3.1}$$

The raw CPT-data consists of the following measured parameters; the cone resistance, q_t, the sleeve friction, f_t, and registered pore pressure buildup, u. [10]. The raw data mainly provides information about the stratigraphy of a soil profile. For instance, variations in cone resistance suggests the existance of an interface between layers of stiffer/softer soil, whereas a sudden drop in pore pressure could indicate contact with a more permeable layer [21]. The soil classification of encountered layers can also be carried out using established charts that describe the soil behaviour type based on the cone resistance, the friction ratio and the pore pressure buildup [42].

However, in order to enable further evaluation of the CPT, data processing is needed. The cone factor used to evaluate the undrained shear strength is one example of the processing. It is obvious that the decided value of N_{kt} has great influence on the evaluated undrained shear strength and, in turn, the estimated bearing capacity. The factor N_{kt} can be evaluated based on other in-situ and/or laboratory test methods that measure the undrained shear strength directly. Various methods have been used in different studies. However, the majority of these studies utilizes field vane tests and/or triaxial tests [19][26][27][28]. Furthermore, it is also possible to use a combination of legitimate test methods for determining the cone factor. Based on Swedish experience for investigation in clay, a relation based on the liquid limit has been established. For overconsolidated soils a further correction factor was added to the relation, resulting in [21]:

$$c_u = \frac{q_t - \sigma_{v0}}{13.4 + 6.65\ w_L} \left(\frac{OCR}{1.3}\right)^{-0.2} \qquad (kPa) \tag{3.2}$$

For highly overconsolidated clays this relation has previously presented cone factors in the range of 20-30 [22]. As this relation presents values that are about twice or tree times the magnitude of the cone factors commonly used in clay till, it is not directly useful. However, another expression based on w_L and/or OCR might be. A relation for use in clay till has been developed by SGI [14]:

$$c_u = \frac{q_t - \sigma_{v0}}{11} \qquad (kPa) \tag{3.3}$$

Other values than 11 seem to be reasonable, although the computer program CONRAD, which is frequently used in Sweden for CPT-evaluation, makes use of Equation 3.3 [14]. A similar relation is established in Denmark where the value 10 is used based on the experience initially gained during the construction of the Storebæltsbroen [45]:

$$c_u = \frac{q_t - \sigma_{v0}}{10} \qquad (kPa) \tag{3.4}$$

Further studies that have been made by Luke [26][27], among others, validate Equation 3.4. Luke [27] also studied the dependency of the cone factor based on different soil parameters. However, the conclusion of this study was that it is not possible to isolate the effect of a single parameter as they are all correlated to one another to some degree. The most promising of these relations was the correlation between the cone factor and the friction ratio, R_f evaluated from the CPT [27]:

$$c_u = \frac{q_t - \sigma_{v0}}{15\ {R_f}^{-0.4}} \qquad (kPa) \tag{3.5}$$

To summarize, the following should serve as a good starting point when investigating the cone factor, N_{kt}, in the clay tills of Skåne:

$$N_{kt} = \begin{cases} f\left(w_L,\ OCR\right) \\ \\ 10\ -\ 11 \\ \\ 15\ R_f{}^{-0.4} \end{cases} \tag{3.6}$$

Furthermore, it is also possible to estimate the preconsolidation stress, σ'_c, and the overconsolidation ratio, OCR, from CPT-measurements [21][42]. Different approaches are available, but they all do share similar traits [21]. Furthermore, the evaluation is very sensitive to the quality of the CPT-data [21]. According to SGI the preconsolidation pressure can be evaluated as [21]:

$$\sigma'_c = \frac{q_t - \sigma'_{v0}}{1.21 + 4.4\ w_L} \qquad (kPa) \tag{3.7}$$

For calculations of the preconsolidation stress in clay till it is advised to use the following approximation though.

$$\sigma'_c \approx \frac{q_t - \sigma'_{v0}}{3} \qquad (kPa) \tag{3.8}$$

However, the values calculated with Equations 3.7 and 3.8 can differ significantly. The overconsolidation ratio is calculated as [21]:

$$OCR = \frac{\sigma'_c}{\sigma'_{v0}} \tag{3.9}$$

The received value of the overconsolidation ratio is not to be seen as a replacement for the oedometer test but as a reference value [21]. Similar to the cone factor, N_{kt}, used for determining the relation between undrained shear strength and cone resistance another cone factor, N_{ct}, can be implemented in the determination of the preconsolidation stress. Its possible values are summarized as:

$$N_{ct} \approx \begin{cases} 1.21 + 4.4\ w_L \\ 3 \end{cases} \tag{3.10}$$

Another way to calculate the overconsolidation ratio and preconsolidation stress was presented by Robertson [42]:

$$OCR = 0.25 \left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \right)^{1.25} \tag{3.11}$$

$$\sigma'_c = \sigma'_{v0}\, OCR \quad (kPa) \tag{3.12}$$

It is not previously known how well the empirical relations derived by Robertson fares in the investigation of clay till.

Finally, due to the continuous nature of the CPT-data its possible to reduce the spiky appearance, caused by the presence of coarse particles and lenses of friction soil, by mathematical smoothing [45]. This way of smoothing CPT-curves was introduced by Mortensen et al [28]. The reasoning behind smoothing the data is to enhance the general trend by reducing or eliminating peaks and noise. However, by removing the local peaks information about the presence of coarse particles and lenses of silt/sand/gravel is lost. The general idea behind the mathematical process is to compare the measured value, q_t, at one level with the mean value and standard deviation over a wider range. The smoothing constraint is presented as [28]:

$$m - f_l\, s \; < \; q_s \; < \; m + f_l\, s \tag{3.13}$$

Where m and s represent the mean value and standard deviation over a set depth and f_l is a constant value. When interpreting the undrained shear strength, c_u, in conjunction with field vane tests the value of f_l has previously been chosen as 1. The range over which m and s are evaluated has been set to 0.2 meters of investigation depth [28].

3.1.2 Field Vane Test

The field vane test is an in-situ method that has been used in quite a few scientific studies with the intention to obtain a value of the undrained shear strength of clay till and/or provide reference material for CPT calibration [2][3][26][28]. The method also sees its fair share of commercial use and test itself is simple to conduct. First the vane instrument is pushed down into the soil until the bottom end of it is at a depth corresponding to at least twice the height of the vane. Secondly a moment is applied via a torque wrench and the moment at which the soil reaches failure is measured. Finally the corresponding undrained vane strength, c_v, is evaluated based on an assumption of the failure surface in the soil.

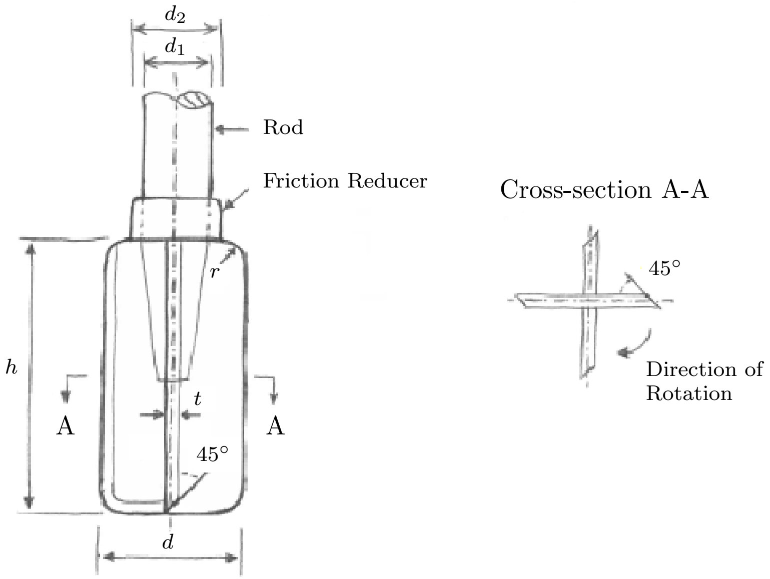


Figure 3.3: Geometry of danish field vanes. Inspired by [7]

Table 3.1: Geometry and load capacity of danish field vanes

Vane No.	h (mm)	d (mm)	r (mm)	t (mm)	d_1 (mm)	d_2 (mm)	P_{max} (kg)	$c_{v,max}$ (kN/m^2)
V4	80	40	10.0	3.0	20	26	50.0	701
V5	100	50	12.5	3.0	20	26	50.0	359
V7.5	150	75	18.8	3.0	20	26	50.0	106
V9.2	350	92	23.0	3.0	20	26	50.0	31
HVA	66	33	7.0	2.5	15	18	25.0	331
HVB	96	48	10.0	2.5	15	18	25.0	107

The vane instrument consists of 2 steel blades attached perpendicular to one another, which are connected to an extendable bar system [9]. As the regular swedish field vane is designed to be used in relatively soft clays it is of little to no use in clay till or clayey till. However, the danish field vane can be used to greater success due to its more robust design, which has its origin in surveying in firm clay till [18]. Vane configuration and geometry are presented in Figure 3.3 and Table 3.1. The vanes V4 - V9.2 are used for deeper investigations, where as the hand vanes, HVA and HVB, suffice in shallow surveys [7]. In accordance with Swedish standards, namely the requirement that the height of the vane instrument needs to be at least twice the diameter, deep vanes V4 - V7.5 can be used for testing the clay till in Skåne [9].

Calculations of the undrained vane strength differ between the Swedish and Danish methods. This is mainly due to the differences in vane instrument geometry. Essential for both methods is that an applied torque is divided by a geometrical factor [7][9]. For the Danish field vane the calculations are carried out as [7]:

$$\tau_v = \frac{P\, g\, a}{M} \qquad (kPa) \tag{3.14}$$

$$M = M_1 + M_2 + M_3 \qquad (m^3) \tag{3.15}$$

$$M_1 = \frac{1}{2}\, \pi\, d^2\, (h - 2r) \qquad (m^3) \tag{3.16}$$

$$M_2 = \frac{4}{3}\, \pi \left(\frac{d}{2}\, r^3 \right) \qquad (m^3) \tag{3.17}$$

$$M_3 = 2\, \pi^2\, r \left(\frac{d}{2} - r \right)^2 + \pi^2\, r^3 + 8\, \pi\, r^2 \left(\frac{d}{2} - r \right) \qquad (m^3) \tag{3.18}$$

where M is a measurement of the static moment of the total shear surface of the vane instrument. For the danish deep vanes, the value of a is almost constantly 0.3 m [7]. The calculated undrained vane strength, c_v, does not necessarily correspond to the undraind shear strength, c_u and there are a couple of different approaches to relating the two parameters. The procedure that is normally used in Sweden for overconsolidated soils is to correct the undrained vane strength based on the liquid limit and the overconsolidation ratio [24]:

$$c_u = c_v \left(\frac{0.43}{w_L}\right)^{0.45} \left(\frac{OCR}{1.3}\right)^{-0.15} \qquad (kPa) \tag{3.19}$$

On the contrary, in Denmark it is stated that the undrained vane strength is about the same as the undrained shear strength:

$$c_u \approx c_v \qquad (kPa) \tag{3.20}$$

However, when comparing field vane- and plate loading tests Jacobsen [15] found the undrained vane strength to be about 90 % of the undrained shear strength measured with the plate loading test.

Furthermore, the overconsolidation ratio and preconsolidation stress can be evaluated from the field vane test results [24]:

$$OCR = \left(\frac{c_v}{0.45\, w_L\, \sigma'_{v0}}\right)^{1.11} \tag{3.21}$$

$$\sigma'_c = {\sigma'_{v0}}^{-0.11} \left(\frac{c_v}{0.45\, w_L}\right)^{1.11} \qquad (kPa) \tag{3.22}$$

This evaluation method is based on Hansbo's relation, albeit slightly modified. These values are not to be seen as a replacement for CRS- or Oedometer tests [24].

Another approach to determine the preconsolidation stress, based on vane tests, was developed in Denmark. This empirical relation is used to obtain rough estimations of the preconsolidation stress, to be used for the consolidation phase in triaxial tests [18]:

$$\sigma'_c = \sigma'_{v0} \left(\frac{c_v}{0.4\, \sigma'_{v0}}\right)^{\frac{1}{0.85}} \qquad (kPa) \tag{3.23}$$

As the clay till is a non homogeneous material, it is difficult to determine an accurate failure surface around the instrument. The occurrence of rough particles at the boundary of the soil volume, being turned by the vane instrument, will disturb the test and provide values that are not entirely representative. The non homogeneous nature of the material might also create an uneven disturbance as the instrument is pushed into the ground [18]. The suggested solution to handle this source of errors is to conduct more tests in the same area. By doing so it is possible to mitigate the variation and eventually obtain a reasonable value for c_v with a greatly decreased uncertainty [2].

CPT-soundings are preferred over the field vane tests as the data output from the CPT presents a continuous reading over the the sounded depth, as stated in Section 3.1.1, whereas the vane test presents single values. The ability to receive a continous reading makes it far easier to follow possible trends in the soil profile, pick up details like lenses of sand and silt and also filter out excessive scatter. Moreover, the CPT is far more efficient as the field vane tests are conducted beneath the bottom of predrilled holes, meaning that a new segment of the hole must be drilled in between the individual vane tests [19], whereas the CPT is done in one continuous probing. Finally a smaller variation in the aquired data has been measured for the CPT compared to field vane tests [19].

3.1.3 Disturbed sampling – Non-continuous Flight Auger

A non-continuous flight auger is a method used to obtain disturbed samples. The equipment features a spiral flight wound round a central stem, which is fitted with a cutter head [32]. The majority of the standard augers have a diameter of 50 – 100 mm. The length of the spiral part of the equipment is commonly in the 500 – 2000 mm range. Above the spiral there is a shaft, which is attached to the bar system of the drill rig [32]. An example of a non-continous auger is presented in Figure 3.4.

Figure 3.4: Non-continuous flight auger, diameter 100 mm. Inspired by [32]

The non-continuous flight auger is often used in conjunction with the field vane tests. First, the flight auger is used to drill down to the desired depth for the vane test, it is then pulled up, and in the process a disturbed sample can be acquired from the material that is adhering to the auger flights [32]. When the flight auger is pulled up it is important not to rotate the equipment as that may cause unnecessary loss of sample material and further disturb the soil [32]. After the flight auger has created an open borehole, the field vane is pushed adequately into the bottom of the hole and the vane test is conducted. These three steps are repeated until the wanted total investigation depth is reached.

The sampling method is useful for investigations in cohesive soils with a certain limitation, based on the amount of big particles, namely stones and boulders [32]. It is important to note that it is hard to evaluate the natural bulk density and void ratio from these disturbed clay till samples as the sample is likely to swell due to the disturbance caused by the auger and by the sudden loss of confining soil pressure [18].

The disturbed samples obtained with the auger drill are, furthermore, useful in selecting depths for taking undisturbed samples As the equipment used for this process generally is less robust than the auger.

3.1.4 Undisturbed sampling – Piston Sampler

One of the commonly used methods to acquire undisturbed soil samples for further investigations in a geotechnical lab is the piston sampler. The equipment is classified as a composite sampler with a set of detachable internal plastic pistons for gathering the samples together with an easily changeable cutting edge [11]. The advantage is that parts that get worn out and/or damaged can easily be replaced instead of discarding the equipment as a whole [11]. For use in rough soils like clay till, where excessive wear can be expected this has a significant impact.

The equipment provides samples that retain the properties they had in-situ. This includes composition, structure, strength, density and water content [11]. However, it is not an easy feat to obtain undisturbed samples in a heterogeneous soil like the clay till. There is a constant risk of the sampling to be interrupted prematurely as the piston sampler encounters boulders, stones, gravel, and also firm lenses of friction soil [18]. Even if the attempt is not halted the sample can still be considered disturbed if rough soil particles have been squeezed into the pistons and/or pushed in front of the sampler.

It is at least possible to use the piston sampler in the Baltic clay till, which is considerably softer than the Northeast clay till and a lot less heterogeneous than the mixed clay till, and retain good samples [6]. Another method that has also been used successfully, for instance at Tornhill, is the Danish type of piston sampling [19]. This method uses a single metal tube, an "Americanerrör" for gathering the sample instead of a set of plastic pistons. In this case the sampler is not pushed into the ground, but instead hammered.

3.1.5 Dilatometer & Pressuremeter

The dilatometer- and pressuremeter tests share a lot of traits. Both methods revolve around measuring the lateral stress-strain properties of soils. This is essentially done by inflating a central part of the probe, seen in Figure 3.5. The inflation process is done in a couple of steps, at first the probe is pressurized until its central part makes proper contact with the surrounding borehole walls. Sequencing this is a couple of loading and unloading steps where the pressure in the probe is gradually increased and the response from the borehole walls is measured. From this process, it is possible to evaluate different strength-stiffness moduli, depending on which instrument is used.

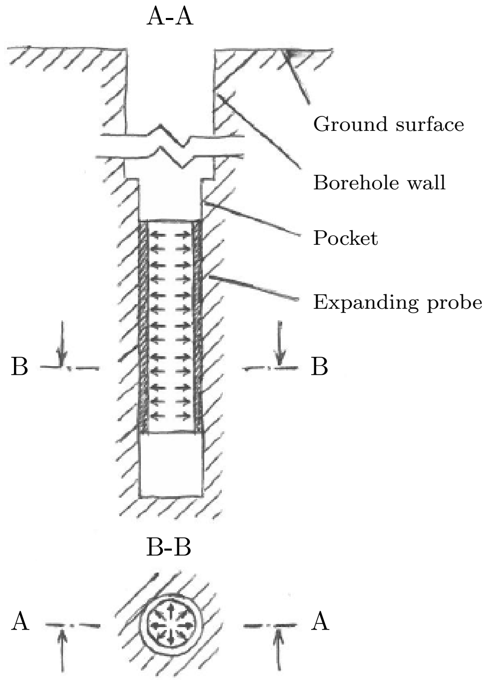


Figure 3.5: Principle Dilatometer/Pressuremeter, installed in a predrilled hole. Inspired by [34]

Furthermore, the coefficient of soil pressure at rest and the effective horizontal stress is evaluated. Lastly, there is also a couple of different ways to interpret the undrained shear strength from dilatometer measurements, for instance by Marchetti, Larsson and Roque et al [19]. The same is true for evaluation of the pressuremeter measurements, for instance by Gibson and Andersson, Mair and Wood, Menard and Briaud [19].

There are a couple of different ways of conducting dilatometer- and pressuremeter tests, which essentially revolves around how the instrument is installed into the ground. More robust versions of the probes can be equipped by a conic shoe and then thrust down into the soil profile to the desired depths [34]. There is also equipment that is denoted as self-boring, where the probe drills its way down into the soil profile instead of being pushed down [6]. Apart from these approaches, there is the alternative to pre-drill the holes and then further prepare the test cavities using special down-the-hole tools. When utilizing this approach it is important to install the equipment and proceed with the tests as soon as possible from the point in time when the cavity is prepared [19].

3.2 Laboratory Tests

3.2.1 Soil Classification

The routine tests conducted in a geotechnical laboratory serve as a way to classify the soil, but also measure basic properties such as bulk density, natural water content and porosity among others. Many of the measured parameters are connected to the relation between the material's three phases; solids, fluid and gas.

Initially, the volume and mass of the specimen is measured in order to determine the bulk density of the material as [36]:

$$\rho = \frac{m}{V} \qquad \left(kg/m^3\right) \tag{3.24}$$

Where m is the mass of the sample and V is its volume. The sample is then dried out in a oven at 105 °C for a period of time according to the European standard [35]. The natural water content in the sample is given by [20][35]:

$$w = \frac{m_{moist} - m_{dried}}{m_{dried} - m_{container}} \iff w = \frac{m_w}{m_s} \tag{3.25}$$

Where m_w is the mass of the water contained within the material, and m_s is the mass of the solids. To enable progression in the calculation sequence the particle density, ρ_s needs to be known. This is easily obtained by pulverizing a small amount of the dried soil and measuring the density of the powder with the aid of a pycnometer. The degree of saturation is then calculated as [20]:

$$S = \frac{\frac{m_w}{\rho_w}}{V - \frac{m_s}{\rho_s}} \tag{3.26}$$

Knowledge of the particle density also enables calculation of the porosity, and thereby of the void ratio [20] as:

$$n = \frac{V - \frac{m_s}{\rho_s}}{V} \tag{3.27}$$

$$e_0 = \frac{n}{1-n} \tag{3.28}$$

Finally, the grain size distribution featuring the clay content can be determined by the use of a combined analysis of sieving and sedimentation [37]. From this set of material data it is possible to use different empirical relations for calculating the undrained shear strength. These relations were established by Hartlén [13], and Jacobsen [15]:

$$c_u = 18\, {w_0}^{-2.05}\, {e_0}^{-1.88}\, {l_c}^{2.66} \quad (kPa) \qquad c_u \leq 200\ kPa \tag{3.29}$$

$$c_u = 10\, e^{\left(0.77\, {e_0}^{-1.2}\right)} \quad (kPa) \tag{3.30}$$

Hartlén's relation, Equation 3.29, is intended to be used for soft clay till whereas Jacobsen's relation, Equation 3.30, is to be used when evaluating coarser clay till and clayey till [13][15]. However, when these empiric relations have been compared to the undrained shear strength, measured with both field vane- and triaxial tests, at a later date, the correlation is bad [4].

Another empirical relation that is partly based on the current effective vertical in-situ stress, which in turn depends on the bulk density of the superimposing material was established by Ladd and Foott [17].

$$\begin{aligned} c_u &= a\ \sigma'_{v0}\ OCR^{\Lambda} \quad (kPa) \\ a &= 0.4 \quad \& \quad \Lambda = 0.85 \end{aligned} \tag{3.31}$$

The validity of this relation for use in clay till has have been subsequently tested by Steenfelt and Foged [44]. At this point the factors a and Λ were adjusted to fit the properties of clay till. The relation has been tested, to a limited extent, with somewhat promising results, in Sweden, by Dueck [5]. However, the method is sensitive to the accuracy with which the overconsolidation ratio is determined, which is problematic when working with clay till.

3.2.2 Fall Cone Test

Use of the fall cone apparatus is part of the routine investigations of an arbitrary soil sample in Sweden. The device is used to estimate the undrained shear strength of undisturbed samples and the liquid limit of both undisturbed and disturbed samples. As the fall cone test practically measures the strength properties of a very small portion of the sample it is very advantageous if the soil is homogeneous [43]. The clay till, however, is a non homogenous material, which results in the undrained shear strength, measured with the fall cone test is to be considered lightly when compared to other methods. The tests conducted with the fall cone apparatus are done according to European standard [29][33].

The apparatus shown in Figure 3.6 consists of a height adjustable stand fixed to a stable base plate. The adjustable part of the stand is fitted with a 0.1 millimeter scale, from which the cone penetration can be read. Held firmly by the stand is a fall cone that can posses different specifications according to Table 3.2. By pressing a button on the apparatus the cone will be instantly disengaged and for a brief period only be affected by gravity [29].

The main device is completed by a mixing bowl made out of iron, ceramic or plastic. The bowl's diameter needs to be at least 55 millimeters, its height 30 mm and its rim parallel to its base [29].

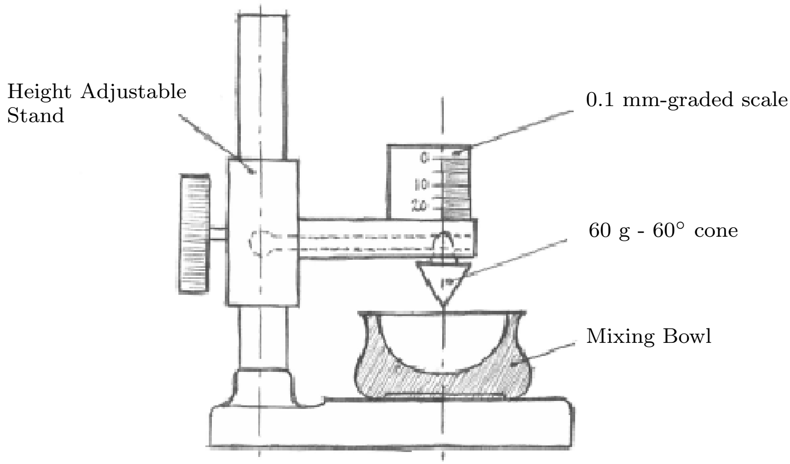


Figure 3.6: Standard fall cone apparatus. Inspired by [20]

Table 3.2: Properties of standard cones - mass and shape [29]

Mass	10 g	60 g	80 g	100 g	400 g
Tip angle	60°	60°	30°	30°	30°

When determining the undrained shear strength with a fall cone apparatus, the test is first conducted with a 60g/60°or an 80g/30°cone. For the measured penetration to be valid it needs to be in the interval 5 to 20 millimeters [29]. If the penetration is insufficient, a different cone needs to be used for the sequent tests. The undrained cone strength is calculated based on the measured penetration and the specifications for the used cone, according to:

$$c_k = K \cdot \frac{m_{cone} \cdot g}{i^2} \qquad (kPa) \tag{3.32}$$

where i is the penetration depth of the cone and m_{cone} its mass. The parameter K is a correction factor that is based on the apex angle of the cone. To calculate the undrained shear strength, c_u, the undrained cone strength, c_k is normally corrected according to the same factors as the undrained vane strength. This is treated in Section 3.1.2.

The other usage of the fall cone apparatus is to obtain a value of the liquid limit, w_L. This is done exploiting the linearity of the relation between the water content and the cone penetration in the vicinity of the liquid limit [20]. The test is carried out according the instructions presented in the European standard [33]. The methodology is to add distilled water to the prepared sample until the desired initial penetration is reached. At this point a sample is taken and the water content measured according to Equation 3.25. More distilled water is then added and the procedure is repeated until the interval between desired initial- and final penetration, according to Table 3.3, is covered in 4-5 tests. The results are plotted in a cone penetration - water content graph, and the water content representing the liquid limit is measured [33].

Table 3.3: Guidelines when determining w_L [33]

Requirement	80 g/30°	60 g/60°
Initial penetration	$\approx$ 15 mm	$\approx$ 7 mm
Final penetration	$\approx$ 25 mm	$\approx$ 15 mm
w_L determined based on penetration	20 mm	10 mm

An alternative is to use the Casagrande device when determining the liquid limit, though it is not advised according to the European standard [33],

3.2.3 Triaxial Test

The basic principle of the triaxial test is loading of a cylindrically shaped specimen in its axial and radial direction. The axial load is applied via a piston arranged to align with the specimens central axis, whereas the radial load is applied by pressurizing the surrounding fluid. To prevent the fluid from infiltrating, a thin rubber membrane is used to cover the sample [12]. As clay till samples normally feature coarse and sharp particles that protrude from its surface double membranes are often used to ensure the integrity of the specimen [18]. The top and bottom parts of the sample are confined by porous stones connected to drainage tubes. When the valves of the drainage are open, water is allowed to be squeezed out of the sample during the testing procedure, allowing for consolidation and drained tests. On the other hand, by closing the valves an undrained analysis can be conducted. During the undrained analysis the pore pressure increase is measured [12].

The test procedure involved in standard drained- and undrained analysis with a triaxial apparatus is covered in ISO/TS 17892-8 respectively ISO/TS 17892-

9 [30][31]. Figure 3.8 shows the setup, for example, used in a consolidated undrained triaxial test according to European standard. However, there are a lot of variations of both the drained- and undrainded test which differ from the European standard. The reason behind these test variants is that they can evaluate the properties of a specific soil type or soil material better than the standard procedure.

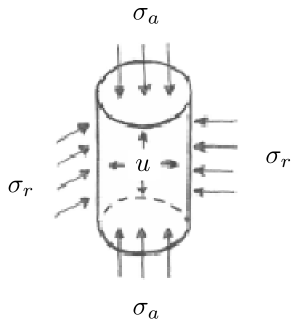


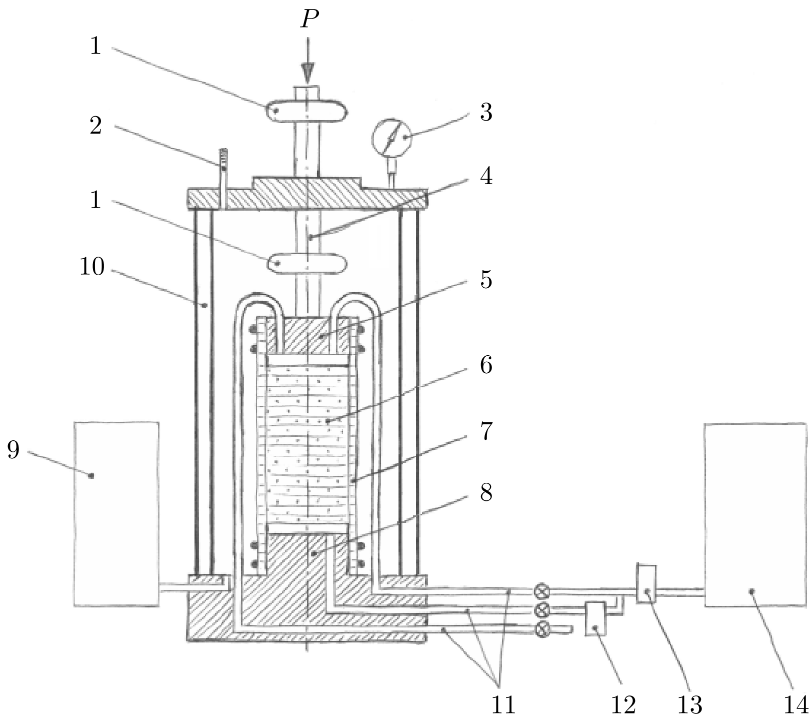
Figure 3.7: Triaxial test stress orientation. Inspired by [12]

Output data from a triaxial text is fairly extensive and consists of stresses in the axial- and radial direction, σ_a, σ_r, pore pressure,u, shown in Figure 3.7, strain in the axial direction, ε_a, together with the change in volume, ε_{vol} [12].

In a triaxial compression test, measuring the active soil strength, the pressure in the axial direction acts as the large principle stress, σ_1, and is based on the load applied by the piston, but also the load applied by the pressurized cell fluid. The pressure in the radial direction acts as the small principle stress, σ_3, and is directly linked to the cell pressure. The stress measurements are usually reformulated in terms of mean stress, p, and deviatoric stress, q, which are related to the principal stresses as:

$$p = \frac{(\sigma_1 + 2\,\sigma_3)}{3} \qquad (kPa) \tag{3.33}$$

$$q = \sigma_1 - \sigma_3 \qquad (kPa) \tag{3.34}$$



No	Description
1	Alternative positions for load measuring device
2	Air bleed
3	Vertical compression measuring device
4	Piston
5	Top cap
6	Soil specimen
7	Membrane
8	Pedestal
9	Device for measurement and control of cell pressure
10	Triaxial cell
11	Drainage Tubes
12	Pore pressure sensor
13	Volume change sensor
14	Device for measurement and control of back pressure
P	Vertical load

Figure 3.8: Standard triaxial setup. Inspired by [31]

One of the variations of the undrained triaxial procedure is the Area Constant Undrained with $u = 0$ ($ACU_{u=0}$). This type of test is commonly used in Denmark when investigating undrained shear strength in clay till [18]. What distinguishes this method is that the in-situ stress situation- and history is recreated as far as possible [4]. This makes the test well suited for investigation of clay till as the consolidation procedure counteracts the disturbance that is inevitably caused to the specimen as these are sampled.

The most distinctive feature of the test is the anisotropic consolidation phase, which allows the current in-situ stresses to be recreated. During this stage 85-90 % of the preconsolidation pressure is gradually applied to the sample by the axial piston. Meanwhile, the cell pressure is continuously adjusted to keep the cross section of the sample constant, by keeping the relation between the axial strain and change in volume constant [4][18]. The consecutive step is to unload the sample to its measured in-situ vertical pressure while adjusting the cell pressure in the same way as when the sample was first consolidated [18]. At this point the sample is in a state that is very similar to in-situ. The knowledge of both vertical- and horizontal pressures enables evaluation of the coefficient of earth pressure at rest in the soil's natural state [4]. Furthermore, the relation between the coefficient of earth pressure at rest and the overconsolidation ratio can be studied during the anisotropic consolidation phase. This relation is described empirically as:

$$K_0 = K_{0nc}\, OCR^{\alpha} \tag{3.35}$$

$$K_{0nc} = 1 - \sin(\phi') \tag{3.36}$$

Where K_{0nc} is the value of the earth pressure at rest for a normally consolidated soil and α is a material parameter [4]. During the consolidation phase in the triaxial test, K_0 for overconsolidated samples approaches K_{0nc} [4]. The general expression for α, together with the empirical value for clay till presented by Steenfelt and Foged is [44]:

$$\alpha = \begin{cases} \sin(\phi') \\ \\ 0.45 \pm 0.03 \end{cases} \tag{3.37}$$

A load step that is part of the anisotropic consolidation phase begins and ends with the same sample diameter. This is done according to the following sequence of actions:

1. The cell pressure is increased. This increases the change in volume, ε_V when the sample shrinks both radially and axially
2. The deviator stress which is used to regulate the axial strain, ε_1, need to be increased for ε_1 to approach ε_V. The sample shrinks axially and expands radially.
3. While increasing the axial load the cell pressure is lowered slightly until $\varepsilon_1 = \varepsilon_V$ is reached

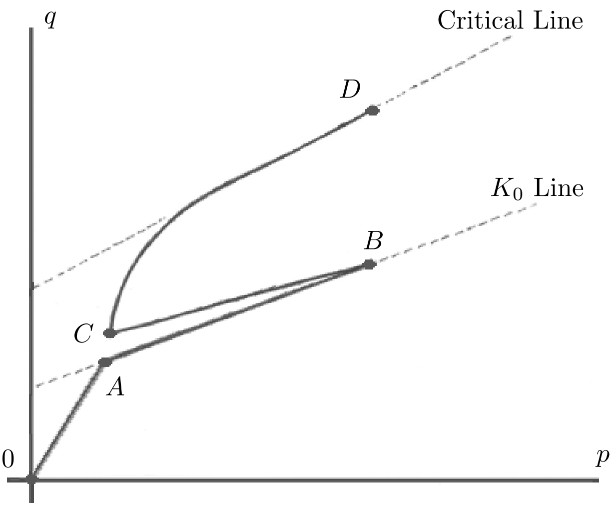


Figure 3.9: Stress path during an $ACU_{u=0}$ test. Inspired by [4]

Sequential to the anisotropic consolidation phase, the sample is tested to failure. In this variant of the triaxial test the sample volume is kept constant, $\varepsilon_V = 0$ and the pore water is constantly in contact with the atmosphere pressure, $u = 0$ [4]. This set of constraints creates a test that features both drained and undrained aspects, which is suitable for a material that is not fully saturated in its natural state. Instead of achieving $\varepsilon_V = 0$ by keeping the drainage valves shut according to the standard procedure, the cell pressure is changed throughout the test [4].

A description of the stress path during the consolidation and loading to failure is presented in Figure 3.9. The sample, mounted in the triaxial apparatus, starts out at 0. It is then anisotropically consolidated with constant cross section area from point A to point B, following the K_0-line. When unloading the sample to its in-site overburden pressure C is reached. From this point the sample is loaded to failure. Point D notes where along the critical line the failure occurs [4].

In contradiction with European standard, Danish tradition suggests that a sample with height equal to the diameter should be used [1]. The European standard, on the other hand, states that the height of the sample should be twice its diameter [31]. The idea behind having a sample with height twice the diameter is to reduce the influence by the roughness of the pressure heads. According to Danish practice, this problem is instead managed by using smooth pressure heads. This essentially consists of covering the pressure heads with a set of rubber membranes with grease in between, reducing the friction at the ends of the sample drastically [1]. The reason behind the Danish approach is to provide a homogenenous stress-strain field in the sample and thereby create a test that mimics a theoretical element test better. The European standard test is likely to create "dead zones" where the sample is in contact with the pressure heads [1]. Moreover, the test setup and the shape of the sample might influence what type of failure is most likely to occur.

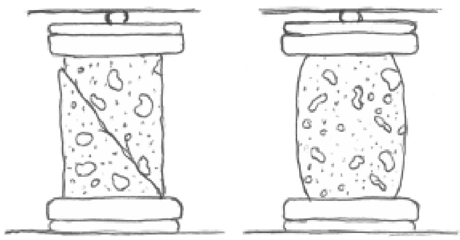


Figure 3.10: Different types of failure in a triaxial test, bifurcation (left) and diffuse strain localization (right)

The most common types of failures are seen in Figure 3.10. The left sample shows a failure by bifurcation. Due to the soil's dilative nature it is likely that water is flowing into the shear band from the adjacent unfractured material, which further weakens the shearing zone [1]. The right sample, on the other hand, displays a failure by diffuse stain localization. Visually the sample is bulging out along its sides as the strain increases at about constant stress.

3.2.4 Oedometer test

The oedometer test is a one dimensional consolidation test used for evaluating of the compressibility characteristics of soils [38]. To ensure that the consolidation only takes place in the vertical direction the sample is encased in a steel ring with very high relative stiffness. The ends of the sample are fitted with filter papers and porous disks to ensure that pore water can be squeezed out during the tests, allowing consolidation of the sample.

Furthermore, the load cell is submerged in water. This ensures that the sample does not dry out during the test as it is in direct contact with the water bath via the filter papers and the porous disks. The water container featuring the load cell is installed in a loading rig, which utilizes a lever arm to amplify the applied weights. A schematic image of the oedometer is seen in Figure 3.12.

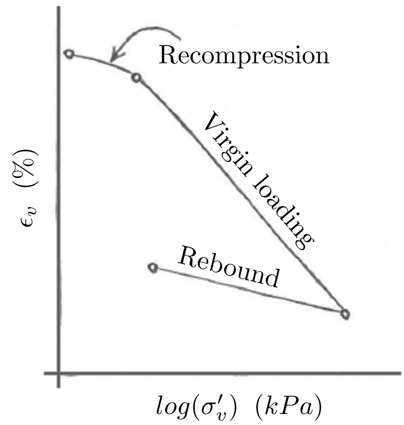
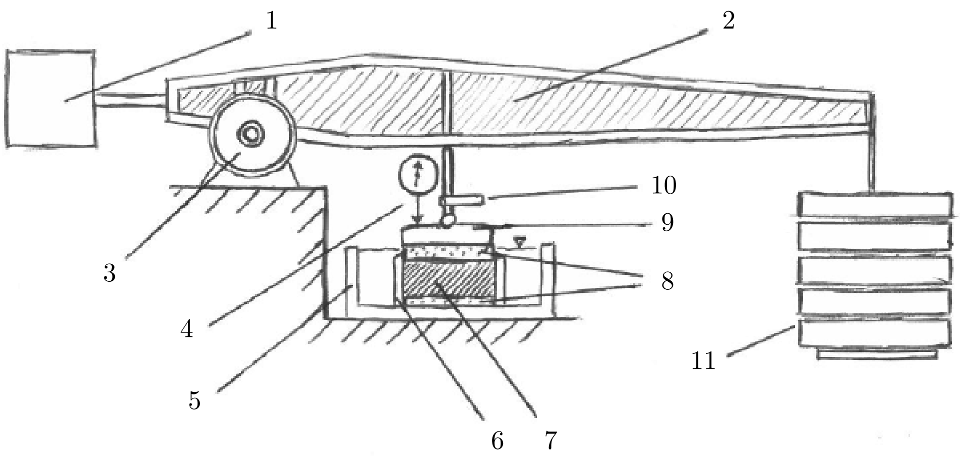


Figure 3.11: Oedometer test curve

In the oedometer test, the soil sample is incrementally loaded. During each step the change in sample height is measured and plotted for the duration of the step. When the change in height is evening out, it is time to add another load increment, as this indicates that the pore pressure build up caused by the previous step has dissipated. The sample is also unloaded in steps in order to obtain a rebound curve, which has a similar inclination to the first part of the loading curve, the recompression. The ideal stress - strain path during an oedometer test is presented in Figure 3.11. For evaluation of the oedometer test, the change in void ratio is plotted against the logarithm of the effective vertical stress in the sample. The stress situation can be considered effecive as the pore pressure build up is given time to dissipate during load steps and the water pressure caused by the surrounding bath is negligible. In an ideal oedometer test, the preconsolidation stress is defined as the intersection between the tangent of the recompression part of the load curve and the virgin part. However, due to sample disturbance this is often impossible. Instead the Europen standard suggests using the Cassagrande method when estimating the preconsolidation stress [38]. Furthermore, depending on the resolution and processing of the test data it is possible to divide the strain into the following parts; initial, consolidation and creep [4].



No	Description
1	Counterweight
2	Lever arm
3	Joint
4	Vertical compression measuring device
5	Water container
6	Oedometer ring
7	Sample
8	Porous disks
9	Top cap
10	Load measuring device
11	Weight

Figure 3.12: Oedometer

4 Conducted tests

4.1 Site I - Lund - Tornhill

4.1.1 Previously conducted tests

As mentioned in Section 2.2.1 the site Tornhill has been involved in previous investigations by Ann Dueck, LTH [3][4], and Rolf Larsson, SGI [19]. A significant amount of data acquired in those studies have been incorporated in this report.

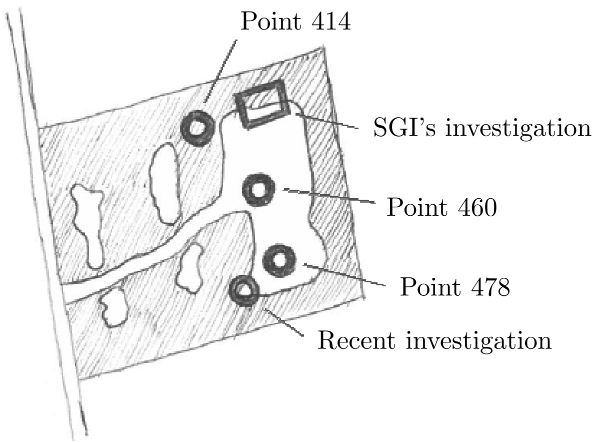


Figure 4.1: Tests conducted at Tornhill

From the initial study conducted by Dueck [3], the incorporated data include:

- results from laboratory routine tests in line with what has been described in Section 3.2.1;
- measured data from field vane tests in two points, 414 and 478 (see Figure 4.1);
- a selection of CPT-data acquired in the vicinity of either field vane- , triaxial- or oedometer tests, points 414, 460 and 478 (see Figure 4.1);

- groundwater measurements.

During this study, 25 CPT-soundings were conducted. The majority of them reached a depth of about 7 meters and showed the same trend over the investigated depth [3]. This trend is well represented in the utilized CPT-data points.

From a later study by Dueck [4], where undisturbed samples were obtained and both triaxial- and oedometer tests were conducted in Aalborg according to Danish practice, the following is included:

- results from Triaxial tests, $ACU_{u=0}$, points 460 and 478 (see Figure 4.1);
- results from Oedometer tests, points 460 and 478 (see Figure 4.1).

Undisturbed samples were obtained using both tube sampling, and a triple tube core drilling system.

The most recent documented study from Tornhill was conducted by Larsson [19]. The most significant feature of this project was a full-scale plate loading test. However, a wide array of complimentary tests were conducted in the close vicinity of the main test rig. Out of these the following have been included:

- results from Triaxial tests, CU;
- results from Oedometer tests;
- a CPT-sounding adjacent to the point where tube sampling was conducted;
- pressuremeter- and dilatometer measurements of the lateral stress in the soil profile, σ'_h.

Undisturbed specimens were obtained using both tube sampling and triple tube core drilling. The specimens obtained with the tube sampler were considered to be of a higher quality [19]. A 10 ton drill-rig was used in this project in an attempt to penetrate further into the Northeast clay till using the CPT than with a standard 5 ton rig. However, this was not successful and a deeper penetration than 7 meters was not achieved.

4.1.2 Recently conducted tests

Two new CPT-soundings were conducted in the southern area of the field, points 1 and 2, presented in Table 4.1). This was done to obtain CPT-data with better resolution as the original CPT-data from Dueck's first investigation were not available and have been, therefore, manually recreated from drawings. The CPT-soundings were carried out according to class 3 and penetrated to 6.96 and 7.02 meters respectively.

Table 4.1: Coordinates, Tornhill - SWEREF 99 13 30

Notification	N	E	h
CPT 1	6178990.79	130824.24	62.42
CPT 2	6178990.57	130825.40	62.43

Close by the CPT-points a combination of auger sampling and field vane tests were performed. This was mainly done to ensure that no significant deviations in the strength properties of the clay till existed in this particular spot, while at the same time contributing to the collection of data available at the location

Auger samples were obtained for the intervals 1-2 m and 2-3 m in the Baltic clay till, and 3-4 and 4-5 in the mixed clay till. Vane tests were carried out at the depths 2, 3.3, and 4 m using the danish field vane V4. Vane tests were evaluated according to Danish standard [7].

Piston sampling was attempted in the same location and a sample was obtained from the depth 3.3 m. The sampler itself was complemented with a pressure gauge to make sure that the equipment was not forced to a point where the risk of damage was considered to be too high.

To further monitor the groundwater level at the site an open tube was installed and the depth to the ground water table measured. During the spring of 2017 the level varied between about 0.5 and 1.2 meter. Determination of bulk density, water content, liquid limit and grainsize distribution has been determined for samples acquired from 1-2 m, 2-3 m, 3-4 m and 4-5 m. The auger samples have been used for determination of bulk density. Archimedes principle was used to measure the volume of 4 relatively undisturbed specimens from each sampling level. The specimens were completely submerged in water and the weight generated was measured. After the volume measurements the specimens were carefully dried and weighted again to make sure that no significant amounts of water had infiltrated. None of the tested specimens had a change in mass exceeding 0.5 % of their total weight. The use of Archimedes principle to determine the volume of clay till auger samples was also used by Dueck [3]. Determination of the natural water content was carried out according to the European standard [35]. One sample per depth interval was used.

The grain size distribution was determined with a combined analysis. First the whole sample was washed in a 0.063 mm sieve in order to separate the fine soil particles. The coarser particles were then sieved while the solution holding the fine material was first oven dried until almost all water was eliminated and then freeze dried. Subsequently, a hydrometer analysis was conducted in a room with constant temperature and humidity at Lund University. One sample per level was tested.

Finally, the liquid limit was measured following the European standard [33]. To separate particles larger than 0.5 mm, the suggested wet method was used. The resulting sample-slurry was then dried to the point where the first cone penetration was about 7 mm, using a 60g/60°-cone.

Due to issues with the triaxial apparatus at LTH, the planned trixial compression tests were unfortunately canceled.

4.2 Site II - Malmö - Lergöken

Two CPT points were established close to the central brush in the area, points 3 and 4. The CPT-soundings were carried out according to class 3 and penetrated to 6.70 and 6.72 meters respectively. Their coordinates are presented in Table 4.2.

Table 4.2: Coordinates, Lergöken - SWEREF 99 13 30

Notification	N	E	h
CPT 3	6158929.37	120150.63	35.10
CPT 4	6158929.63	120150.20	35.12

Close by the CPT-points a combination of auger sampling and field vane tests was conducted. This was done to verify the roughly estimated soil profile for the site and to get an idea of the undrained shear strength in the material. Auger samples were obtained for the intervals 2.3 - 3 m and 3 - 4.8 m in brownish sandy clay till, 4.8 - 5.5 m in silty sand, and also 5.5 - 7 in grey sandy clay till. Superimposing the glacial deposits, are the following layers; sandy topsoil 0 - 0.8 m and pure sand 0.8 - 2.3 m.

Vane tests were carried out at the depths 3 and 4 m using the danish field vane V4. Vane tests were evaluated according to Danish standard [7]. The vanes almost reached their max capacity in the assumed Baltic clay till and it was considered ineffective to use the field vane test in the lower clay till layer as this was judged to be much stiffer, based on the auger samples acquired.

Piston sampling was attempted at the depth 3 m but it was impossible to push the equipment into the soil without damaging it. To monitor the groundwater level at the site an open tube was installed. The depth to the groundwater table has been recorded to be between 3.0 - 4.0 meters.

Determination of bulk density, water content, liquid limit and grainsize distribution has been determined for samples acquired from 2 - 2.3 m, 2.3 -4.8 m, 4.8 - 5.5 m and 5.5 - 7 m. The procedure followed was the same as for the samples acquired at Tornhill, 4 specimens where chosen for bulk density measurement whereas a single specimen was used in the following tests. Special care was taken when handling the sandy silt specimens during the bulk density test as these specimens were not as firm as the clay till ones. Nevertheless, sandy silt specimens experienced a small change in volume during the test, less than 1 %.

4.3 Site III - Staffanstorp - Torreberga

No further tests have been conducted at Torreberga, and the data presented were acquired during a project carried out by Dueck and Garin [6]. The utilized data consists of:

- results from laboratory routine tests in line with what has been described in Section 3.2.1;
- measured data from field vane tests in one point, 5, at depths that corresponds to clay till;
- CPT-data from one point, 5, at depths that corresponds to clay till;
- estimated lateral pressure from pressuremeter tests.

5 Results

5.1 Site I - Lund - Tornhill

The investigations carried out by Dueck [3] and Larsson [19] provide a very comprehensive description of the soils found at Tornhill. The specimens tested in the current project fall within the bounds set by previous investigations. The parameters can be divided into two groups, one that describes the material composition of the soil, and one that describes the stress state.

In Figure 5.1 the representative soil parameters: natural water content, liquid limit, void ratio, and clay content, have been presented along with estimated trend lines for their variation with the depth. From this distribution the soil was initially divided into 3 different layers, Baltic-, mixed- and Northeast clay till, by Dueck [3]. The liquid limit and the clay content share a trend of high values in the first 3 meters of the soil profile and then steadily decreasing. It has been shown by Dueck [3] that the liquid limit is fairly linearly related to the clay content, according to the approximated relation:

$$l_c = 0.97 + 0.92\ w_L \tag{5.1}$$

The relation between the clay content and the liquid limit is strikingly obvious when handling specimens of different types of clay till during the liquid limit test. Moreover, there is a shared trend between the void ratio and the natural water content, which is logical as the content of water in the material is directly influence by the porosity.

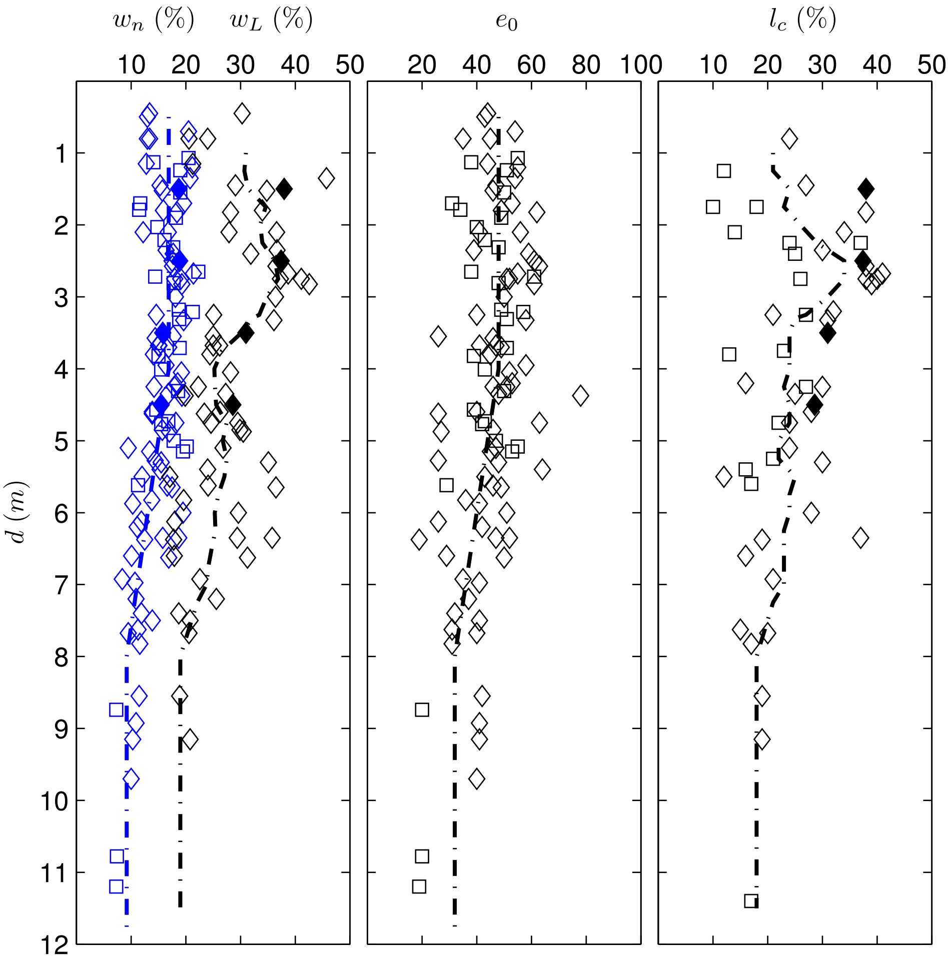


Figure 5.1: Soil properties - natural water content, liquid limit, void ratio and clay content. Squares - measurement by Larsson, diamonds - measurements by Dueck, filled diamonds - measurements from present project

The second set of parameters, presented in Figure 5.2, shows the preconosolidation stress, the horizontal stress, and also their relations to the effective overburden stress, OCR and K_0 respectively. The preconsolidation stress and the corresponding parameter OCR are almost exclusively evaluated from Oedometer tests, excluding the 3 measurements below 9 meters that are evaluated using a triaxial apparatus. The horizontal stress and the parameter K_0 are evaluated from dilatometer and pressuremeter tests. Two series of measurements have been carried out with both dilatometer and pressuremeter. Both the preconsolidation stress and the lateral stress originate from the consolidation of the soil by the glacial ice mass, as the horizontal stress originates from hindered lateral deformation caused by the consolidation stress, they should be linked via Poisson's ratio.

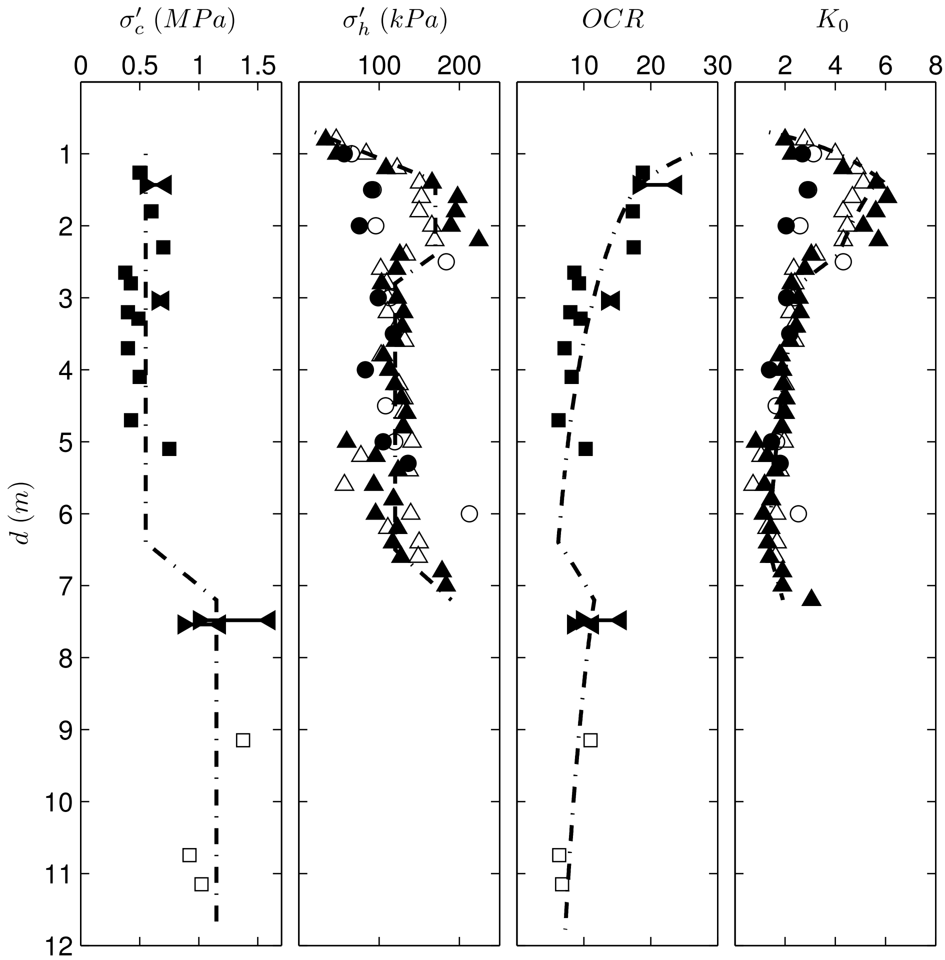


Figure 5.2: Vertical and Lateral stress situation. Circles - pressuremeter tests, triangles - dilatometer tests, filled squares - oedometer tests, squares - triaxial tests, triangular-spanned intervals - oedometer tests by Dueck

The preconsolidation stress evaluated using the empirical relations presented for field vane tests, and also CPT-soundings are of an entirely different magnitude than the parameter evaluated from oedometer tests. For vane test utilizing Equation 3.23, and also CPT-data evaluated using the relation proposed by SGI a preconsolidation stress that is about 50-100 % higher than the oedometer results was obtained. When evaluating the preconsolidation stress from CPT-data, exploiting Robertson's relation the difference was even larger. However, due to disturbance caused by sampling, and also handling of the specimens used in the oedometer tests the preconsolidation stress evaluated in the laboratory may serve as a lower bound.

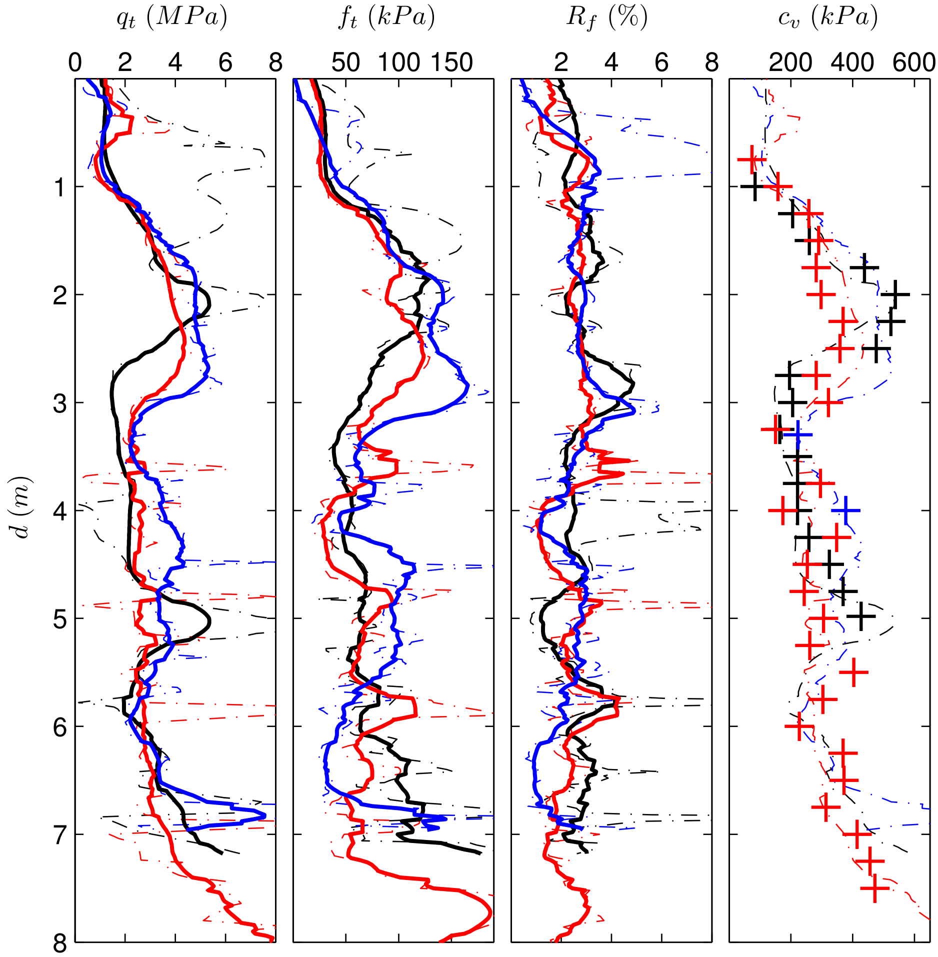


Figure 5.3: Processed CPT-data from Tornhill, CPT 414 - black, 478 - red and 1 - blue, and also a comparison to field vane tests

From the CPT, the key parameters q_t and f_t have been filtered to obtain a cone resistance- and sleeve friction distribution for further evaluation, this is shown in Figure 5.3 along with the friction ratio, R_f. Processed data is plotted as solid lines, whereas raw data for comparison purpose is plotted as dash-dotted lines. Furthermore, a comparison to the conducted vane test are included. The dash-dotted lines in the vane plot represent measured cone resistance, of the adjacent CPT-sounding, divided by 10.

The method proposed by Mortensen described in Section 3.1.1 was initially used to filter the CPT-data. This processing was, however, considered not sufficient. A basic floating mean value filter was also tested, filtering over 20 data-points. This filter had issues handling thicker lenses of coarse soil, and it excessively decreased the resolution of the data. The filter that was finally used is based on Mortensens theory with a doubled floating data-interval and parameter f_l of 0.2. The filter was supported by manually pre-smoothing thicker lenses of friction soil, when it was obvious that the adjacent vane tests have failed to hit the lens.

Lastly, the results from the triaxial tests, conducted by Dueck [4] and at the SGI [19], are presented in the table below:

Table 5.1: Triaxial test results, Tornhill

Depth (m)	c_u (Dueck) (kPa)	c_u (SGI) (kPa)
1.15		160
2.20	180	175
2.70		128
3.00	350	
3.20		105
3.80		223
4.25		130
4.75		115
5.15		118
5.60		140
7.00	1025	
9.18	825	

The majority of triaxial tests from the Baltic and mixed clay till presents suspiciously low values. However, the undrained shear strength evaluated from triaxial tests in clay till are highly dependant on how the tests are conducted and the quality of the samples. This is further discussed in Section 6.

5.2 Site II - Malmö - Lergöken

The data sample from Lergöken involves soil classification parameters for the Baltic and Northeast clay till, and also the interglacial sandy silt presented in Table 5.2 as well as strength parameters measured by CPT and field vane tests, Figure 5.2. The data is smoothed using the same filter as for the CPT-data from Tornhill.

The clay till layer with an upper bound at about 2.3 meters below the ground surface is brownish and features a high content of sand. This clay till is significantly firmer and more brittle compared to the Baltic clay till at Tornhill. Within the layer, both the clay content and liquid limit increase with depth.

The Northeast clay till is located beneath a layer of interglacial sandy silt. The material contains more coarse particles, has a low clay content, and is very firm.

Table 5.2: Soil parameters, Lergöken

Depth (m)	w_n (%)	w_L (%)	l_c (%)
2.6	12.7	20.7	13
3.9	12.0	22.9	16
5.2	20.9	20.0	10
6.3	11.9	19.0	10

The layer denoted as sandy silt, between 4.8 and 5.5 meters below the ground surface, based on the disturbed samples obtained using the flight auger, corresponds well to the layer with exceptionally high cone resistance seen in Figure 5.2.

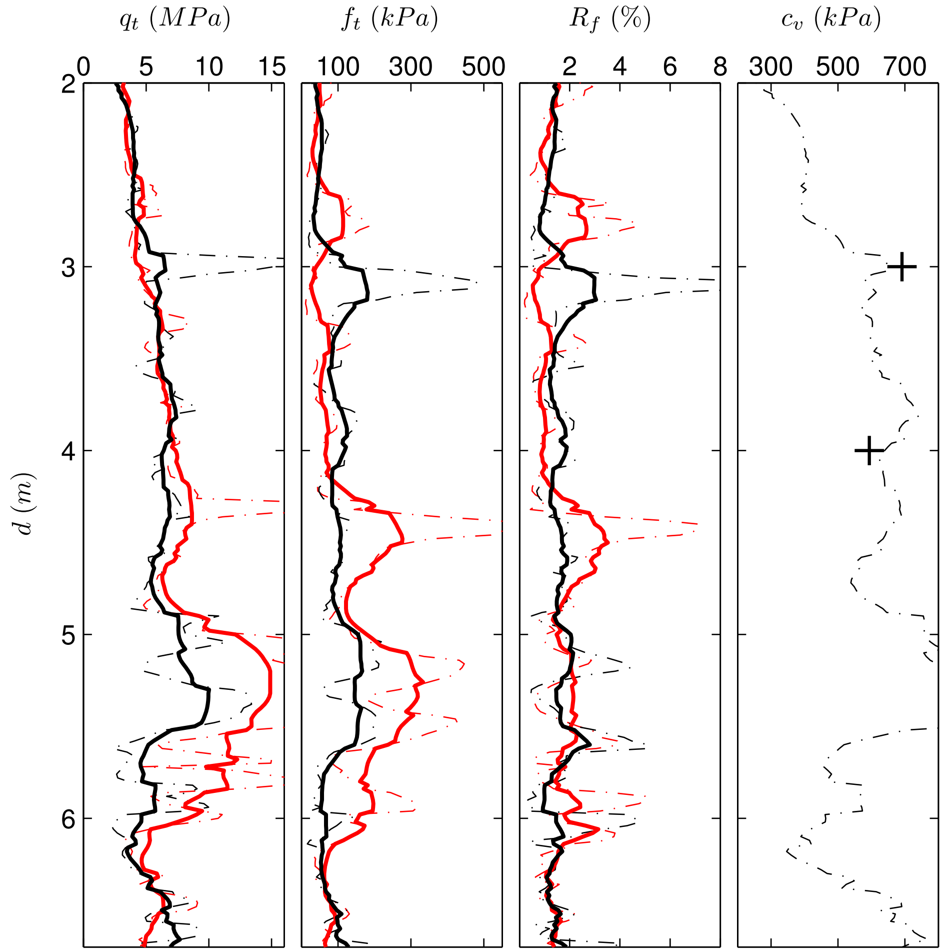


Figure 5.4: Processed CPT-data from Lergöken, CPT 3 - black, CPT 4 - red, and also a comparison to field vane tests

5.3 Site III - Staffanstorp - Torreberga

From the investigation carried out by Dueck and Garin [6] basic soil classification parameters, strength parameters, measured with both CPT and field vane tests, and an estimate of the lateral stress situation have been used for Torreberga. The data represents the layer of clay till located between the depths 5 and 8 meters, out of which the top 0.1 - 0.2 meters resembles silty clay. The clay till deposit continues below 8 meters. The soil classification parameters are presented in Table 5.3.

Table 5.3: Soil parameters, Torreberga

Depth (m)	w_n (%)	w_L (%)	l_c (%)	K_0	σ'_h (kPa)
5.1	21.1	27	30		
5.3	17.5				
5.5	16.0	21			
6.3	13.2	18.9	30		
6.5				1.7	156
6.7	13.0	20			
7.0				2.0	255
8.0				1.8	275

The clay till is denoted fine with a generous clay content. The available data indicates that most of the parameters seem fairly constant over the depth, though there is a trend of increasing lateral stress in the clay till. This, however, is not entirely reliable as the lateral stress, at depths 7.0 and 8.0 meters, is evaluated at higher developed strains than usually when using a pressuremeter [6].

Measurements from CPT and field vane tests are presented in Figure 5.5. The data is smoothed using the same filter as for the CPT-data from Tornhill. However, the level of filtering needed was relatively low. Based on the not so jagged appearance of the CPT-data, the high clay content and the successful use of a standard piston sampler to obtain samples, it seems that the clay till at Torreberga is relatively homogeneous and contains fewer coarse particles than the other investigated clay tills. It also seems to be softer.

The ground water level was measured in an open hole and with piezometers. The groundwater table was determined to be located 1.2 meter below the ground surface, with a significant variation throughout the year.

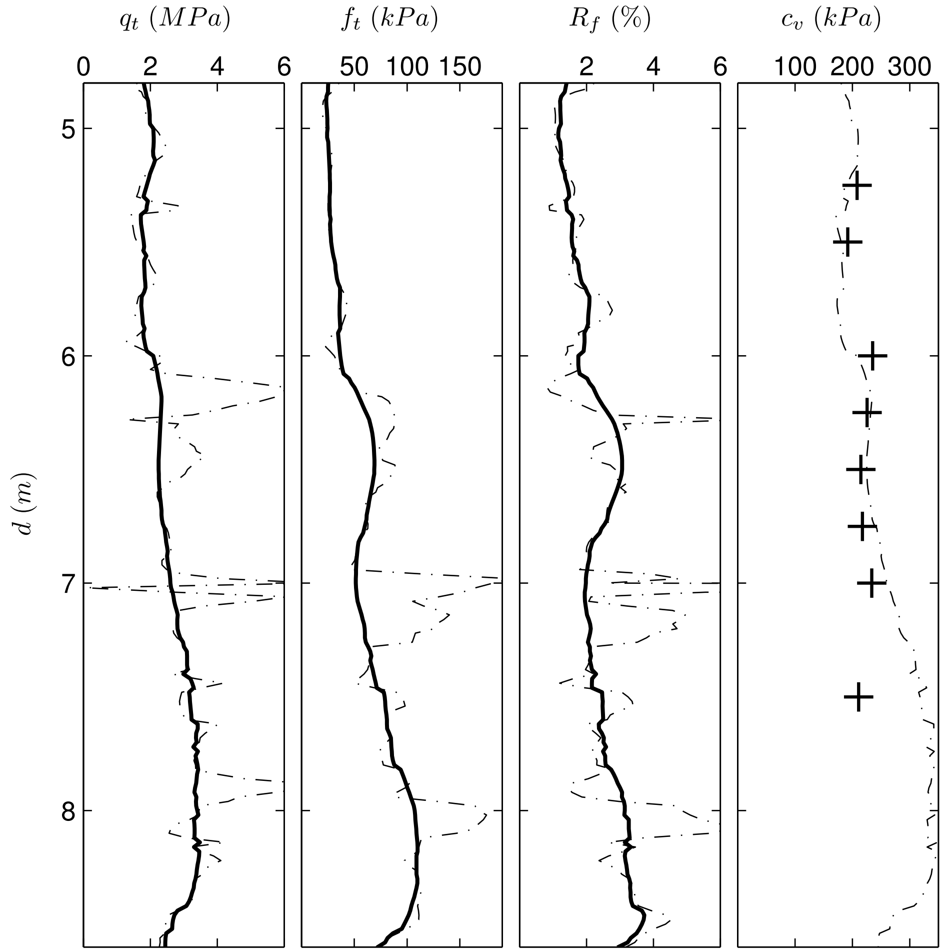


Figure 5.5: Processed CPT-data from Torreberga, CPT 5, and also a comparison to field vane tests

5.4 Evaluated cone factors

The cone factor has been evaluated, comparing filtered net cone resistance to undrained vane strength, for the five different types of clay till investigated in this project, and is presented in Figure 5.6:

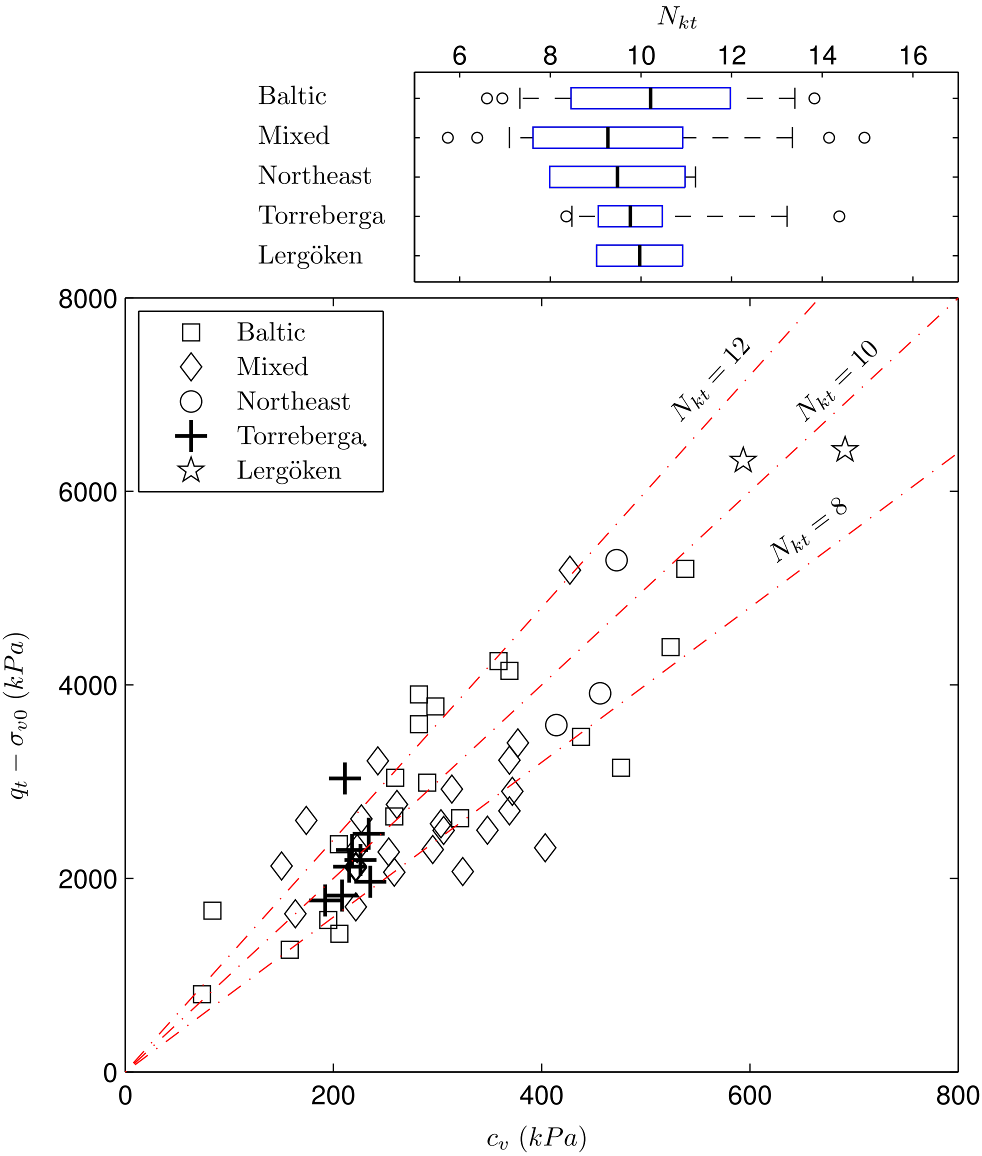


Figure 5.6: N_{kt}, evaluated according to $(q_t - \sigma_{v0})/c_v$

For this evaluation the different types of clay till are denoted as Baltic, Mixed, Northeast, Torreberga and Lergöken, out of which the first three originates from Tornhill.

The bottom part of the Figure 5.6, shows the comparison as a whole. The different variants of clay till covers the range of undrained shear strengths from approximately 100 - 700 kPa. The top part of Figure 5.6 presents the plausible cone factors for the different types of clay till in the shape of a box and whiskers plot. The data presented features the mean value of the cone factor along with its standard deviation, and also the tenth- and ninetieth percentile. Values that exceed the outer limits are denoted outliers and do not contribute to calculation of the mean value and standard deviation. It is not possible to either determine, or rule out, that the same cone factor can be used for all of the different soils, based on the available statistical data.

Furthermore, the relation Luke proposed for evaluation of the cone factor, Equation 3.5, was investigated. This was done by comparing the relation to the cone factors presented in Figure 5.6, and their corresponding friction ratio.

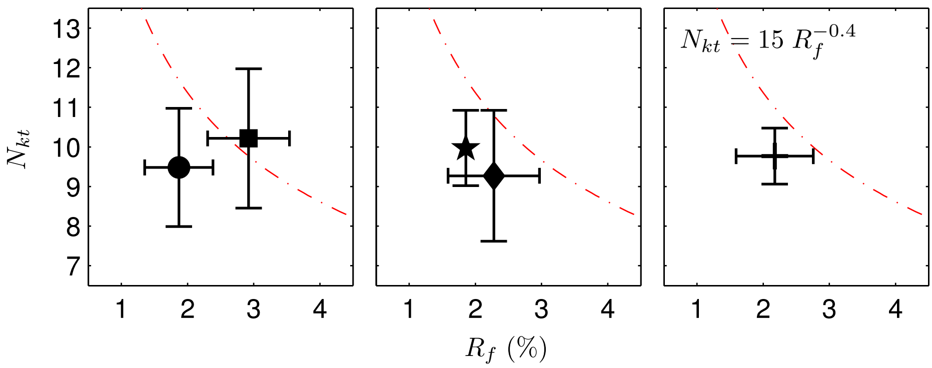


Figure 5.7: Evaluated cone factors compared to Luke's relation

In the Figure 5.7 the obtained cone factors are plotted as a cross, the midpoint represents the mean cone factor and the corresponding mean friction ratio, the horizontal bar represents the standard deviation of the friction ratio and the vertical bar translates to the standard deviation of the cone factor. A square and a circle are used in the left plot to denote Baltic respectively Northeast clay till, whereas a diamond and a star are used in the middle plot to denote Mixed clay till and the clay till encountered at Lergöken. In the right plot the data from Torreberga is presented.

Lastly, in Figure 5.8 four different approaches to determine the cone factor for the clay tills at Tornhill are presented. The approaches are the following: static values of 10 and 11, a semi dynamic solution exploiting the mean values from figure 5.6 and a dynamic solution based on Equation 5.2.

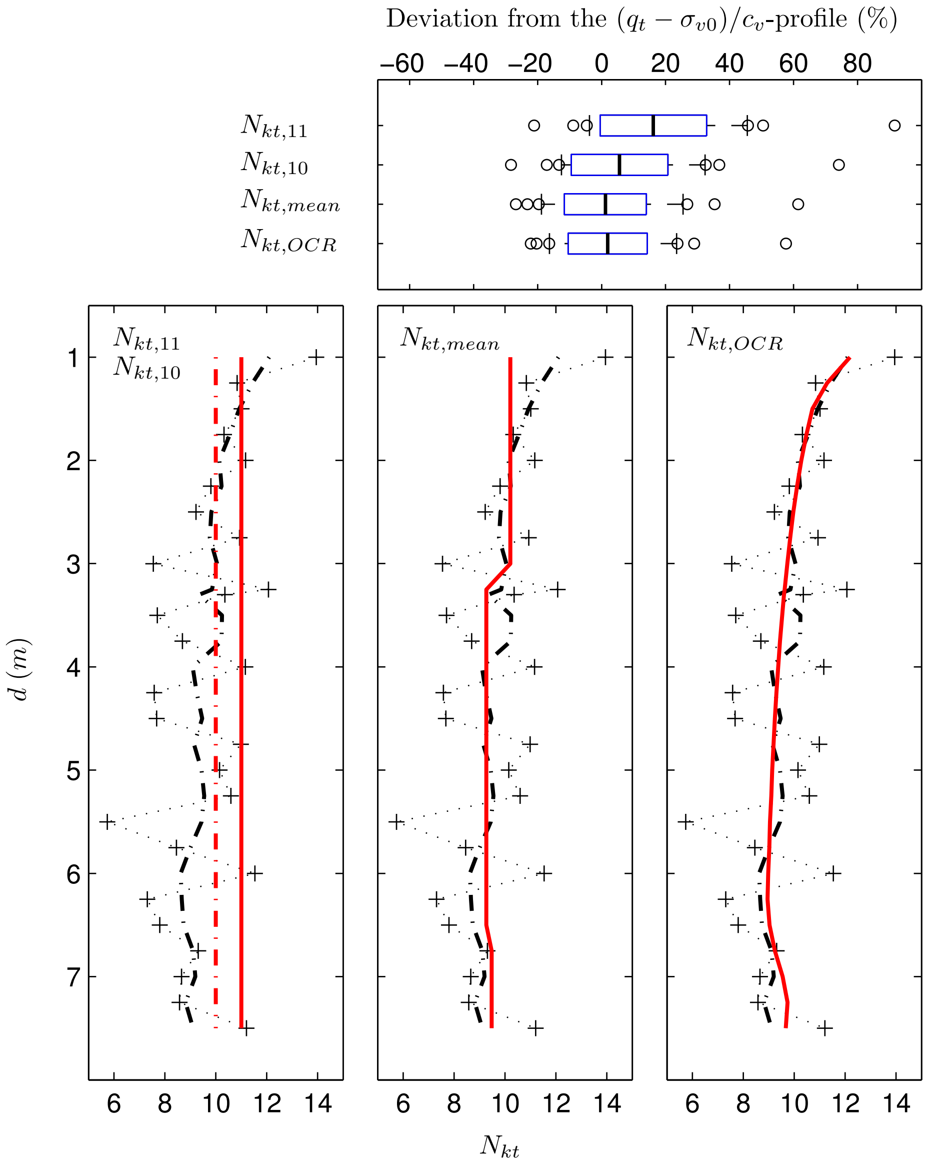


Figure 5.8: Comparison between the evaluated $(q_t - \sigma_{v0})/c_v$-profile and different estimations of N_{kt}

The different approaches are compared to the values of the cone factor received from comparing field vane tests to CPT-soundings. At depths where multiple field measurements exists, a mean value has been chosen. The evaluated profile for the cone factor is shown in the figures as a dotted black line, along which, depths with conducted field vane tests are noted as crosses. In order to enhance the trend, the data has been smoothed, resulting in the thicker, dash-dotted, black line. This profile was used for comparison with the parameters that were found interesting, based on SGI's relation for determination of the cone factor in clay, OCR and w_L. Furthermore, K_0 and l_c were also tested as they shared similar trends with OCR and w_L.

Both linear and nonlinear least square fits were employed for the different parameters and different combinations of them. The solution that presented the least error, when compared to the evaluated N_{kt}-profile, is based solely on the overconsolidation ratio:

$$N_{kt} = 8.14 + \left(\frac{OCR}{7.65}\right)^{1.14} \tag{5.2}$$

The top portion of the Figure 5.8 consists of a box and whiskers plot that presents the same statistic data as in Figure 5.6. However, in this plot the deviation between the evaluated N_{kt}-profile and the different possible cone factors are presented. The mean deviation for $N_{kt,mean}$ and $N_{kt,OCR}$ are close to 0, whereas a cone factor of 11 is almost 20 % higher than the measured value. However, the standard deviation is significant and the results are to be interpreted cautiously.

6 Analysis

To compare the data obtained by the CPT to the undrained shear strength measured by field vane it is necessary to filter the CPT-data. Without smoothing of the cone resistance data, the scatter, and also the amount of data points that statistically are considered outliers increases drastically. The used filter, which essentially is the middle ground between a floating mean value and the smoothing criterion presented by Jacobsen, showed promising results as it efficiently dealt with both local spikes caused by single coarse particles and the wider deviations corresponding to thicker lenses of friction soil. However, in order to enhance the comparison to the field vane, it was necessary to manually tone down deviations of large width and magnitude when it was obvious that the field vane had missed the part of the soil volume that caused the anomaly. The general trend of the filtering process is that it lowers the measured strength parameters, which in turn induces a degree of safety to the proceeding evaluation, previously to the application of a cone factor.

The different types of clay till analyzed in this project display a great variation in undrained shear strength, and also in grain size distribution. Furthermore, the degree of consolidation varies a lot between the different soils. Considering this, the mean cone factor, evaluated from field vane tests is in the range of 9.3 to 10.2 for the investigated types of clay till. As seen in Figure 5.6 the values are connected to a considerable standard deviation, in the range of 0.7 to 1.7. Furthermore, from the available statistical data it is not possible to either confirm or rule out that a single cone factor can be used for all of the different clay tills.

Traditionally a static approach is used when determining the cone factor for clay till, either 10 or 11 is used for the whole soil deposit. However, based on the investigation at Tornhill, there seems to be a trend where the cone factor does vary, as seen in Figure 5.8. In turn, this suggests that it could be beneficial to use a dynamic approach where the cone factor is set to different values for adequately chosen depth increments. The trend at Tornhill, when smoothed, shares similar traits to the distribution of OCR, K_0, w_L, and also l_c to some extent. From a wide selection of least square fits, it was possible to determine that a relation, which bases the cone factor solely on the overconsolidation ratio yielded the least error. Furthermore, this relation presents a depth-profile that is similar to a semi-dynamic approach that uses the average cone factor for the different layers of clay till. However, in order to verify the relation between the cone factor and the overconsolidation ratio more data from various locations is needed. The solutions featuring both OCR, and w_L or K_0 also seemed promising, and might have more flexibility than a relation solely based on the OCR, if data from different sites are compared.

Whether a static or dynamic approach to the cone factor is utilized, the problem with large standard deviations remains. The deviation is to some extent to be expected when investigating a non homogeneous material such as clay till. This is, however, magnified by using field vane tests for determining the undrained shear strength. The vane tests are known to provide very scattered data when used for testing in clay till. Tests at the same depth just a few meters apart can give values that differ from tens to hundreds of kPa. In order to handle this problem, it has been previously suggested to conduct more field vane tests and, by doing so, acquiring plausible values of the undrained shear strength by averaging. An alternative is to carry out more sophisticated tests, for example triaxial compression tests, on samples obtained from representative depths as a mean to support field tests.

The triaxial tests results analyzed in this project show values of undrained shear strength of a different magnitude than those obtained from the field vane tests or from the CPT-data, with a sensible cone factor applied. An explanation to this could be that the in-situ stress situation has not been properly recreated in the laboratory, more specifically the confining cell pressure in the triaxial tests has been lower than the in-situ lateral soil pressure.

From the extensive data set available for the different types of clay till at Tornhill it would seem plausible that the undrained shear strength is related to the preconsolidation stress, alternatively the overconsolidation ratio, and the lateral stress, alternatively the coefficient of earth pressure at rest. The horizontal stress is in turn directly connected to the preconsolidation stress via the Poisson's ratio, as the horizontal stress represents the prevented lateral displacement of the soil during the consolidation phase. This further emphasizes the importance of applying radial pressure of the correct magnitude in the triaxial tests, in order to obtain good measurements of the strength parameters.

Furthermore, at the same level of preconsolidation stress the Baltic clay till, which is rich in clay, experiences a significantly higher horizontal stress than the clay till denoted as mixed. This could indicate a relation between the horizontal stress and the clay content of the soil, that in turn is also related to the liquid limit, which can be summarized as:

$$\begin{cases} \tau_{fu} = f(\sigma'_c,\ \sigma'_h,\ \sigma'_{v0}) \\ \\ \sigma'_h = f(\sigma'_c,\ l_c) \\ \\ \sigma'_h = f(\sigma'_c,\ w_L) \end{cases} \tag{6.1}$$

Finally, the empirical relations introduced earlier in the report for evaluating the key parameters undrained shear strength and the preconsolidation stress yielded poor results. The general trend is that the relations underestimate the parameters when compared to field tests. Furthermore, the empirically evaluated trend is sometimes not even correct, when compared to directly measured data. A stronger empirical foundation could have been used for guidance when evaluating the scattered field test data, thus improving the quality of the chosen value. It would also enable the person who evaluates the measurements to take a less defensive and more realistic stance.

7 Conclusions

The comparison between the undrained shear strength measurements from field vane tests and the filtered net cone resistance presents a mean value of N_{kt} in the range of 9.3 to 10.2. Due to the significant degree of scatter in the data, partially caused by the inhomogeneous nature of the soil and partially influenced by primarily using field vane tests to evaluate the undrained shear strength, the mean values are connected to a significant standard deviation, ranging from 0.7 to 1.7. Despite the relatively high standard deviations, it still seems reasonable to use a cone factor lower than 11, especially when filtering the CPT-data repeating the method used in this project.

Furthermore, it also appears that the cone factor is closely linked to the overconsolidation ratio and a dynamic approach to evaluate the cone factor featuring OCR seems reasonable. Moreover, K_0 and w_L also seem connected to the variations in the cone factor.

Finally, the empirical relations available for evaluation of key parameters when investigating clay till are insufficient. However, it seems conceivable to empirically relate the undrained shear strength of clay till to the preconsoidation stress in combination with either the lateral stress or the liquid limit. Using sensible cone factors also gives good agreement between the net cone resistance and the undrained shear strength.

8 Proposed Further Research

CPT-sounding

In the current project, as well as in previous investigations, it has been concluded that, in a profile entirely comprised of clay till, CPT-sounding is feasible only down to a depth of about 7 meters. A stop is reached about a meter into the bottom layer of Northeast clay till. However, it has been noted that this stop is often not related to the probe hitting a larger particle like a stone but instead that the friction against the drill rods become to great. This problem could possibly be mitigated by using a set of sleeves, welded onto the drill rods, that widens the hole above the CPT-probe slightly, and decreases the contact area between the surrounding soil and the rods to a great deal.

Another approach could be to initially proceed as usual and use the CPT to the point where it gets stuck, at this point the CPT-probe and the drill rods are withdrawn and an auger is used to widen the hole down to about a meter above where the stop was reached. From this point it might be possible to continue the CPT-sounding further into the soil profile.

Sampling

The quality of the specimens are absolutely crucial when performing advanced laboratory tests. The methods used to obtain undisturbed samples includes various types of tube samplers, for example "Americanerrör", triple tube core drilling systems, for example "Geobor S" and regular double tube push samplers like the piston sampler, St II. However, all these methods present different problems. The standard piston sampler can only be used in softer clay till and/or clay till with small amounts of coarse particles, it also suffers from having a small sampling dimension. The tube samplers can be used in all types of clay till. The great advantage is the expendable nature of the sampling tubes. When utilized, the sampling tube is hammered into the ground, which means that it can be driven further down into firm clay till, compared to a push sampler. This, however, may have some compacting and disturbing effects on the soil, especially in the outer rim of the sample. It has been shown that the core drilling method is most suitable for firm clay tills with few lenses of coarse soil. In softer clay tills larger particles are not drilled through but instead pushed in front of the sampler. The flushing medium used to remove drill cuttings have a tendency to disturb the outer parts of the sample, as well as washing away lenses of friction soil.

Another sampler that has not been tested to a degree worth mentioning is the moraine sampler, "Mullvaden". It is a double tube sampler with a ring shaped drill bit that utilizes pushing and rotating motion just like the core drilling system. The big difference from the core drilling system is that the moraine sampler is installed on a regular drill rig, whereas the triple tube coring system requires a separate, much bigger, rig. Furthermore, testing a double or triple tube hollow stem auger sampler could yield good results as relatively big dimensions are available.

The proposed method is to compare undisturbed specimens taken with different samplers, exploiting the fact that the breaking point between the recompression and virgin loading curve, in an oedometer test, becomes more diffuse as the sample quality decreases. It is also possible to evaluate the sample quality based on the water content and volumetric strain during reconsolidation.

Advanced laboratory testing

Strength parameters evaluated from field vane tests are connected to significant uncertainty. One approach to tackle this and to reinforce the evaluated strength profile from the vane tests is to consider more points for field vane tests and simply increase the amount of tests and improve the evaluated parameters by averaging. On the other hand it is possible to employ advanced lab tests, essentially presenting the opportunity to replace quantity with quality. For instance, from the advanced tests a wider array of soil parameters can be evaluated, and in turn be utilized in sophisticated numerical calculations. Another benefit of evaluating soil parameters exploiting advanced tests is to avoid extensive usage of empirical relations in the attempt to evaluate too many parameters from one test.

In order to conduct these tests it is of utmost importance to have reliable measurements of the in-situ stress and preconsolidation condition, and samples of as high quality as possible.

Evaluation of the undrained shear strength of clay till

By conducting further comprehensive investigations of clay till at different locations it might be possible to derive a set of empirical relations that are based on more sophisticated tests, and the parameters that seem to influence the soils behaviour the most. This could then possibly be linked to the more affordable tests that are commonly used in regular geotechnical investigations.

Based on which parameters are most likely to influence the properties, and specifically the undrained shear strength, of clay till it is possible to design a more suitable test program that can be used in further investigations. This should involve a mean of determining the lateral stress condition, the preconsolidation stress, the composition of the soil material, and, also, a reliable way of directly measuring the undrained shear strength.

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