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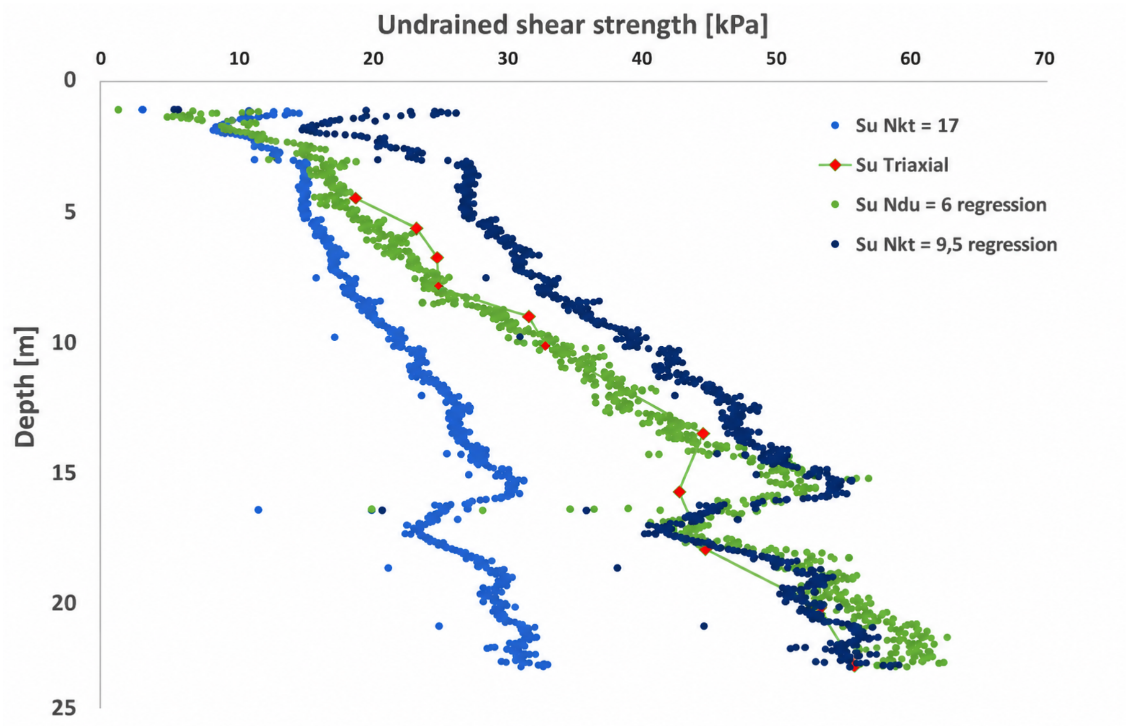

Evaluation of Cone Factors For Two Sensitive Clays From The West Coast of Sweden

Master's thesis in Infrastructure and Environmental Engineering

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CHALMERS UNIVERSITY OF TECHNOLOGY
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Abstract

Data from cone penetration tests with pore pressure measurement, CPTu, is commonly used in geotechnical engineering for evaluating undrained shear strength, s_u, in clay deposits. However, s_u cannot be obtained directly from CPTu soundings, but is instead estimated using empirical relationships such as the cone factors N_{kt} and $N_{\Delta u}$. The latter relate the normalized penetration resistance or the normalized excess pore pressures from CPTu to the emerging undrained shear strength. These factors may vary with soil conditions, depth, and site-specific characteristics.

The aim of this study was to evaluate the relationship between CPTu data and undrained shear strength obtained from triaxial tests, using data from the Kärra and Utby test sites. Furthermore, the study investigated the feasibility of using N_{kt} and $N_{\Delta u}$.

The results from this study indicate that the excess pore pressure, Δu, showed a stronger correlation with the undrained shear strength evaluated from anisotropically consolidated triaxial tests sheared in compression (CAUC), than the net cone resistance, q_{net}. The results also indicate that cone factors should not be treated as constant values for specific soil types, but instead require site-specific calibration. The study highlights the importance of combining CPTu measurements with laboratory testing and soil classification when interpreting undrained shear strength in clay.

Keywords: Geotechnic, Undrained shear strength s_u, CPTu, Triaxial

Acknowledgements

I would like to sincerely thank Professor Jelke Dijkstra for his dedication and continuous support throughout this work. Special thanks are also extended to my supervisor Per Nylander at SWECO for his invaluable guidance and commitment during the course of this study. This thesis would not have been possible without their expertise.
During the writing process, AI-based language assistance was used to improve grammar, clarity, and flow.

Davood Jafari, Gothenburg, June 2026

List of Acronyms

Below is the list of acronyms that have been used throughout this thesis:

ASTM	American Society for Testing and Materials
BH	Borehole
CAUC	Consolidated Anisotropically Undrained Compression
CAUE	Consolidated Anisotropically Undrained Extention
CPT	Cone Penetration Test
CPTu	Cone Penetration Test with pore pressure measurement / Piezocone test
DSS	Direct Simple Shear
ISO	International Organization for Standardization
OCR	Overconsolidation Ratio
SBT	Soil behavior type
SCPT	Seismic Cone Penetration Test
SCPTu	Seismic Cone Penetration Test with pore pressure measurement
SGI	Swedish Geotechnical Institute
TC	Triaxial Compression
TE	Triaxial Extension

Nomenclature

The list below is the parameters, and variables that have been used throughout this thesis.

A_c	Projected area of the cone
A_s	Surface area of the friction sleeve
A_{sb}	Cross-sectional area at the bottom of the sleeve
A_{st}	Cross-sectional area at the top of the sleeve
a	Net area ratio of the cone
B_q	Pore pressure parameter
c_h	Coefficient of horizontal consolidation
D_r	Relative density
E	Young's modulus
f_s	Measured sleeve friction
f_t	Corrected sleeve friction
F_r	Normalized friction ratio
F_s	Total force acting on the friction sleeve
G	Shear modulus
G_0	Small-strain shear modulus
G_s	Specific gravity of soil particles
k	Permeability / hydraulic conductivity
K_0	In-situ earth pressure coefficient
M	One-dimensional constrained modulus
N_{kt}	Cone factor for cone-resistance-based evaluation of s_u
$N_{\Delta u}$	Pore pressure factor for evaluation of s_u
p_a	Atmospheric pressure
q_c	Measured cone resistance
q_t	Corrected cone resistance
q_{net}	Net cone resistance, $q_t - \sigma_{v0}$
Q_c	Total force acting on the cone tip
R_f	Friction ratio
s_u	Undrained shear strength
s_{uTC}	Undrained shear strength from triaxial compression
s_{uSS}	Undrained shear strength from simple shear
s_{uTE}	Undrained shear strength from triaxial extension
$s_{u(ave)}$	Average undrained shear strength
S_t	Soil sensitivity
t_{50}	Time required for 50% pore pressure dissipation
u	Measured pore water pressure
u_0	Equilibrium pore water pressure / in-situ pore pressure
u_1	Pore pressure measured on the cone face
u_2	Pore pressure measured just behind the cone
u_3	Pore pressure measured behind the friction sleeve
V_p	Compression wave velocity
V_s	Shear wave velocity
w_L	Liquid limit
Δu	Excess pore pressure, $u_2 - u_0$
γ	Total unit weight of soil
γ_w	Unit weight of water
ϕ'	Peak effective friction angle
Ψ	State parameter
σ_{v0}	Total vertical overburden stress
σ'_{v0}	Effective vertical overburden stress

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1 Introduction

This chapter briefly introduces the background of this thesis, and presents the aim, objectives and limitations of the study.

1.1 Background

Geotechnical design in clay is strongly dependent on a reliable evaluation of undrained shear strength s_u. Cone penetration testing with pore pressure measurement (CPTu) is a versatile method and is commonly used for this purpose because it provides continuous measurements with depth and can also provide data for the estimation of soil stratigraphy, groundwater conditions and other geotechnical properties [1, 3]. However, the undrained shear strength cannot be measured directly from CPTu, and must therefore be interpreted through empirical or semi-empirical correlations.

A common approach is to estimate undrained shear strength from CPTu using cone factors such as N_{kt} and $N_{\Delta u}$. The cone factor N_{kt} relates the net cone resistance q_{net} to undrained shear strength, while the pore pressure factor $N_{\Delta u}$ relates the excess pore pressure Δu to undrained shear strength [4, 2]. Since these factors are empirical, they should preferably be calibrated against independent reference data from laboratory or field tests [4, 2, 5].

High-quality laboratory tests, such as triaxial tests, can provide reference values of undrained shear strength at selected depths. By comparing the laboratory-determined shear strength with CPTu parameters from corresponding soil levels, site-specific values of N_{kt} and $N_{\Delta u}$ can be back-calculated [4, 2]. This type of calibration is important because CPTu correlations may vary between sites and soil deposits, and using general cone factor values may lead to uncertainty in the estimated shear strength.

Several CPTu studies have used laboratory-determined undrained shear strength as reference values when evaluating CPTu correlations [2, 5]. In this type of analysis, the CPTu parameters are matched with laboratory samples from corresponding depths, and the relationship between s_u, q_{net} and Δu is evaluated. This provides a basis for comparing whether the traditional cone resistance-based method using N_{kt} or the pore pressure-based method using $N_{\Delta u}$ gives a more representative estimate of undrained shear strength for the investigated site.

In this thesis, the calibration is based mainly on anisotropically consolidated triaxial tests sheared in compression (CAUC). The triaxial results are used as reference values, while CPTu data from corresponding depth intervals are used to calculate average values of net cone resistance and excess pore pressure. These values are then used to back-calculate N_{kt} and $N_{\Delta u}$ and to evaluate how well each factor represents the laboratory-determined shear strength.

1.2 Aims and objectives

This report focuses on a comparative analysis of CPTu properties and the correlation between undrained shear strength from triaxial tests, the traditional cone factor N_{kt} and the pore pressure factor $N_{\Delta u}$. The study aims to highlight how the choice of cone factor influences the estimated undrained shear strength, and to evaluate whether calibration against triaxial data can provide more representative site-specific cone factors.

The objectives of this study are:

1. Determine undrained shear strength s_u from available triaxial test data.

2. Match CPTu parameters with corresponding triaxial sample for further analysis.

3. Calculate the cone factor N_{kt}, and the pore pressure factor $N_{\Delta u}$ from CPTu and triaxial-based undrained shear strength.

4. Compare the performance of N_{kt} and $N_{\Delta u}$ in estimating undrained shear strength for the investigated sites.

5. Evaluate how pore pressure response Δu and cone resistance q_{net} correlated to the estimated undrained shear strength s_u

1.3 Limitations

Due to time constraints and limited available data, this thesis focuses on a comparative analysis of cone resistance and pore pressure response with undrained shear strength based on available triaxial tests and CPTu soundings from the Kärra and Utby test sites. The analysis is therefore based on a limited number of laboratory reference values, which means that the results should be interpreted as site-specific rather than as general correlations for all clay deposits.

Another limitation is that the triaxial samples and CPTu measurements do not represent exactly the same soil volume. In addition, the CPTu soundings extend to greater depths than the available triaxial test data. Since the calibration of N_{kt} and $N_{\Delta u}$ requires laboratory-determined reference values of s_u, the analysis was limited to the depth range where triaxial data were available. The CPTu data were therefore averaged around the corresponding laboratory sample levels to reduce the influence of local variation, but some uncertainty remains due to natural soil variability, layer boundaries and measurement uncertainty.

Furthermore, the study does not include a detailed evaluation of all factors that may influence cone factors, such as soil plasticity, anisotropy, strain-rate effects or full stress history. These factors may affect CPTu interpretation, but they are outside the scope of this thesis because the available analysis is mainly based on calibration against triaxial shear strength. Therefore, the results should primarily be used as a site-specific comparison between N_{kt} and $N_{\Delta u}$ for the investigated data set.

Another limitation is that the study focuses on empirical back-calculation of cone factors rather than developing a general predictive model. The results may therefore be useful for understanding the relationship between CPTu parameters and triaxial-based undrained shear strength at the investigated sites, but generalization to other sites should only be considered when similar soil conditions, testing procedures and reference data are available.

1.4 Societal, ethical and ecological impacts

Reliable estimation of undrained shear strength in clay is directly relevant to the safety of geotechnical constructions such as embankments, foundations, and infrastructure. Sensitive clays are common in Scandinavia and can lose a large part of their strength when they are remoulded. This means that slopes in such clay deposits can be more vulnerable to rapid and retrogressive landslides, which may affect large areas and cause damage to infrastructure and nearby communities [6, 7]. The Göta river valley is also known as one of the more landslide-prone areas in Sweden, and several areas along the river have been identified with elevated landslide risk [8].

From a societal perspective, improved methods for site-specific calibration of CPTu-based cone factors can contribute to safer and more reliable geotechnical assessments. This is important because undrained shear strength is used in stability evaluations for embankments, foundations, slopes and infrastructure. If the shear strength is overestimated, the design may become unsafe because the real stability is lower than assumed. If the shear strength is underestimated, the design may become unnecessarily conservative, which can increase construction costs and material use. A recent study on Norwegian clay deposits indicates that locally calibrated cone factors can differ significantly from general recommended values, which highlights that using a general value without calibration may misrepresent the actual undrained shear strength at a given site [2].

The ethical aspect is mainly connected to responsibility in interpretation and design. CPTu correlations are empirical and should not be used without considering local soil conditions and available reference data. As previous studies indicate, CPTu correlations are site-dependent and empirical cone factor values derived from one geological context should not be uncritically applied to another without validation against local reference data [2, 5].

Ecologically, CPTu can contribute to a more efficient ground investigation because it provides continuous in-situ data with depth and may reduce the need for extensive sampling and additional boreholes. Reliable CPTu interpretation, supported by site-specific calibration, may help limit unnecessary field investigations, reduce material handling and decrease disturbance at the investigation site.

2
Theory

This chapter presents the fundamental principles behind the cone penetration test combined with pore pressure measurement, commonly referred to as CPTu. The aim is to provide readers with sufficient understanding of how the method is performed and how the recorded parameters can be interpreted. The general description of CPT and CPTu in this chapter is primarily based on *Guide to Cone Penetration Testing* by Robertson and Cabal, *Cone Penetration Testing in Geotechnical Practice* by Lunne, Robertson and Powell, and SGI Information 15 by Larsson [1, 9, 3].

2.1 General overview of the Cone Penetration Test (CPT)

The cone penetration test (CPT) is an in-situ geotechnical method used to investigate subsurface conditions and identify soil stratification [1, 3]. Several CPT variants exist, but this study focuses on the electric cone with pore pressure measurement, known as the piezocone or CPTu. Geotechnical engineers utilize CPTu data primarily used to identify the soil strata, assess ground water condition and obtain parameters for geotechnical design.

During a CPTu sounding, the cone probe is advanced into the ground at a controlled penetration rate, typically 20 mm/s [1, 3]. As penetration proceeds, the system continuously, or nearly continuously, records cone resistance, sleeve friction, and pore water pressure with depth. Since the introduction of the electric cone, various additional sensors have been incorporated into CPT probes, as summarized in Table 2.1.

Table 2.1: Examples of additional sensors for CPTu probes (Modified from [1])

Sensors		
Temperature	Camera	pH
Geophones/accelerometers	Radioisotope	Redox
Pressuremeter	Electrical resistivity/conductivity	LIF/UVOST
Dielectric	Membrane interface probe	

Despite these additions, three primary parameters remain central to CPTu: cone resistance q_c, sleeve friction f_s, and pore water pressure u [1, 10]. The cone resistance is obtained by dividing the axial force at the cone tip, Q_c, by the projected area of the cone, A_c:

$$q_c = \frac{Q_c}{A_c}$$

The sleeve friction is derived by normalizing the measured force on the friction sleeve, F_s, by the sleeve surface area, A_s:

$$f_s = \frac{F_s}{A_s}$$

Pore water pressure can be measured at various positions on the cone [1, 10]. The most common location is u_2, immediately behind the cone, because this measurement is required for correcting the cone resistance. Alternative positions include u_1 on the cone face and u_3 behind the friction sleeve. Figure 2.1 provides a schematic illustration of the different sensor locations.

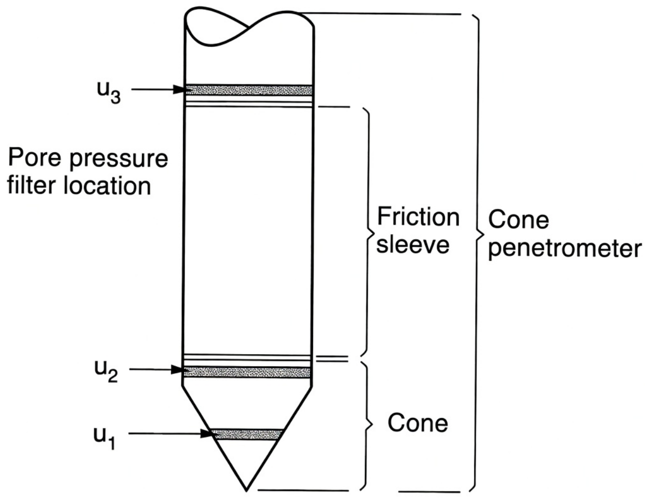

Figure 2.1: Illustration of a piezocone with terminology (Modified from [1])

2.2 Historical development

The earliest CPT equipment originated in the Netherlands [1]. The initial mechanical design featured an outer pipe with a diameter of 35 mm and an inner rod with a diameter of 15 mm. The cone had a projected area of 10 cm^2 and an apex angle of 60°. In 1935, Delft Soil Mechanics developed a manually operated pushing machine with a capacity of approximately 100 kN.

The addition of a friction sleeve in 1953 enabled a second measurement to be recorded during sounding [1, 9]. By comparing sleeve friction with cone resistance, the friction ratio could be used as an indicator of soil type during interpretation.

The first CPT-based soil classification was developed for the mechanical Begemann cone, as shown in Figure 2.2.

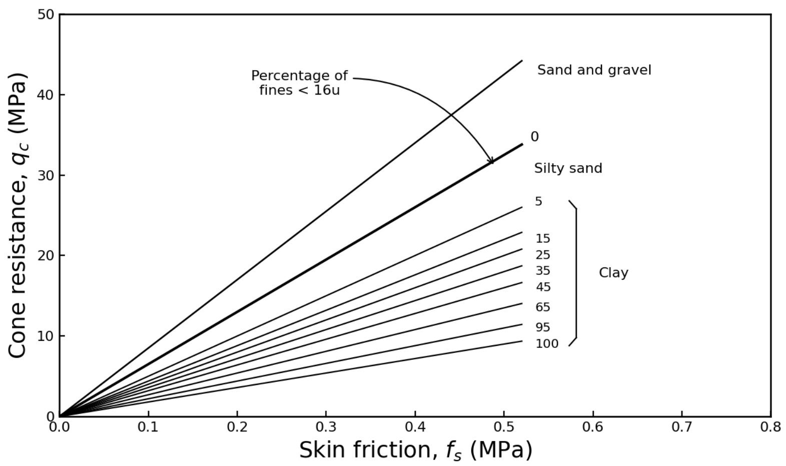

Figure 2.2: The first CPT-based soil classification system (Modified from [1])

The introduction of electric cones in the 1960s brought significant improvements [1]. Compared to mechanical cones, electric versions reduced errors caused by rod friction and enabled continuous data recording during penetration. In 1965, Fugro introduced an electric CPT penetrometer that became the basis for modern cone designs and later influenced international standards and ASTM procedures.

Pore water pressure measurement was added to CPT in 1974, giving rise to the CPTu or piezocone test [1, 10]. Early piezocone designs used various filter positions, but the u_2 position behind the cone eventually became standard because it is necessary for correcting the measured cone resistance.

2.3 Testing procedures, equipment, and maintenance

CPT penetrometers are available in various sizes depending on the application [1]. Smaller cones, such as 2 cm^2, are suitable for shallow investigations, while larger probes, such as 40 cm^2, can be used in coarser materials or for deeper soundings. Figure 2.3 shows a scaled comparison of different probe sizes.

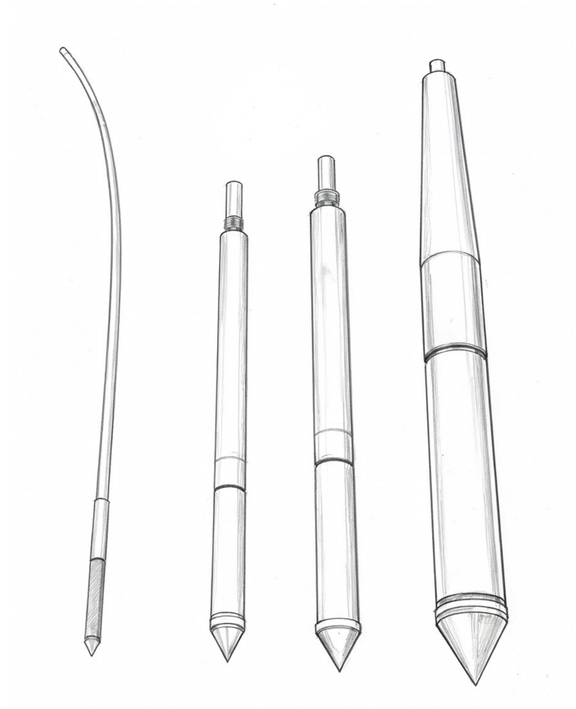

Figure 2.3: CPT probes of different sizes, from left: 2 cm^2, 10 cm^2, 15 cm^2, and 40 cm^2 (Modified from [1])

CPT testing can be performed both onshore and offshore [1, 3]. Penetration becomes limited if the ground contains hard materials such as gravel, stones, or rock. In soft soils, significant depths can be reached with appropriate pushing equipment. Rod friction can be reduced to improve penetration depth. CPT can also be integrated with sonic drill rigs, where vibrations help pass through hard layers.

2.3.1 Pre-drilling

When hard soil layers or coarse-grained materials are encountered, pre-drilling is recommended in order to avoid damaging the CPT equipment [1]. This can be done using a solid steel probe with a slightly larger diameter than the CPT cone. In urban areas, an auger is often used in the upper part of the soil profile to avoid underground utilities.

2.3.2 Verticality

The inclination of the probe during testing affects both depth measurement and data quality [10]. The pushing machine must therefore be positioned vertically, and the push rods should be checked for straightness. Modern CPT cones often include inclination sensors to monitor the verticality of the sounding[1].

2.3.3 Reference measurements

To achieve high accuracy, zero-load readings should be recorded before and after each sounding [1]. This allows any drift caused by temperature changes or other factors to be tracked and accounted for.

2.3.4 Penetration rate

The usual standard penetration rate is 20 mm/s [1, 3]. Penetration occurs under drained conditions in coarse-grained soils, such as sand, undrained conditions in fine-grained soils, such as clay, and partially drained conditions in silt.

2.3.5 Reading interval

Electric cone probes generate continuous analog data, but these are typically converted to digital values at selected intervals [1]. Most systems can record data at intervals between 10 and 50 mm, with 20 mm being the most common.

2.3.6 Dissipation test

During a dissipation test, penetration is halted and the decay of excess pore pressure is monitored over time [1, 3]. The dissipation rate is influenced by the probe diameter and the soil's coefficient of consolidation, which depends on permeability and compressibility. A commonly used parameter is t_{50}, defined as the time required for 50% dissipation of the excess pore pressure. To determine the equilibrium pore pressure u_0, the dissipation test must continue until no further significant dissipation occurs, as illustrated in Figure 2.4.

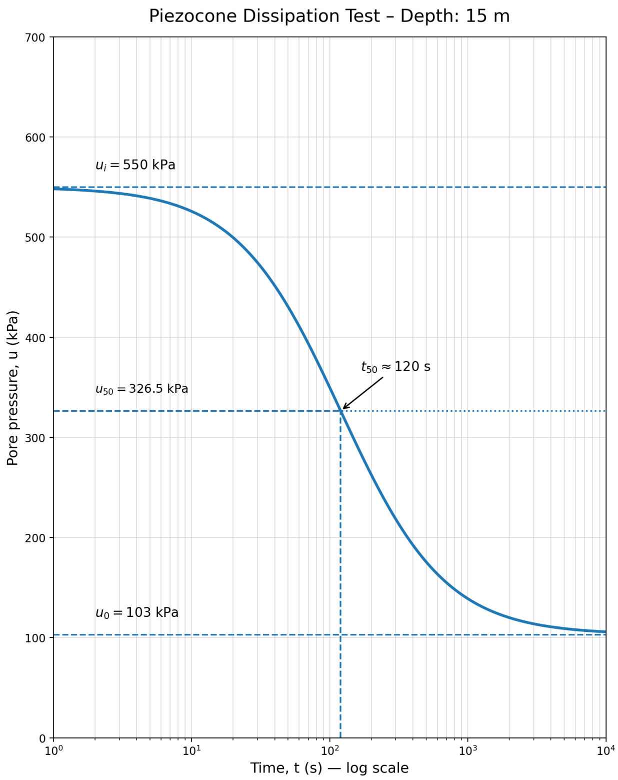

Figure 2.4: Schematic of a dissipation test

2.3.7 Maintenance and calibration

To ensure reliable measurements, calibration should be performed at intervals based on the stability of zero-load readings [1, 10]. If zero-load drift becomes unstable, calibration may be necessary. Table 2.2 summarizes recommended check and calibration intervals.

Table 2.2: Calibration and maintenance intervals for CPT (Modified from [1])

Maintenance item	Project start	Test start	Test end	End of day	Monthly	Every 3 months
Wear check	✓	✓			✓	
O-ring seals	✓			✓		
Push rods		✓			✓	
Pore pressure filter	✓	✓				
Calibration						✓
Computer					✓	
Cone					✓	
Zero-load reading		✓	✓			
Cables	✓				✓	

2.3.8 CPT penetrometer design

The way cone resistance and sleeve friction are measured depends on the internal configuration of the CPT probe [1, 10]. Two main approaches exist: probes that use dedicated load cells for each measurement, and those that derive sleeve friction indirectly by computing the difference between two force readings, commonly referred to as subtraction-type probes. Figure 2.5 illustrates three different design principles for CPT probes.

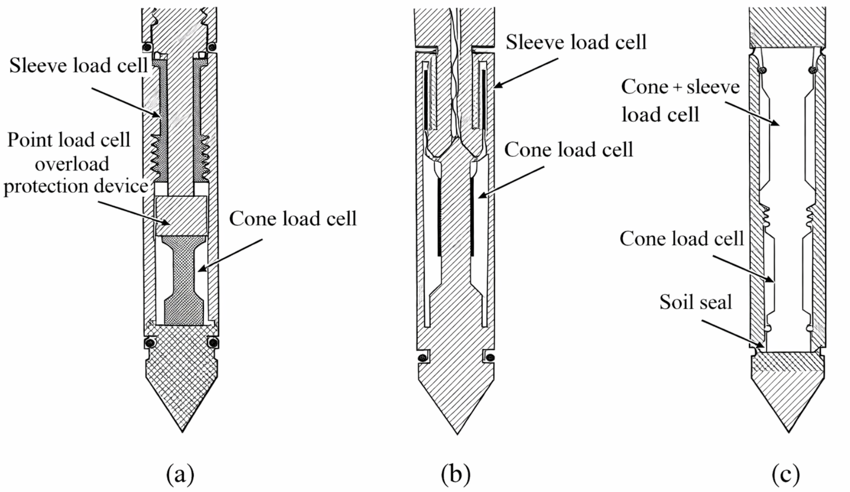

Figure 2.5: Different design principles for CPT probes (Modified from [1])

Subtraction-type probes gained widespread adoption during the 1980s due to their mechanical durability under field conditions [1, 10]. A known drawback of this

design, however, is reduced sleeve friction accuracy — particularly in soft soils where friction values are small. Even minor zero-load offsets in the load cells can introduce significant relative errors in the computed sleeve resistance, making this a critical consideration in soft ground investigations.
For projects requiring reliable sleeve friction data — especially in soft or sensitive soils — probes with independent load cells for cone and sleeve are therefore preferred [10]. Regardless of probe type, measurement quality relies heavily on systematic quality control: verifying zero-load readings, checking dimensional tolerances, and maintaining appropriate sleeve surface roughness [1, 10]. It should be noted that sleeve friction f_s is inherently less precise than cone resistance q_c, and this discrepancy becomes particularly pronounced in soft soils where measured friction values approach the resolution limits of the instrumentation [10].

2.4 Effects of pore water pressure

As mentioned earlier, the pore pressure sensor can be placed at several locations. The u_2 position behind the cone is the most common [1, 10]. Due to the cone's internal geometry, the measured cone resistance is affected by pore water pressure, requiring correction when pore pressures are significant. This is particularly important in soft fine-grained soils and in offshore or underwater investigations. The corrected cone resistance q_t is calculated as:

$$q_t = q_c + u_2(1 - a)$$

where a represents the net area ratio determined from laboratory calibration [1]. The value of a typically ranges between 0.70 and 0.85. In sandy soils, the correction is often negligible because q_c is high and u_2 has less influence.
Pore pressure may also affect the measured sleeve friction if water pressure acts on the end areas of the sleeve [1]. The corrected sleeve friction can therefore be expressed as:

$$f_t = f_s - \frac{u_2 A_{sb} - u_3 A_{st}}{A_s}$$

where A_s is the sleeve surface area, and A_{sb} and A_{st} are the cross-sectional areas at the bottom and top of the sleeve, respectively. To minimize the need for correction, many standards require that penetrometers have equal end areas, $A_{st} = A_{sb}$. In practice, the difference between measured and corrected sleeve friction then becomes small.

2.5 Interpretation of CPTu data

CPTu interpretation relies largely on empirical and semi-empirical correlations [1, 9, 2]. Although many correlations have theoretical foundations, they typically require calibration or validation against field and laboratory data. Due to the complexity of soil behavior, these correlations should be applied with consideration of local conditions.

The CPT penetrometer can be equipped with additional sensors, such as pore pressure sensors (CPTu) or seismic sensors (SCPT), which influence which soil parameters can be estimated [1]. Table 2.3 presents an assessment of the CPTu method's suitability for estimating various soil parameters. A rating scale from 1, highest reliability, to 5, lowest reliability, is used. Adding a seismic sensor, SCPTu, can significantly improve the estimation of soil stiffness.

Table 2.3: Assessment of CPTu applicability for estimating different soil parameters (Modified from [1])

Soil type	$\mathbf{D_r}$	$\mathbf{\Psi}$	$\mathbf{K_0}$	**OCR**	$\mathbf{S_t}$	$\mathbf{s_u}$	ϕ'	**E, G**	**M**	$\mathbf{G_0}$	**k**	$\mathbf{c_h}$
Coarse-grained soil, sand-like	2–3	2–3	5	5	–	–	2–3	2–3	2–3	2–3	3–4	3–4
Fine-grained soil, clay-like	–	–	2	1	2	1–2	4	2–4	2–3	2–4	2–3	2–3

2.5.1 Groundwater assessment

Since in-situ effective stresses govern soil behavior, accurately assessing groundwater conditions is essential [1, 3]. The CPTu provides a detailed and continuous pore water pressure profile. Dissipation tests can be used to determine the equilibrium pore water pressure u_0, which helps define the piezometric profile. Groundwater conditions are often assumed to be hydrostatic, but factors such as ground slope or proximity to water bodies can alter this assumption.

2.5.2 Soil classification

Robertson's SBT framework classifies soils based on their in-situ response during cone penetration rather than directly from grain-size distribution [1, 9]. The classification uses normalized CPT parameters, primarily cone resistance, friction ratio, and, for CPTu, pore pressure response. The normalized cone resistance is given by:

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}}$$

The normalized friction ratio is calculated as:

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} \cdot 100$$

The pore pressure parameter is defined as:

$$B_q = \frac{\Delta u}{q_t - \sigma_{v0}} = \frac{u_2 - u_0}{q_t - \sigma_{v0}}$$

Additionally, the soil behavior type index I_c can be used to describe the continuous variation of soil behavior with depth [9, 1]:

$$I_c = \left[(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2\right]^{0.5}$$

The normalized cone resistance with stress exponent is defined as:

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{p_a}\right)\left(\frac{p_a}{\sigma'_{v0}}\right)^n$$

where p_a is the atmospheric reference pressure, commonly taken as 100 kPa, and n is the stress exponent. For clay-like soil behavior, n can often be taken as 1.

2.5.3 Unit weight (γ)

Laboratory testing on samples of known volume provides the most direct determination of total unit weight [1]. When such data are unavailable, CPT-based empirical correlations can be used. Robertson and Cabal present the following semi-empirical relationship:

$$\frac{\gamma}{\gamma_w} = \left(0.27 \log R_f + 0.36 \log \frac{q_t}{p_a} + 1.236\right)\frac{G_s}{2.65}$$

where R_f is the friction ratio, γ_w is the unit weight of water, p_a is atmospheric pressure, and G_s is the specific gravity of the soil particles.

2.5.4 Undrained shear strength (s_u)

Undrained shear strength s_u cannot be regarded as a unique material parameter with a single constant value [1, 4]. The undrained response of soil depends on several factors, including loading direction, anisotropy, strain rate, and stress history. A common CPTu-based interpretation of undrained shear strength uses the net cone resistance divided by an empirical cone factor:

$$s_u = \frac{q_t - \sigma_{v0}}{N_{kt}} = \frac{q_{net}}{N_{kt}}$$

Reported N_{kt} values vary considerably, typically ranging from 10 to 18 [1, 4]. For Swedish cohesive soils, SGI Information 3 presents an empirical relationship where the cone factor is related to the liquid limit [11]. In very soft clay where cone resistance measurements may be uncertain, the undrained shear strength can instead be estimated from the excess pore water pressure [1, 11]:

$$s_u = \frac{u_2 - u_0}{N_{\Delta u}} = \frac{\Delta u}{N_{\Delta u}}$$

where $N_{\Delta u}$ typically varies between 2 and 10 [1].

3

Method

3.1 Introduction

This chapter describes the methodology that has been used to estimate the cone factor N_{kt} and the excess pore pressure factor $N_{\Delta u}$, and how these parameters are correlated to the undrained shear strength s_u. The method is based on empirical correlation between laboratory-determined shear strength values and CPTu field data. More specifically, the cone factors were back-calculated by comparing undrained shear strength obtained from triaxial tests with CPTu parameters from corresponding soil levels. Site-specific calibration against laboratory reference values is described as an important approach for evaluating cone factors when such data are available [4].

Previous CPTu studies have used laboratory-determined undrained shear strength together with CPTu parameters such as net cone resistance q_{net} and excess pore pressure Δu to evaluate correlations for clay [2, 5]. Therefore, the comparison between triaxial shear strength and CPTu data used in this study follows an established approach in geotechnical engineering.

3.2 Analysis setup

The analysis was carried out by first compiling the results from the triaxial tests. For each triaxial test from a specific depth, the undrained shear strength s_u was identified from the deviator stress at failure. The relationship used in the analysis is:

$$s_u = \frac{q_f}{2}$$

where:

- s_u = undrained shear strength
- q_f = deviator stress at failure

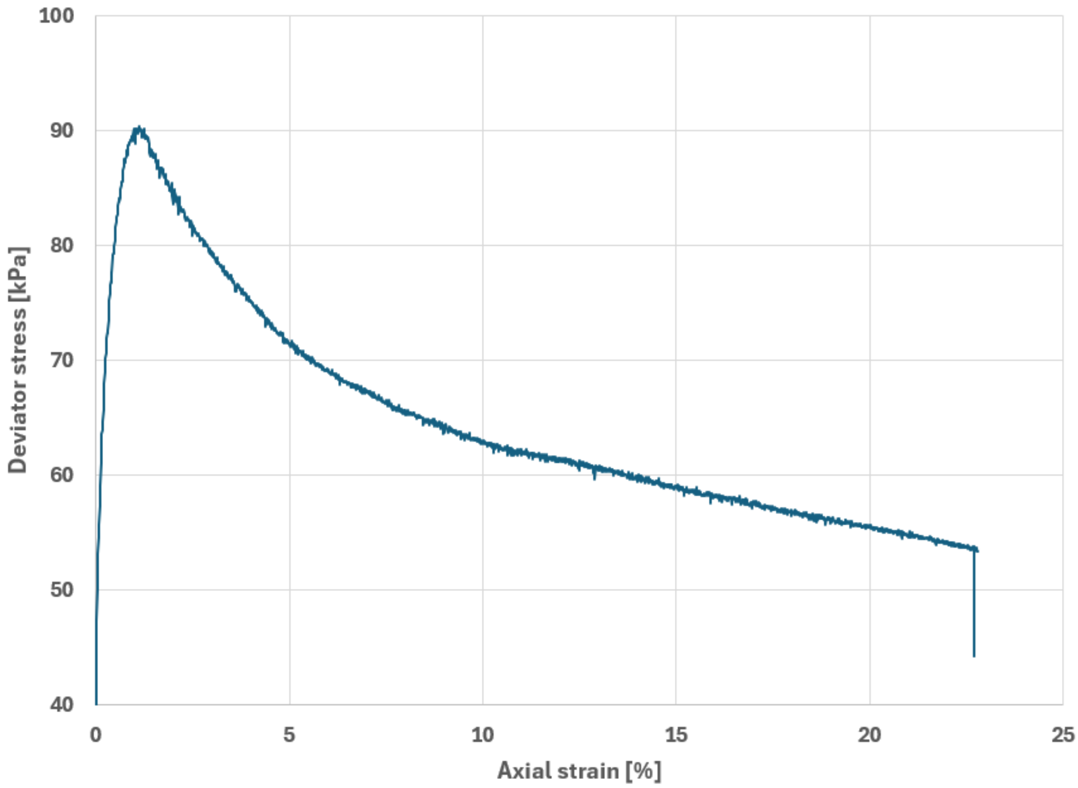

Figure 3.1: Typical result from CAUC test used for determination of undrained shear strength s_u. For this particular result on a sample from 16 m depth the undrained shear strength is estimated as 45.2 kPa.

Figure 3.1 shows a stress-strain curve where the deviator stress is measured as function of increasing axial strain. The undrained shear strength s_u was determined as half of the maximum deviator stress q_f, where the sample reaches failure. To avoid visual interpretation errors, the maximum deviator stress q_f was determined numerically using a spreadsheet. These maximum values were used as reference values for each corresponding depth. Subsequently, all shear strength values were compiled in a graph where undrained shear strength was presented as a function of depth in order to compare it with the calculated shear strength from CPTu.

The purpose of this initial step was to obtain a clear picture of how the measured shear strength varies throughout the soil profile. Since the triaxial tests provide direct laboratory values of undrained shear strength s_u, these values were used as the basis for back-calculating the cone factor N_{kt} and the excess pore pressure factor $N_{\Delta u}$. Laboratory-determined shear strength has also been used as reference data when evaluating CPTu correlations in previous studies on clay [2, 5].

For calculation of basic undrained shear strength s_u directly from CPTu without any laboratory data, the empirical relationship of cone factor N_{kt} is used. It is based on SGI information 3 that gives rough estimates of the cone factor to be 16.3 for clay when liquid limit or laboratory data is not available, but it is rounded up to be 17 in this study [11].

3.2.1 Processing of triaxial tests

For each triaxial test, the relationship between deviator stress and axial strain was studied. The length of the triaxial sample was assumed to be approximately 100 mm. Since each triaxial sample represents a specific sample length within the soil profile, the mid-level of the sample was used as the reference depth. For instance, a sample from 16 m depth was matched with the closest CPTu data points around the same level
:

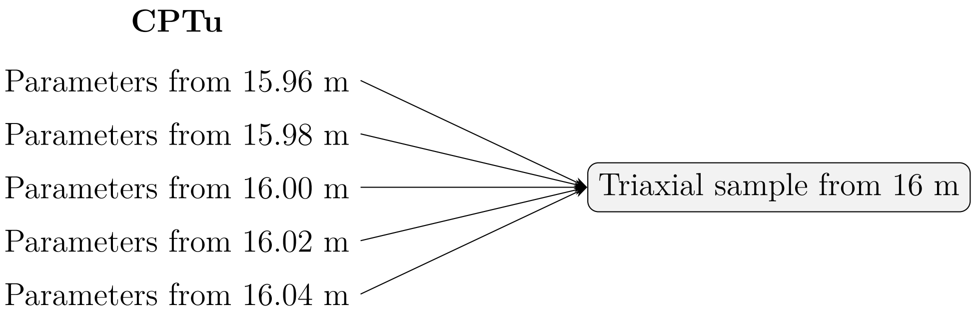

Figure 3.2: Matching procedure between CPTu data and triaxial data.

This method allows a direct comparison between laboratory and in-situ results from CPTu sounding. The triaxial test gives one value of undrained shear strength s_u, while the CPTu sounding provides several measurements within the same depth interval. Therefore, a mean value for 5 data points from the CPTu was calculated to match the obtained shear strength from the triaxial test. Averaging CPTu values around the relevant sample depth is supported by previous CPTu correlation studies where CPTu measurements were compared with reference values from corresponding soil levels [5].

Table 3.1: Typical CPTu parameters and calculated values for matching with triaxial data

Depth (m)	q_T **(MPa)**	f_T **(kPa)**	u_0 **(kPa)**	Δu **(kPa)**	**Unit weight (kN/m^3)**	σ_v **(kPa)**	σ'_v **(kPa)**	**OCR (-)**	**Su (kPa)**	B_q **(-)**	R_{ft} **(-)**	**B (m/s)**
15,96	0,64	5,19	149,60	277,60	15,50	244,74	95,14	1,30	23,29	0,70	0,81	17,00
15,98	0,65	5,28	149,80	280,70	15,50	245,05	95,25	1,30	23,56	0,70	0,82	22,00
16,00	0,65	5,35	150,00	287,80	15,50	245,36	95,36	1,30	23,58	0,72	0,83	22,00
16,02	0,65	5,43	150,20	293,70	15,50	245,67	95,47	1,30	23,99	0,72	0,83	21,00
16,04	0,66	5,32	150,40	296,80	15,50	245,98	95,58	1,30	24,14	0,72	0,81	22,00
Mean	**0,648462**			**287,32**		**245,36**			**23,71188**			

For each sample level, the average value of the net cone resistance was calculated as:

$$q_{net,ave} = \frac{1}{n} \sum_{i=1}^{n} (q_{t,i} - \sigma_{v0,i})$$

where:

- $q_{net,ave}$ = average net cone resistance

- $q_{t,i}$ = corrected cone resistance at point i
- $\sigma_{v0,i}$ = total vertical stress at point i
- n = number of CPTu points within the selected interval

Correspondingly, the average value of the excess pore pressure was calculated as:

$$\Delta u_{ave} = \frac{1}{n}\sum_{i=1}^{n}(u_{2,i} - u_{0,i})$$

where:

- Δu_{ave} = average excess pore water pressure
- $u_{2,i}$ = pore water pressure measured behind the cone tip at point i
- $u_{0,i}$ = initial in-situ pore water pressure at point i
- n = number of CPTu points within the selected interval

Using the average value over the sample interval reduces the influence of local variation in the CPTu sounding and makes the comparison with the triaxial sample more representative. This is important because the triaxial sample and CPTu sounding do not represent exactly the same soil volume.

3.2.2 Calculation of N_{kt}

The cone factor N_{kt} was calculated by relating the average net cone resistance from CPTu to the undrained shear strength from the triaxial test. The use of net cone resistance divided by undrained shear strength follows the common CPTu interpretation principle for evaluating N_{kt} [4, 2].

$$N_{kt} = \frac{q_{net,ave}}{s_u}$$

where:

- N_{kt} = back-calculated cone factor
- $q_{net,ave}$ = average net cone resistance from CPTu
- s_u = undrained shear strength from the triaxial test

This means that the laboratory-determined shear strength was used as a reference value, while the CPTu parameter $q_{net,ave}$ was used to back-calculate the corresponding cone factor for each depth and borehole.

3.2.3 Calculation of $N_{\Delta u}$

In addition, the pore pressure factor $N_{\Delta u}$ was also calculated. The pore pressure factor is commonly defined as the ratio between excess pore pressures measured at the u2 position of the CPTu and the undrained shear strength tested in the laboratory [2]. This parameter describes the relationship between the measured excess pore pressure during CPTu sounding and the undrained shear strength:

$$N_{\Delta u} = \frac{\Delta u_{ave}}{s_u}$$

where:

- $N_{\Delta u}$ = back-calculated pore pressure factor
- Δu_{ave} = average excess pore water pressure from CPTu
- s_u = undrained shear strength from the triaxial test

This factor was evaluated in addition to N_{kt} because excess pore pressure is an important CPTu parameter for the interpretation of undrained shear strength in clay. Previous CPTu studies have also included both net cone resistance and excess pore pressure when evaluating correlations with undrained shear strength [2].

3.2.4 Comparison between CPTu boreholes

Four CPTu boreholes from the Kärra site and one borehole from the Utby site were used in this analysis. The cone factor N_{kt} and the excess pore pressure factor $N_{\Delta u}$ were calculated separately for each borehole and sample level. This made it possible to study both the variation with depth and the differences between the CPTu boreholes at the same site.

For each level, the same laboratory-based shear strength s_u value was compared with CPTu data from each respective borehole. In this way, the analysis could show whether the CPTu boreholes yielded similar values of cone factor N_{kt} and excess pore pressure factor $N_{\Delta u}$, or whether there was a clear difference between the soundings. Previous CPTu studies have shown that correlations may include scatter and may be affected by site conditions and natural variability [2, 5]. Therefore, the boreholes were evaluated separately before general trends were interpreted.

3.2.5 Correlation and trends

The laboratory-determined undrained shear strength was plotted together with estimated undrained shear strength from CPTu to evaluate the differences between the laboratory-based and CPTu-based values. The laboratory-determined undrained shear strength s_u was also plotted against the average net cone resistance $q_{net,ave}$ and the average excess pore pressure Δu_{ave} to identify the relationship between CPTu parameters and shear strength for each borehole.

Regression analysis has been used in previous CPTu studies to evaluate correlations between CPTu parameters and laboratory-determined undrained shear strength [2, 4]. Based on this, regression analysis was used in this study to evaluate the relationship between laboratory shear strength and CPTu parameters. When $q_{net,ave}$ is plotted on the vertical axis and s_u on the horizontal axis, a zero-intercept regression gives a slope corresponding to N_{kt}. Correspondingly, when Δu_{ave} is plotted on the vertical axis and s_u on the horizontal axis, the slope corresponds to $N_{\Delta u}$. This made it possible to compare individually back-calculated factors with trend-based factors from regression analysis.

3.2.6 Soil classification

To determine the soil type in the CPTu soundings, Robertson's soil behavior type (SBT) classification was used. CPT-based soil classification is commonly described as soil behavior type classification, because CPT measurements respond to the in-situ mechanical behavior of the soil rather than directly to grain-size distribution [1]. The data used in this study were normalized cone resistance Q_t, normalized friction ratio F_r, pore pressure ratio B_q, and soil behavior type index I_c.

The normalized cone resistance was calculated as:

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}}$$

where:

- Q_t = normalized cone resistance
- q_t = corrected cone resistance
- σ_{v0} = total vertical stress
- σ'_{v0} = effective vertical stress

The normalized friction ratio was calculated as:

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} \cdot 100\%$$

where:

- F_r = normalized friction ratio
- f_s = sleeve friction
- q_t = corrected cone resistance
- σ_{v0} = total vertical stress

The normalized pore pressure ratio was calculated as:

$$B_q = \frac{\Delta u}{q_t - \sigma_{v0}} = \frac{u_2 - u_0}{q_t - \sigma_{v0}}$$

where:

- B_q = pore pressure ratio
- Δu = excess pore water pressure
- u_2 = pore water pressure measured behind the cone tip
- u_0 = initial in-situ pore water pressure
- q_t = corrected cone resistance
- σ_{v0} = total vertical stress

The soil behavior type index I_c was calculated as:

$$I_c = \left[(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2\right]^{0.5}$$

where:

- I_c = soil behavior type index
- Q_{tn} = normalized cone resistance with stress exponent
- F_r = normalized friction ratio

The normalized cone resistance normalized for stress was calculated as:

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{p_a}\right)\left(\frac{p_a}{\sigma'_{v0}}\right)^n$$

where:

- p_a = atmospheric reference pressure, taken as 100 kPa
- n = stress exponent used to account for the effect of effective overburden stress

For clay-like soils, the stress exponent can often be taken as $n = 1$ [1]. In this study, the soil classification was evaluated using three approaches: the Q_t–F_r chart, the Q_t–B_q chart, and the depth profile of I_c. The Q_t–F_r chart is based on normalized cone resistance and normalized friction ratio, while the Q_t–B_q chart includes pore pressure response and is therefore relevant for CPTu soundings in fine-grained soils. The I_c profile was used as a continuous indicator of soil behavior type with depth.

4
Results

4.1 Introduction

In this chapter, the result from the performed triaxial tests and the subsequent evaluation of CPTu data are presented. The data for this analysis from two different sites, Kärra and Utby site that belong to Chalmers university of technology. The Kärra site is located along side the Göta river, and Utby site is along side the Säveån creek in the city of Gothenburg. Kärra test side is including 4 boreholes, and triaxial data from 11 different depth. The Utby site has only one CPTu borehole and triaxial data from 7 different depth.

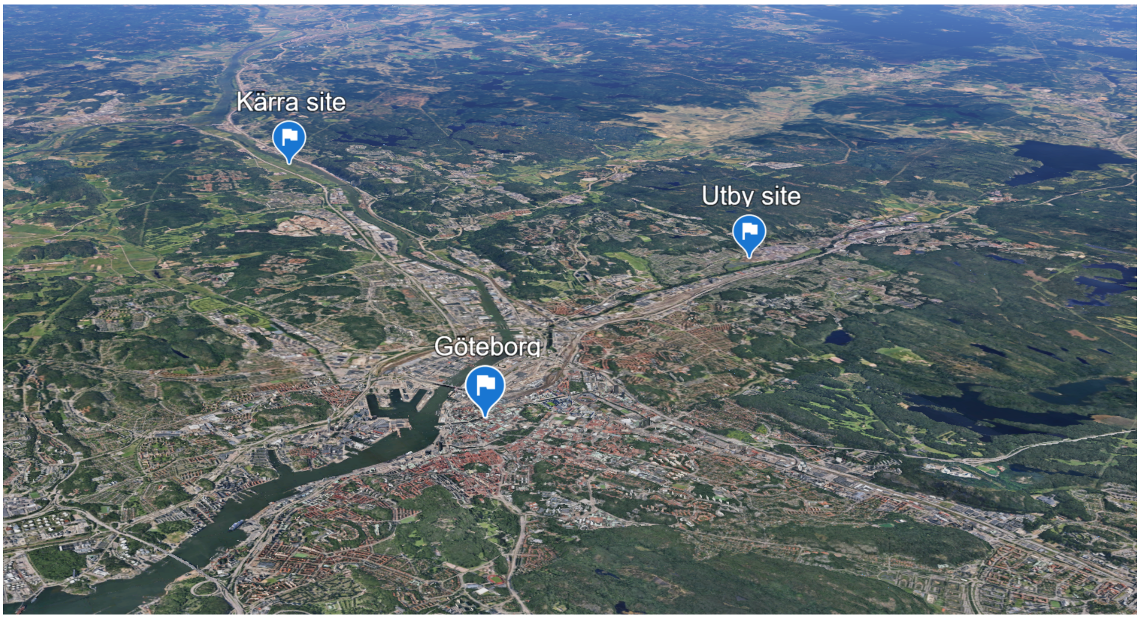

Figure 4.1: Location of test sites in Gothenburg

4.2 Kärra test site

The location of the boreholes in the Kärra test site is shown in Figure 4.2. In order to do a regression with the triaxial data from the site, the CPTu data have been used only for the highest depth that has triaxial sample which is 20 meter.

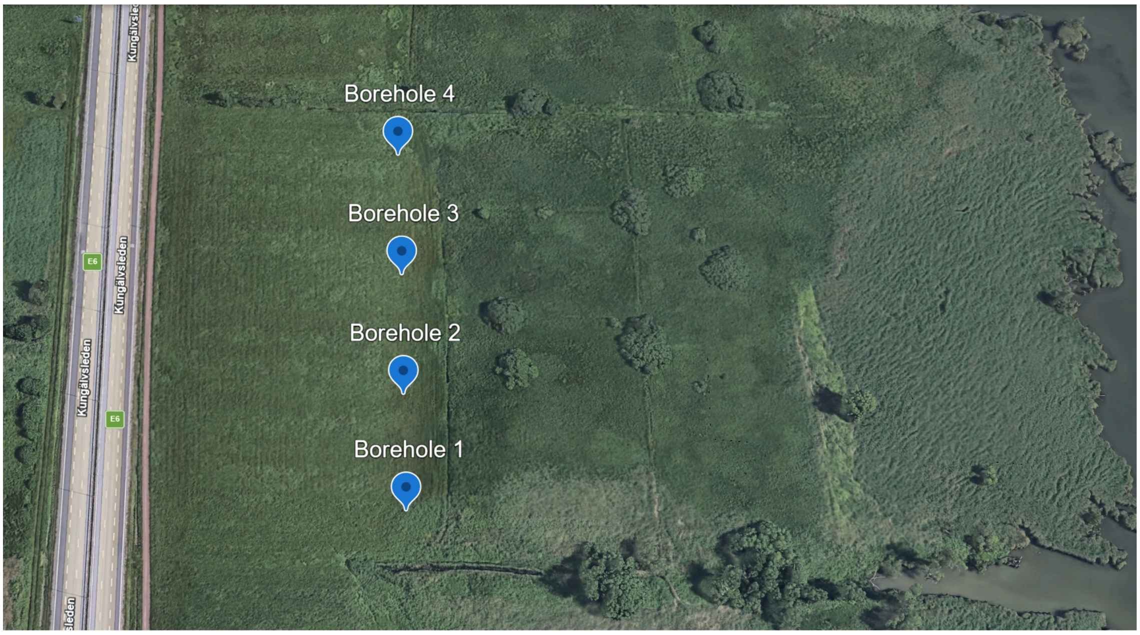

Figure 4.2: Location of boreholes in Kärra test sites

4.2.1 Soil classification

Initially, a soil classification was carried out to describe the soil profile within the investigation area. The classification was based on Robertson soil classification from available CPTu data. The perpose was to identify the main layers to justify the assumption in different calculations. The result from the soil classification show that the investigated peofile mainly consist of clay to silty clay with some organic soil mixed with the clay in the first layers. the soil classification shows a quite constant and homogeneous soil type for all four boreholes with minimal diviation in the first four to five meters. the figures below show the soil classification in the four boreholes, with three different methods. Normalized friction ratio, Normalized pore pressure and I_c index. The I_c index has a clear advantage that can classify the soil profile against the depth, while other methods show some general characterization.

Table 4.1: Soil behaviour type zones (From [1]).

Zone	Soil behaviour type	Zone	Soil behaviour type	Zone	Soil behaviour type
1	Sensitive, fine-grained	4	Silt mixtures: clayey silt to silty clay	7	Gravelly sand to sand
2	Organic soils – peats	5	Sand mixtures: silty sand to sandy silt	8	Very stiff sand to clayey sand
3	Clays: clay to silty clay	6	Sands: clean sands to silty sands	9	Very stiff fine-grained

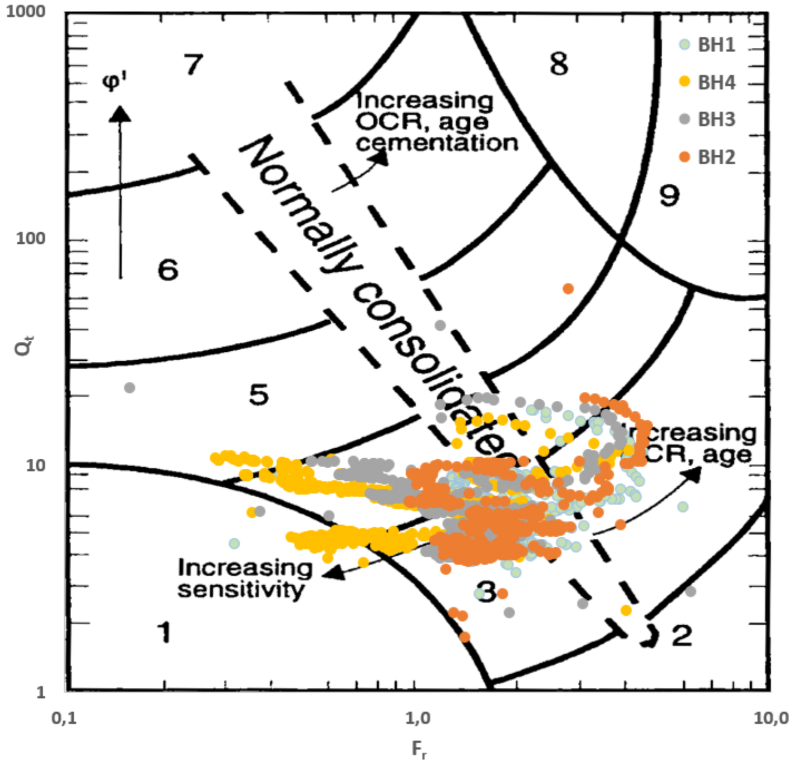

Figure 4.3: Soil classification using normalized friction F_r

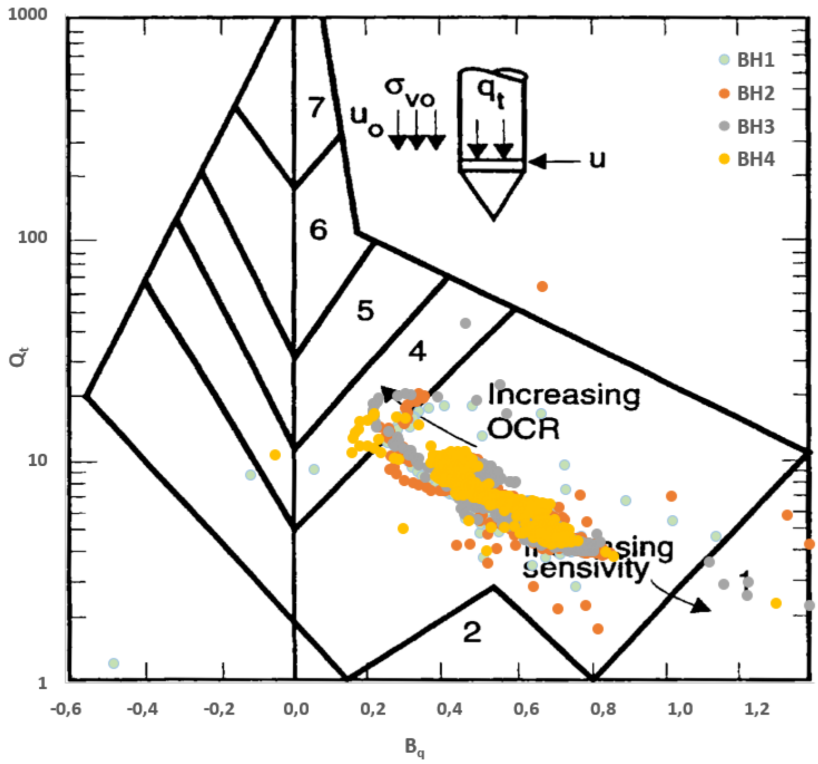

Figure 4.4: Soil classification using normalized pore pressure B_q

For I_c index method the result is flowing by this figure:

Table 4.2: Soil classification based on the I_c index (From [1])

Zone	Soil behavior type	Zone	Soil behavior type	Zone	Soil behavior type
1	Sensitive fine-grained	4	Silt mixtures: clayey silt & silty clay	7	Dense sand to gravelly sand
2	Clay – organic soil	5	Sand mixtures: silty sand to sandy silt	8	Stiff sand to clayey sand*
3	Clays: clay to silty clay	6	Sands: clean sands to silty sands	9	Stiff fine-grained*

*Overconsolidated or cemented

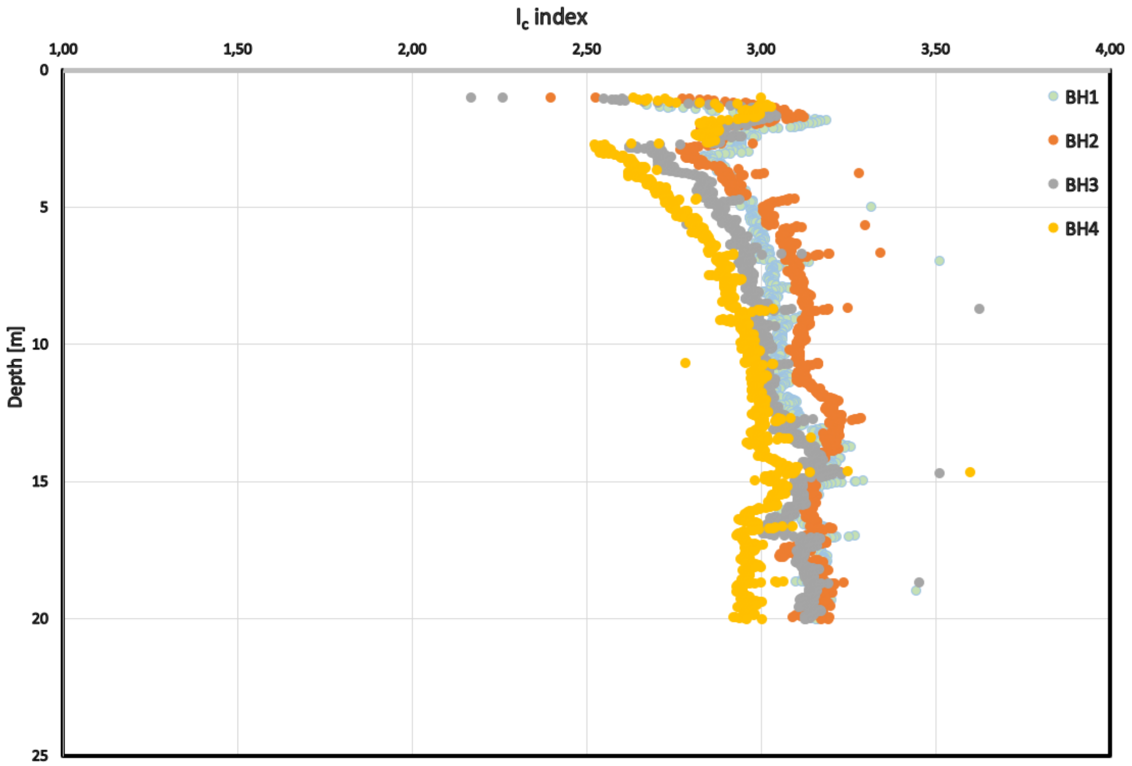

Figure 4.5: Soil classification using I_c index

4.2.2 Undrained shear strength from triaxial test

This section present the result from the performed triaxial test from samples that collected from Kärra test site. In total , triaxial test from eleven different depth have been analyzed. For each depth level, the maximum value of deviator stress was divided by two and identified as undrained shear strength s_u for that specific depth. Furthermore, all estimated undrained shear strength was plotted in one figure to create the strength profile. This profile was used to calculate the CPTu-based cone factor N_{kt}, and excess pore pressure factor $N_{\Delta u}$.

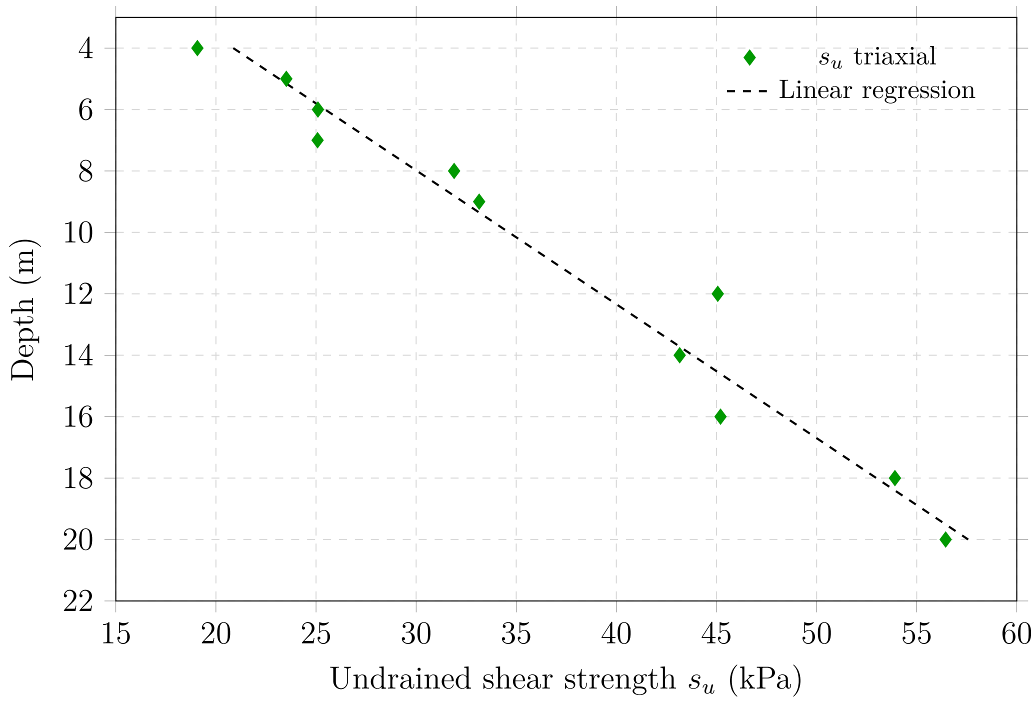

Figure 4.6: Variation of undrained shear strength s_u for Kärra test site.

4.2.3 CPTu data selection

Four CPTu boreholes have been used in this analysis. For each triaxial sample, CPTu data have been selected within a depth interval corresponding to the length of the triaxial sample, shich is approximatly 100 *mm*, thus each CPTu data has a length of 20 *mm* which led to five CPTu ponts for each triaxial sample. Table 4.3 shows an example from BH1 for depth of four meters:

Table 4.3: Typical result from CPTu parameters and calculated values.

Djup m	q_T **MPa**	f_T **kPa**	u_0 **kPa**	Δu **kPa**	**Tunghet** **kN/m^3**	σ_v **kPa**	σ'_v **kPa**	**Bh 1** **kPa**	B_q -	R_{ft} -	**B** **m/s**
3,96	0,28	2,89	29,60	100,10	15,00	59,40	29,80	13,23	0,45	1,02	20,00
3,98	0,28	2,99	29,80	100,50	15,00	59,70	29,90	13,20	0,45	1,05	18,00
4,00	0,29	2,89	30,00	100,30	15,00	60,00	30,00	13,43	0,44	1,00	21,00
4,02	0,29	2,89	30,20	100,30	15,00	60,30	30,10	13,36	0,44	1,00	18,00
4,04	0,29	2,81	30,40	94,70	15,00	60,60	30,20	13,23	0,42	0,98	18,00
Mean	**0,29**			**99,18**		**60,00**		**13,29**			

Table 4.4: Summary of mean q_n, mean Δu, and triaxial undrained shear strength s_u for BH1–BH4.

Depth (m)	BH1		BH2		BH3		BH4		Triaxial
	q_n (kPa)	Δu (kPa)	q_n (kPa)	Δu (kPa)	q_n (kPa)	Δu (kPa)	q_n (kPa)	Δu (kPa)	s_u (kPa)
4	225.93	99.18	204.92	105.52	231.40	94.46	259.47	99.68	19.08
5	216.84	116.82	214.66	124.38	253.41	119.30	276.66	120.14	23.52
6	277.37	132.40	235.16	147.82	265.14	131.40	297.00	136.18	25.10
7	272.86	147.58	256.79	160.22	284.40	162.32	312.53	146.84	25.08
8	330.67	186.58	278.04	173.34	330.43	176.48	345.23	179.44	31.90
9	360.85	197.76	290.22	191.56	368.77	207.36	376.13	208.28	33.15
12	451.30	261.38	315.28	209.46	472.48	281.04	452.43	258.30	45.07
14	364.35	246.70	344.85	246.88	384.66	250.38	524.03	313.92	43.16
16	485.54	310.92	403.10	287.32	453.82	298.20	447.44	300.24	45.20
18	431.10	313.90	471.61	339.26	475.67	324.86	510.12	339.80	53.91
20	503.92	360.84	495.88	267.74	496.42	360.82	556.13	352.18	56.45

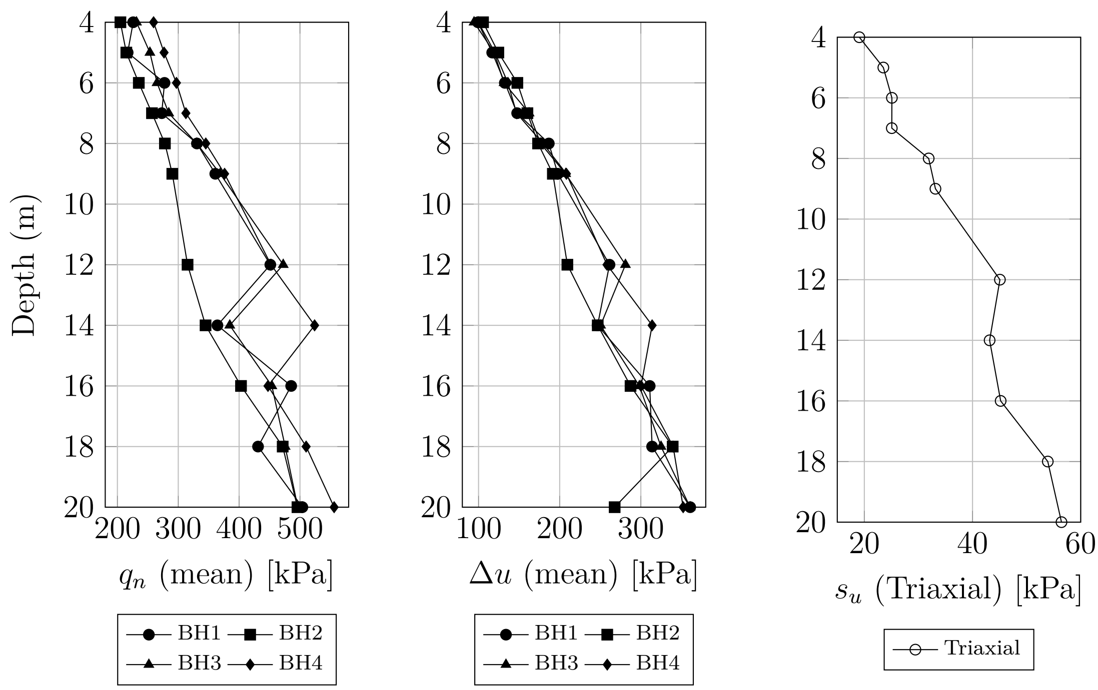

Figure 4.7: Variation of mean net cone resistance q_n, mean excess pore pressure Δu, and triaxial undrained shear strength s_u with depth for BH1–BH4.

4.2.4 Calculation of N_{kt}, and $N_{\Delta u}$

Further, the cone factors N_{kt} and $N_{\Delta u}$ have been calculated for each analyzed depth. The cone factor N_{kt} was calculated from the mean value of net cone resistance divided by the undrained shear strength from triaxial test and excess pore pressure $N_{\Delta u}$ obtained from excess pore pressure Δu divided by undrained shear strength from triaxial test.

Table 4.5: Summary of the cone factors N_{kt} and $N_{\Delta u}$ for BH1–BH4.

Depth (m)	**BH1**		**BH2**		**BH3**		**BH4**	
	N_{kt}	$N_{\Delta u}$	N_{kt}	$N_{\Delta u}$	N_{kt}	$N_{\Delta u}$	N_{kt}	$N_{\Delta u}$
4	11.84	5.20	10.74	5.53	12.13	4.95	13.60	5.23
5	9.22	4.97	9.13	5.29	10.78	5.07	11.76	5.11
6	11.05	5.28	9.37	5.89	10.57	5.24	11.84	5.43
7	10.88	5.88	10.24	6.39	11.34	6.47	12.46	5.86
8	10.37	5.85	8.72	5.43	10.36	5.53	10.82	5.63
9	10.89	5.97	8.76	5.78	11.12	6.26	11.35	6.28
12	10.01	5.80	7.00	4.65	10.48	6.24	10.04	5.73
14	8.44	5.72	7.99	5.72	8.91	5.80	12.14	7.27
16	10.74	6.88	8.92	6.36	10.04	6.60	9.90	6.64
18	8.00	5.82	8.75	6.29	8.82	6.03	9.46	6.30
20	8.93	6.39	8.78	4.74	8.79	6.39	9.85	6.24

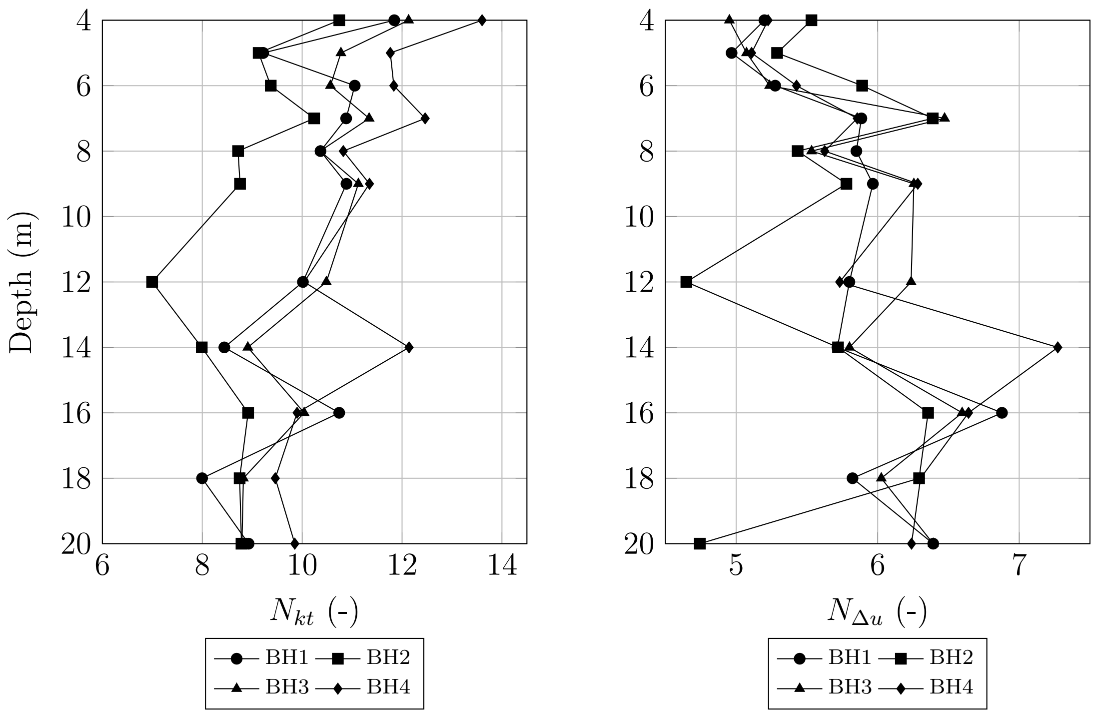

Figure 4.8: Variation of the cone factors N_{kt} and $N_{\Delta u}$ with depth for BH1–BH4.

4.2.5 Relationship between undrained shear strength s_u vs. net cone resistance q_n and excess pore pressure Δu

To evaluate the relationship between laboratory-determined parameters and CPTu parameters, net cone resistance q_n and excess pore pressure Δu was plotted against the undrained shear strength s_u.

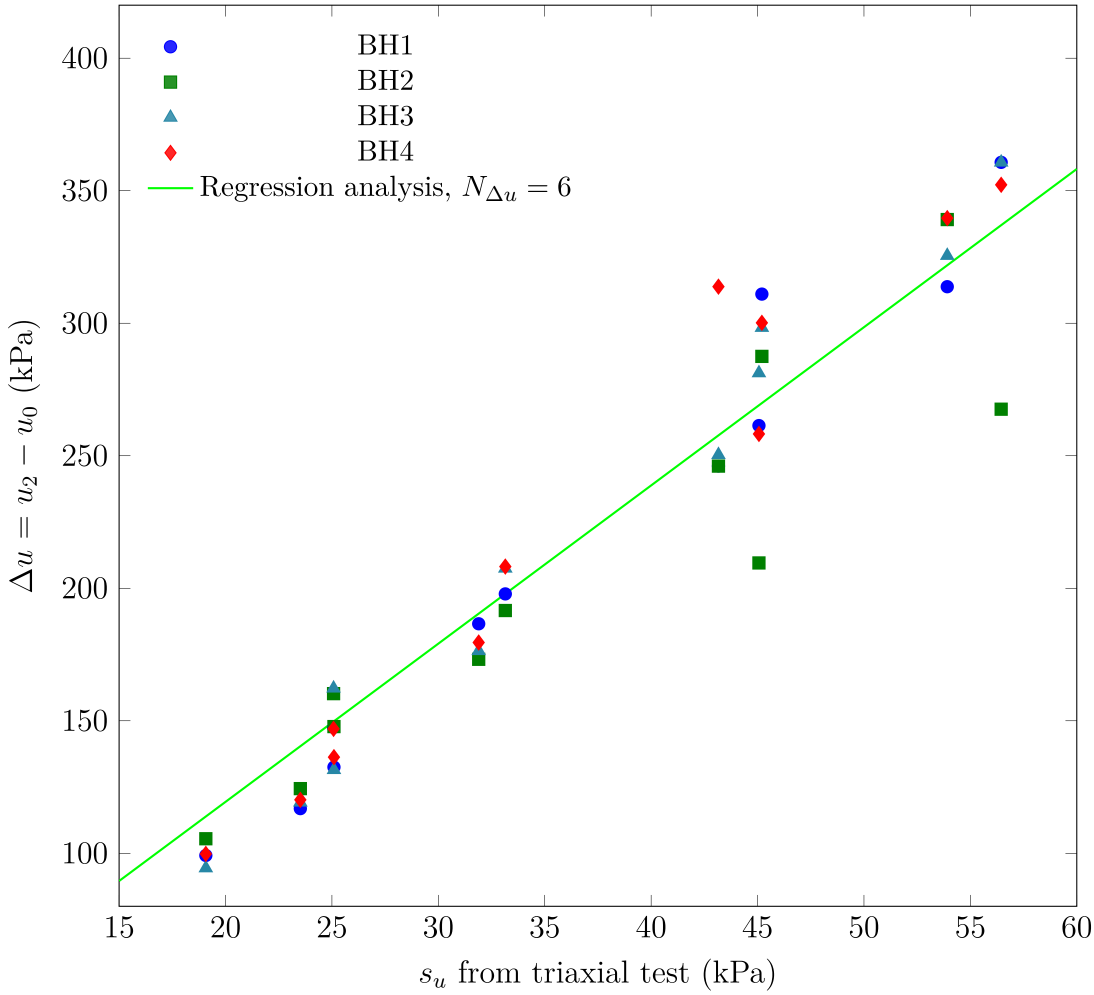

Figure 4.9: Relationship between triaxial undrained shear strength s_u and the CPTu parameters Δu, the green trend line represent the $N_{\Delta u}$ which has an inclination of 6 in this figure.

The relationship between undrained shear strength s_u and net cone resistance q_n is shown in Figure4.9 below:

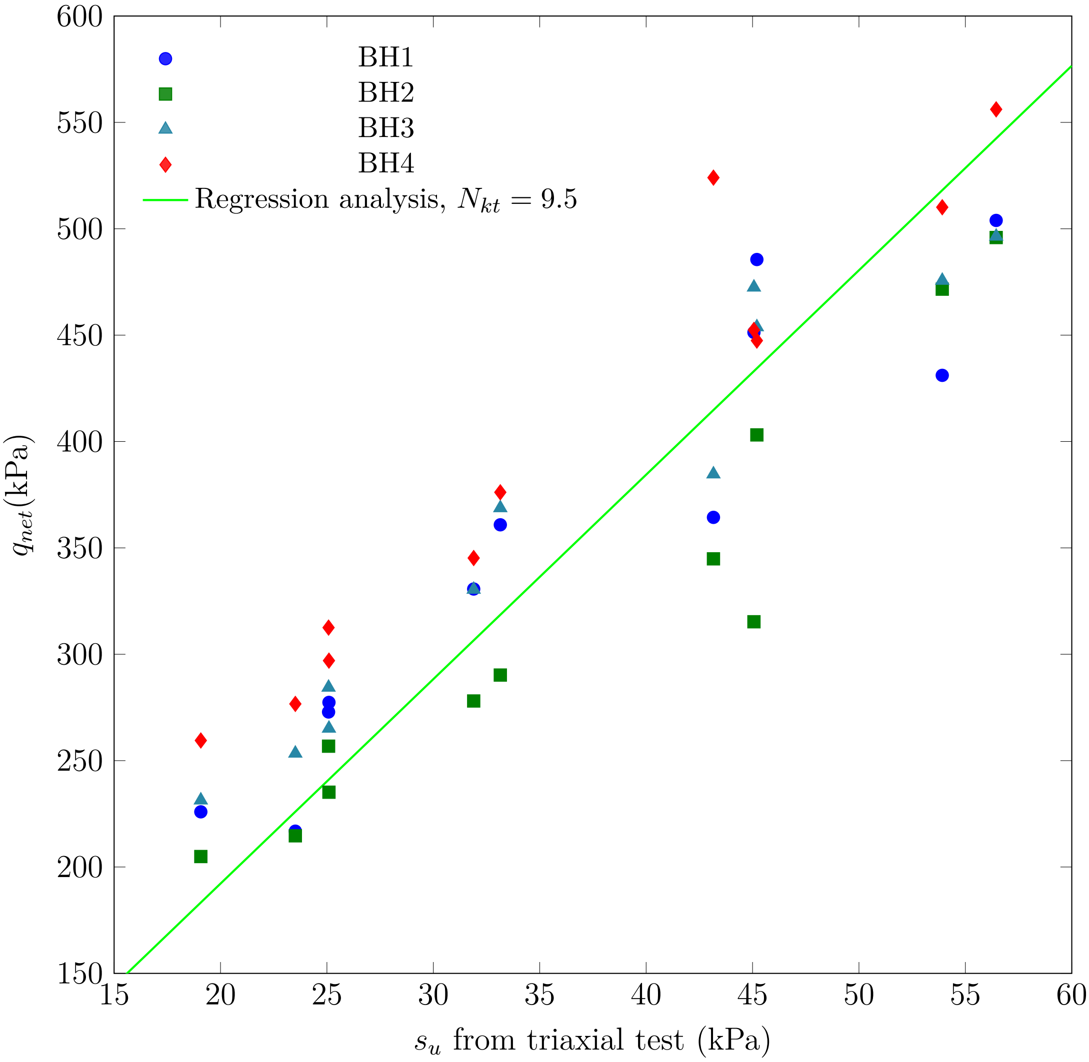

Figure 4.10: Relationship between triaxial undrained shear strength s_u and the CPTu parameters q_n, the green trend line represent the N_{kt} which has an inclination of 9,5 in this figure.

4.2.6 Regression result

The following Figures present the range in derived undrained shear strength s_u obtained for different cone factors.

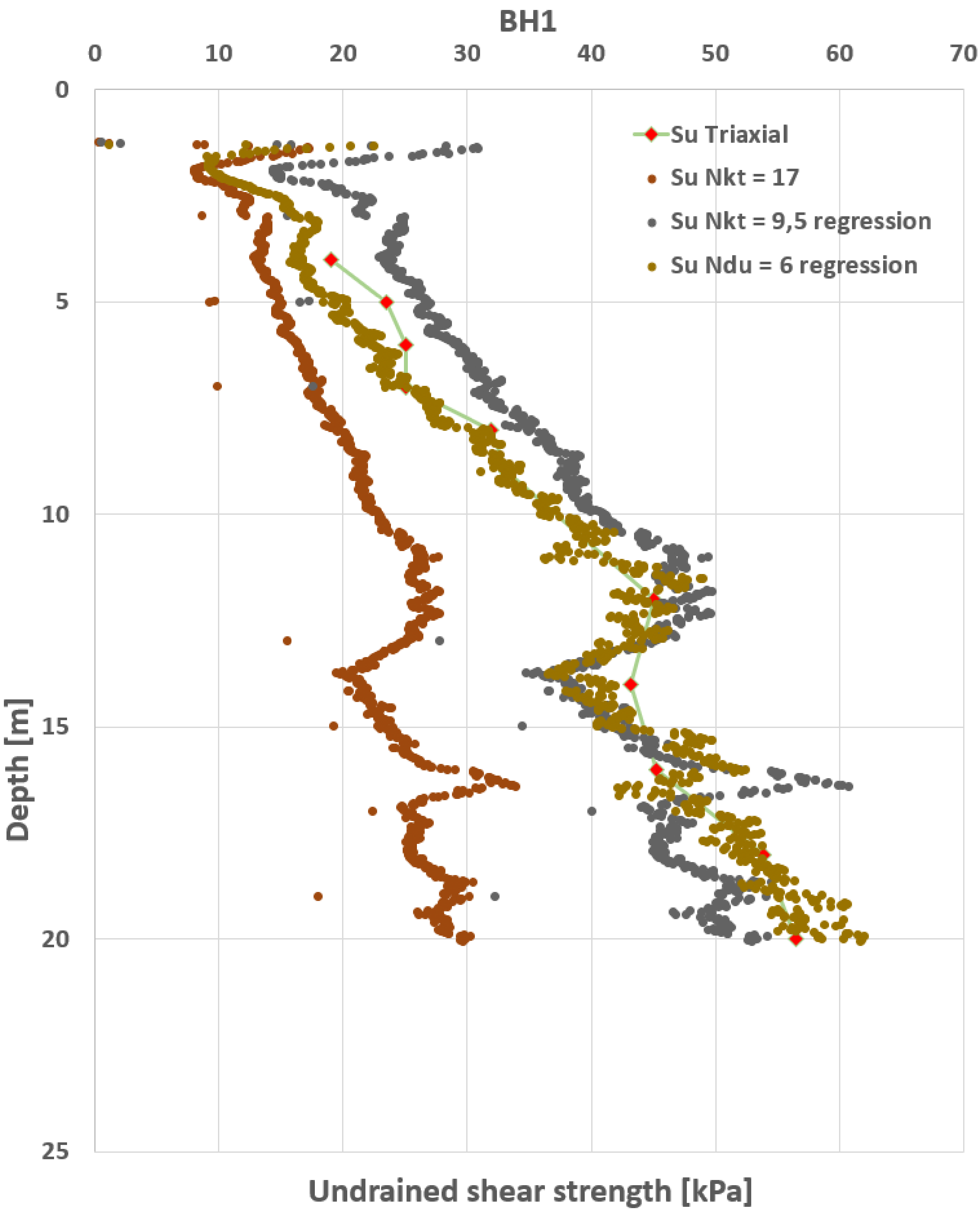

Figure 4.11: Undrained shear strength s_u driven by different cone factors for BH1

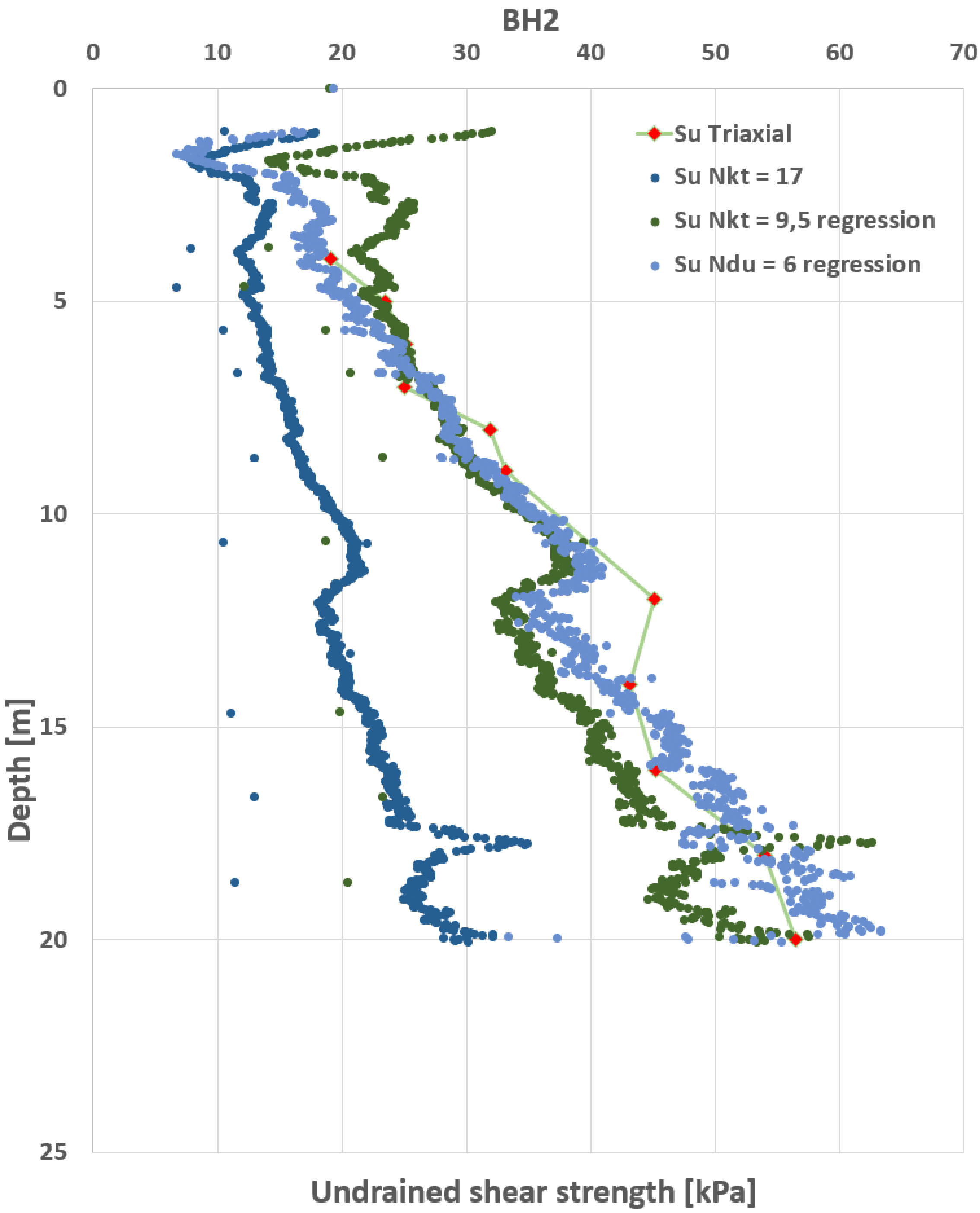

Figure 4.12: Undrained shear strength s_u driven by different cone factors for BH2

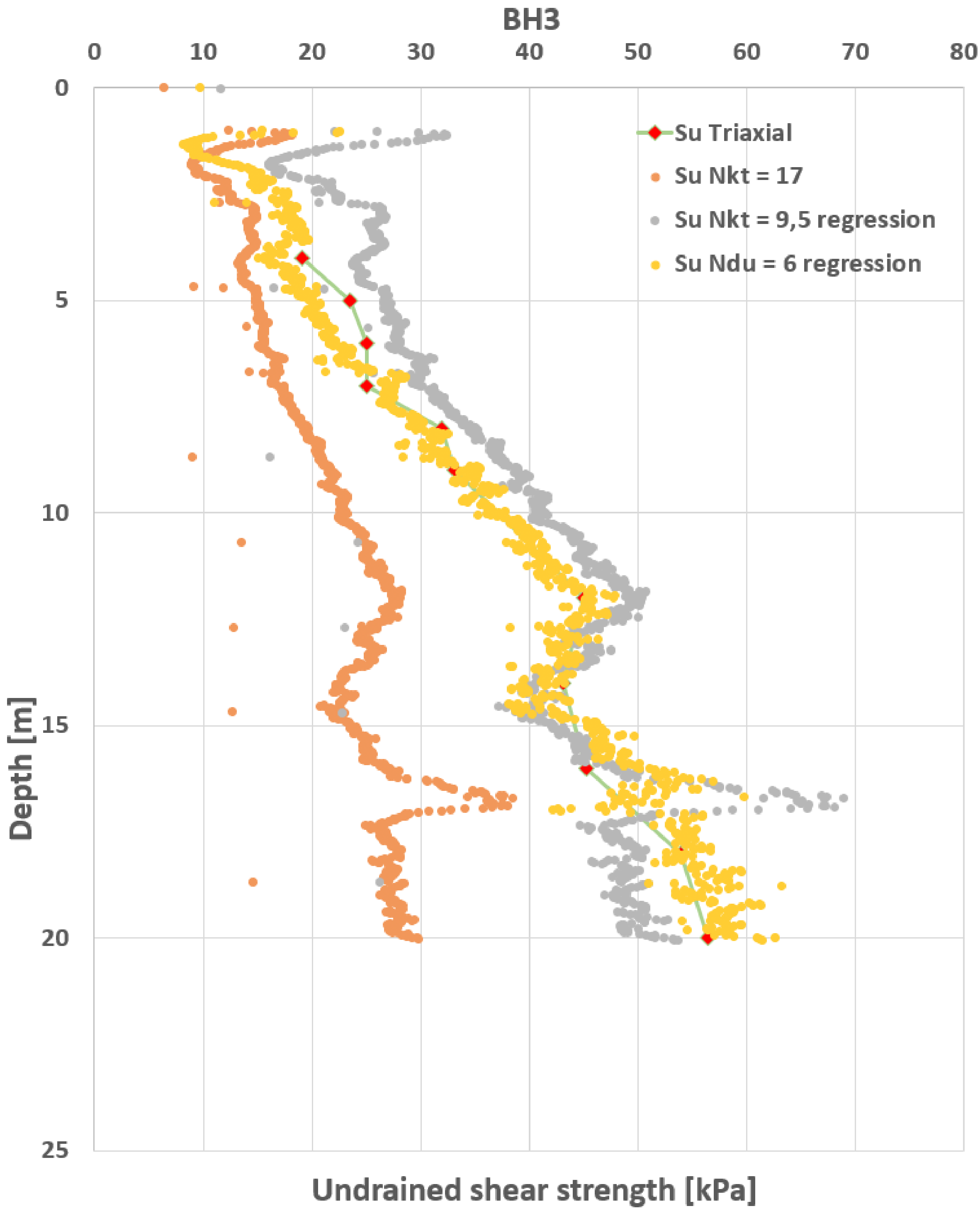

Figure 4.13: Undrained shear strength s_u driven by different cone factors for BH3

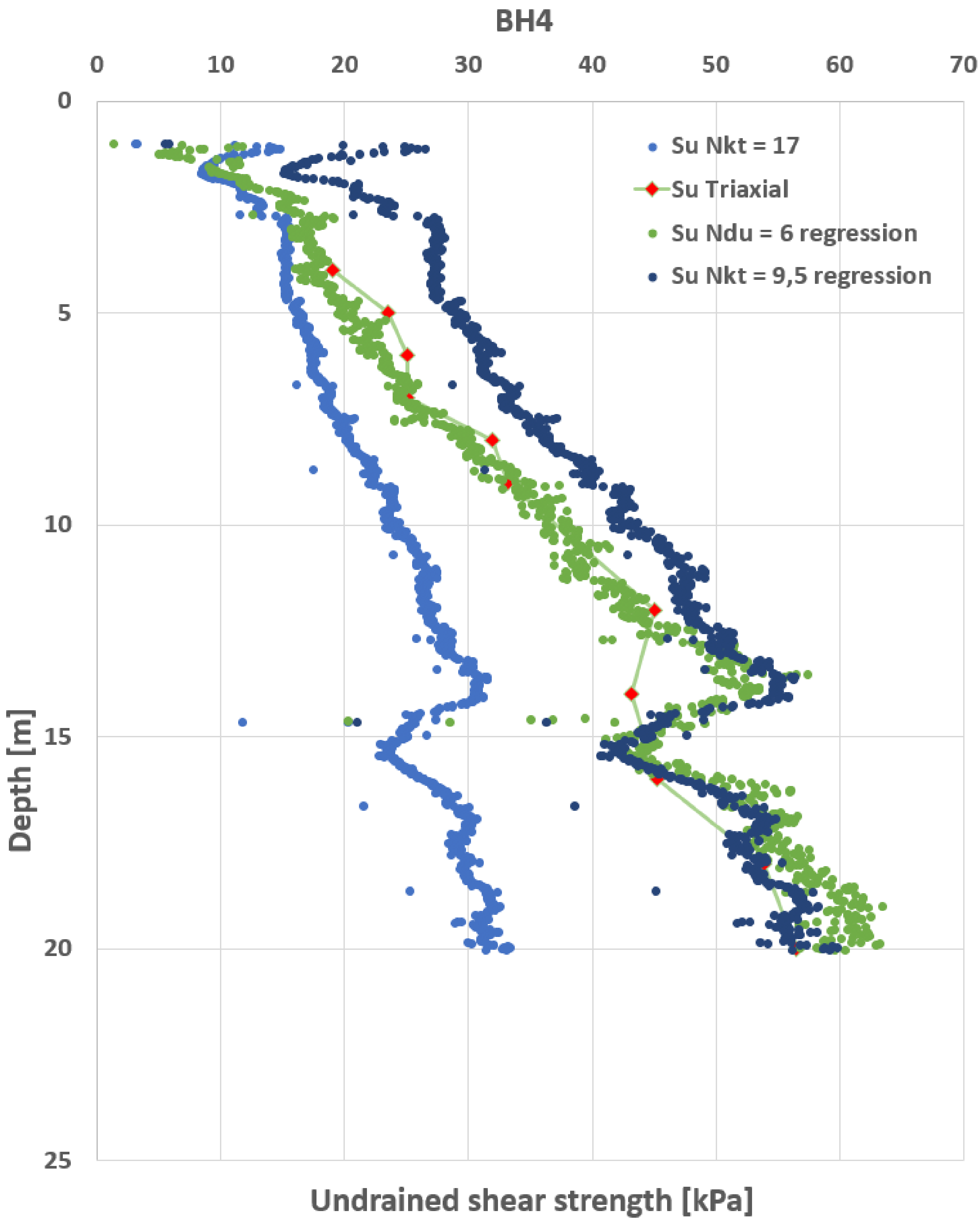

Figure 4.14: Undrained shear strength s_u driven by different cone factors for BH4

4.3 Utby test site

The location of boreholes in the Utby test site is shown in Figure 4.15. At this site, CPTu data were evaluated down to 25m depth, which corresponding to the deepest level where triaxial data were available. In contrast to the Kärra test site, only one CPTu borehole was available for the Utby analysis. The same evaluation procedure as described for Kärra was applied to the Utby test site. Therefore this section is presented more briefly, with focus on the obtained results rather than a repeated description of procedure. The data selection, averaging method and calculation of N_{kt} and $N_{\Delta u}$ are described detailed in Kärra section.

Figure 4.15: Location of borehole in Utby test site

4.3.1 Soil classification

A soil classification was also carried out for the Utby site to describe the soil profile for the investigation area, which is based on Robertson classification. The results show that the investigated profile mainly consists of clay to silty clay here as well. Some minor variation was observed in the upper part, but the remaining part is a relatively uniform soil behavior. The result from the normalized friction ratio F_r deviates slightly in comparison to normalized pore pressure ratio B_q and I_c index method, but all three methods indicates mainly clay to silty clay.

Table 4.6: Soil behavior type zones (From [1]).

Zone	Soil behavior type	Zone	Soil behavior type	Zone	Soil behavior type
1	Sensitive, fine-grained	4	Silt mixtures: clayey silt to silty clay	7	Gravelly sand to sand
2	Organic soils – peats	5	Sand mixtures: silty sand to sandy silt	8	Very stiff sand to clayey sand
3	Clays: clay to silty clay	6	Sands: clean sands to silty sands	9	Very stiff fine-grained

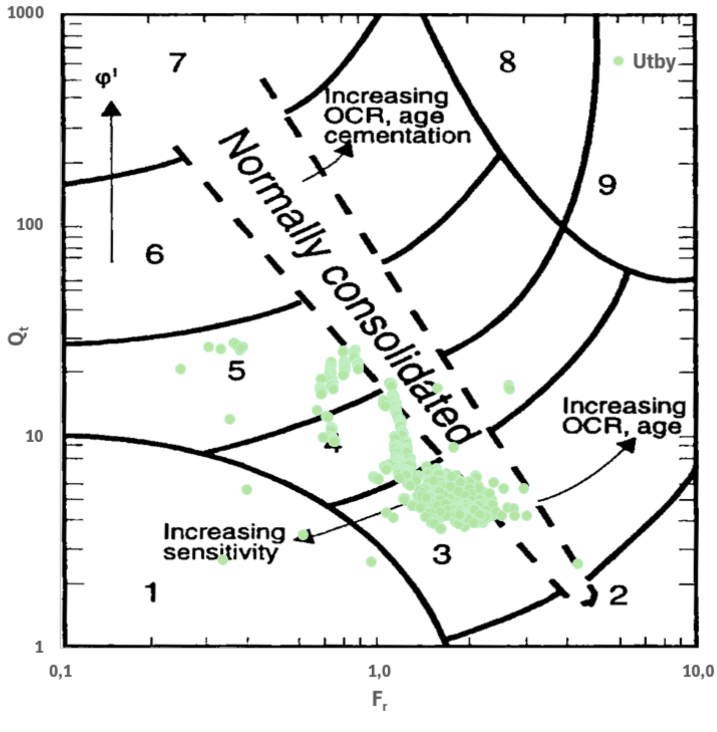

Figure 4.16: Soil classification using normalized friction F_r

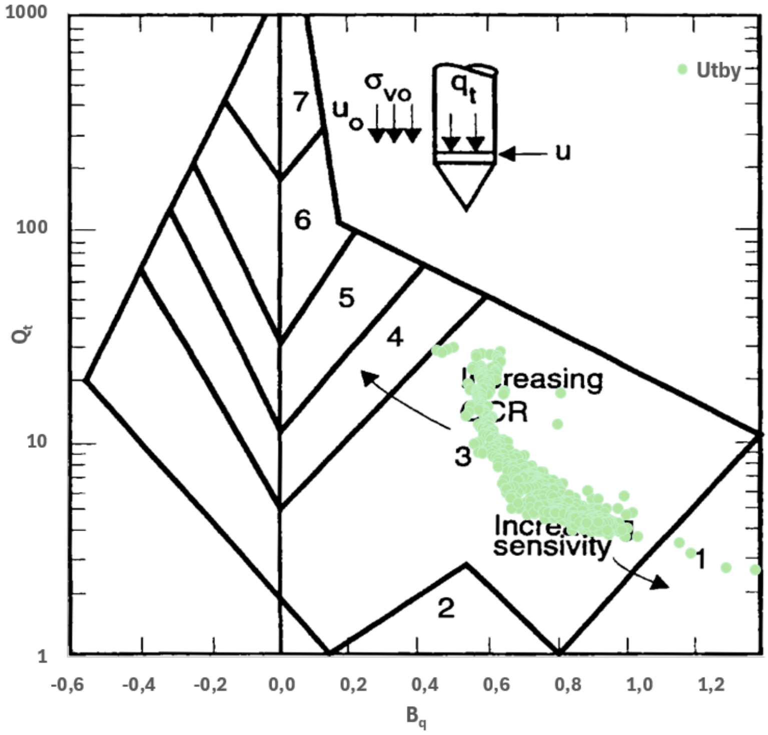

Figure 4.17: Soil classification using normalized pore pressure B_q

For I_c index method the result is flowing by this figure:

Table 4.7: Soil classification based on the I_c index (From [1]).

Zone	Soil behavior type	Zone	Soil behavior type	Zone	Soil behavior type
1	Sensitive fine-grained	4	Silt mixtures: clayey silt & silty clay	7	Dense sand to gravelly sand
2	Clay – organic soil	5	Sand mixtures: silty sand to sandy silt	8	Stiff sand to clayey sand*
3	Clays: clay to silty clay	6	Sands: clean sands to silty sands	9	Stiff fine-grained*

*Overconsolidated or cemented

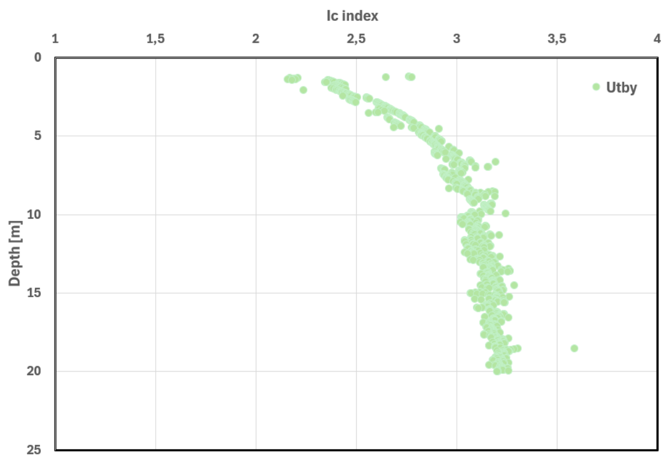

Figure 4.18: Soil classification using I_c index

4.3.2 Undrained shear strength from triaxial test

The result from the performed triaxial test from samples that collected from Utby test site presents here. In total, triaxial test from seven different depth have been analyzed. All estimated undrained shear strengths s_u were plotted in one figure to create the strength profile for the investigated area. This profile was used to calculate the CPTu-based cone factor N_{kt}, and excess pore pressure factor $N_{\Delta u}$.

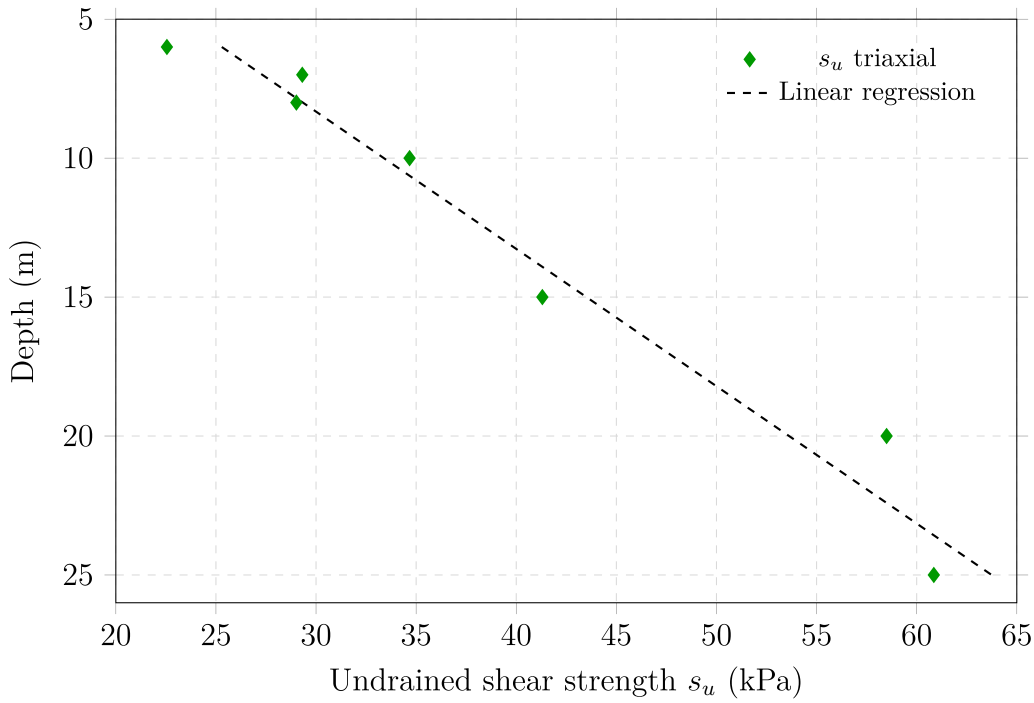

Figure 4.19: Variation of undrained shear strength s_u for Utby test site.

4.3.3 CPTu data selection

The same procedure has been applied for Utby test site, by matching the CPTu data corresponding to the length of the triaxial sample, around 100 mm, hence, five CPTu points chosen for each triaxial sample. Table 4.8 shows an example for depth of six meters:

Table 4.8: Typical result for CPT parameters and calculated values.

Depth (m)	q_T **(MPa)**	f_T **(kPa)**	u_0 **(kPa)**	Δu **(kPa)**	**Unit weight (kN/m^3)**	σ_v **(kPa)**	σ'_v **(kPa)**	**CPT G32 (kPa)**	B_q **(-)**	R_{ft} **(-)**	**B (m/s)**
5.96	0.29	2.39	59.60	144.40	15.50	89.74	30.14	12.06	0.70	0.81	18.00
5.98	0.30	2.37	59.80	149.20	15.50	90.05	30.25	12.09	0.73	0.80	19.00
6.00	0.29	2.39	60.00	143.00	15.50	90.36	30.36	12.02	0.70	0.81	17.00
6.02	0.31	2.40	60.20	140.80	15.50	90.67	30.47	13.04	0.64	0.77	18.00
6.04	0.31	2.39	60.40	143.60	15.50	90.98	30.58	13.05	0.65	0.76	24.00
Mean	**0.30**			**144.20**		**90.36**		**12.45**			

Table 4.9: Summary of mean values of q_n, Δu, and s_u with depth.

Depth (m)	q_n (mean) [kPa]	Δu (mean) [kPa]	s_u [kPa]
6	211.6952	144.2	22.5524
7	186.1688	164.8	20.3125
8	268.3744	182.4	29.0183
10	285.3768	227.8	34.6732
15	409.5416	293.6	41.30425
20	446.7660	391.0	58.4966
25	504.3196	476.6	60.85635

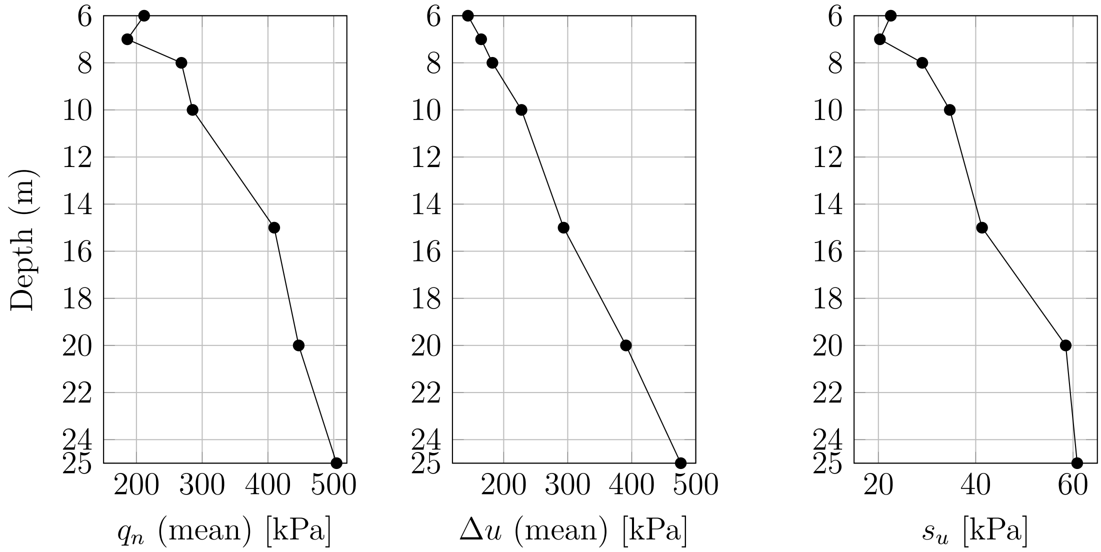

Figure 4.20: Variation of mean net cone resistance q_n, mean excess pore pressure Δu, and undrained shear strength s_u with depth.

4.3.4 Calculation of N_{kt}, and $N_{\Delta u}$

The same procedure as Kärra test site, the cone factor N_{kt} and excess pore pressure factor $N_{\Delta u}$ was calculated by data from CPTu and undrained shear strength s_u from triaxial.

Table 4.10: Summary of N_{kt} and $N_{\Delta u}$ with depth.

Depth (m)	N_{kt} (-)	$N_{\Delta u}$ (-)
6	9.3868	6.3940
7	9.1652	8.1132
8	9.2485	6.2857
10	8.2305	6.5699
15	9.9152	7.1082
20	7.6375	6.6841
25	8.2870	7.8316

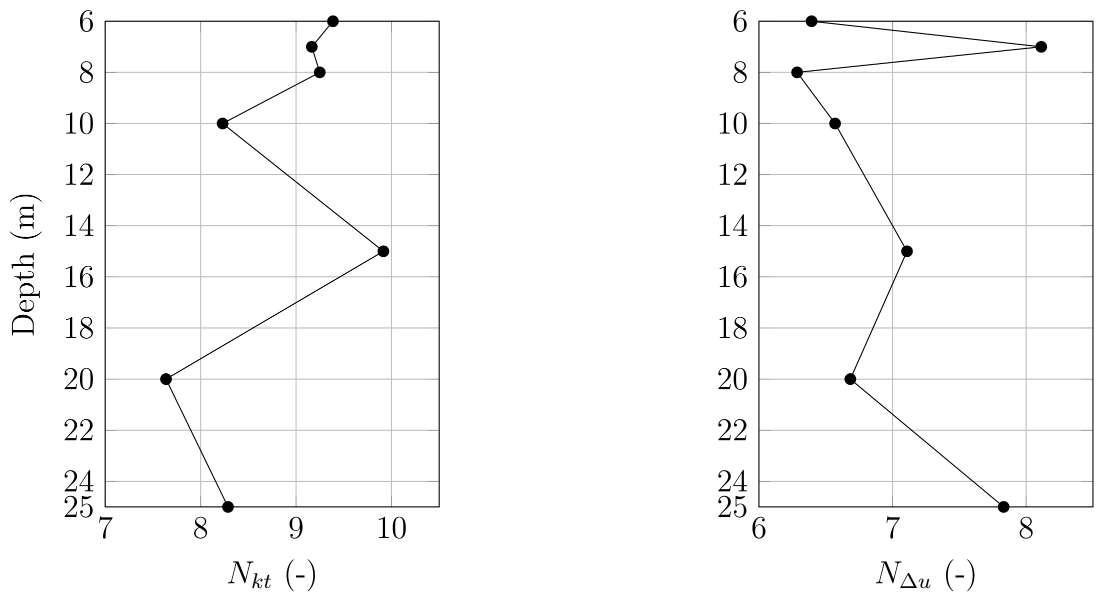

Figure 4.21: Variation of N_{kt} and $N_{\Delta u}$ with depth.

4.3.5 Relationship between undrained shear strength s_u versus net cone resistance q_n and excess pore pressure Δu

Same procedure for evaluating the relationship between laboratory-determined parameters and CPTu parameters, net cone resistance q_n and excess pore pressure Δu was plotted against the undrained shear strength s_u.

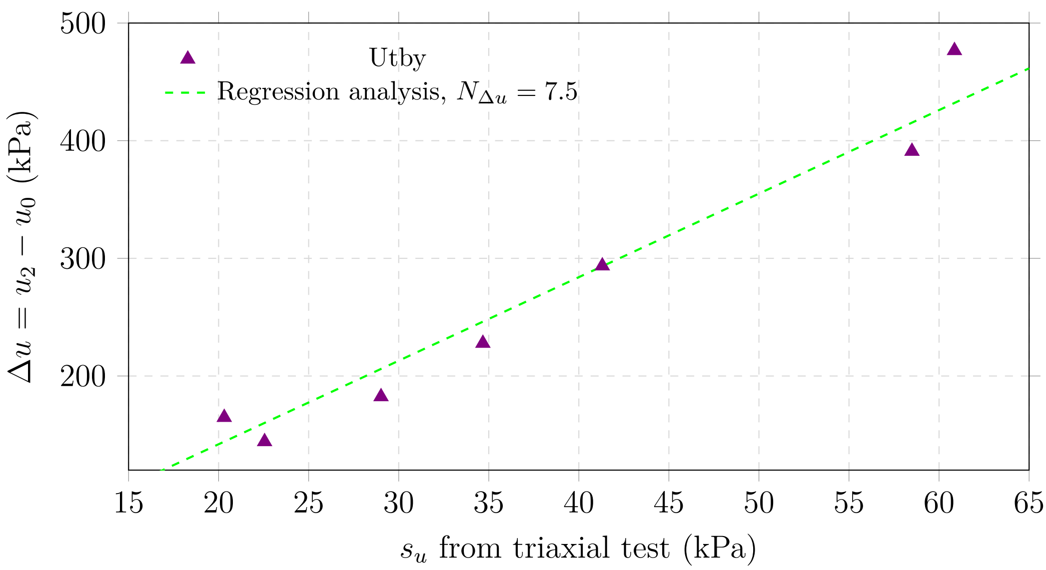

Figure 4.22: Relationship between triaxial undrained shear strength s_u and the CPTu parameters Δu, the green trend line represent the $N_{\Delta u}$ which has a inclination of 7.5 in this figure.

The relationship between undrained shear strength s_u and net cone resistance q_n shows in figure below:

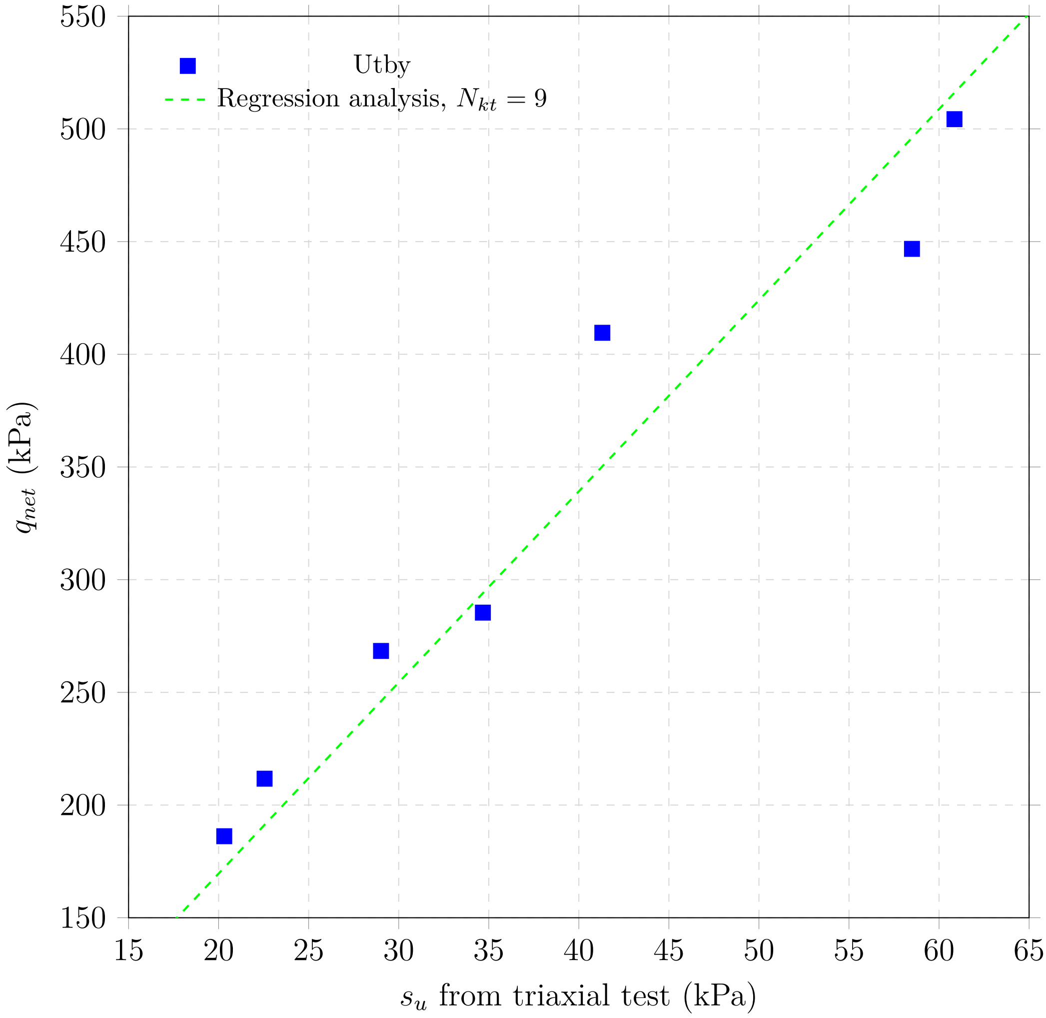

Figure 4.23: Relationship between triaxial undrained shear strength s_u and the CPTu parameters q_n, the green trend line represent the N_{kt} which has a inclination of 9 in this figure.

4.3.6 Regression result

The following Figure presents the range in derived undrained shear strength s_u obtained for different cone factors.

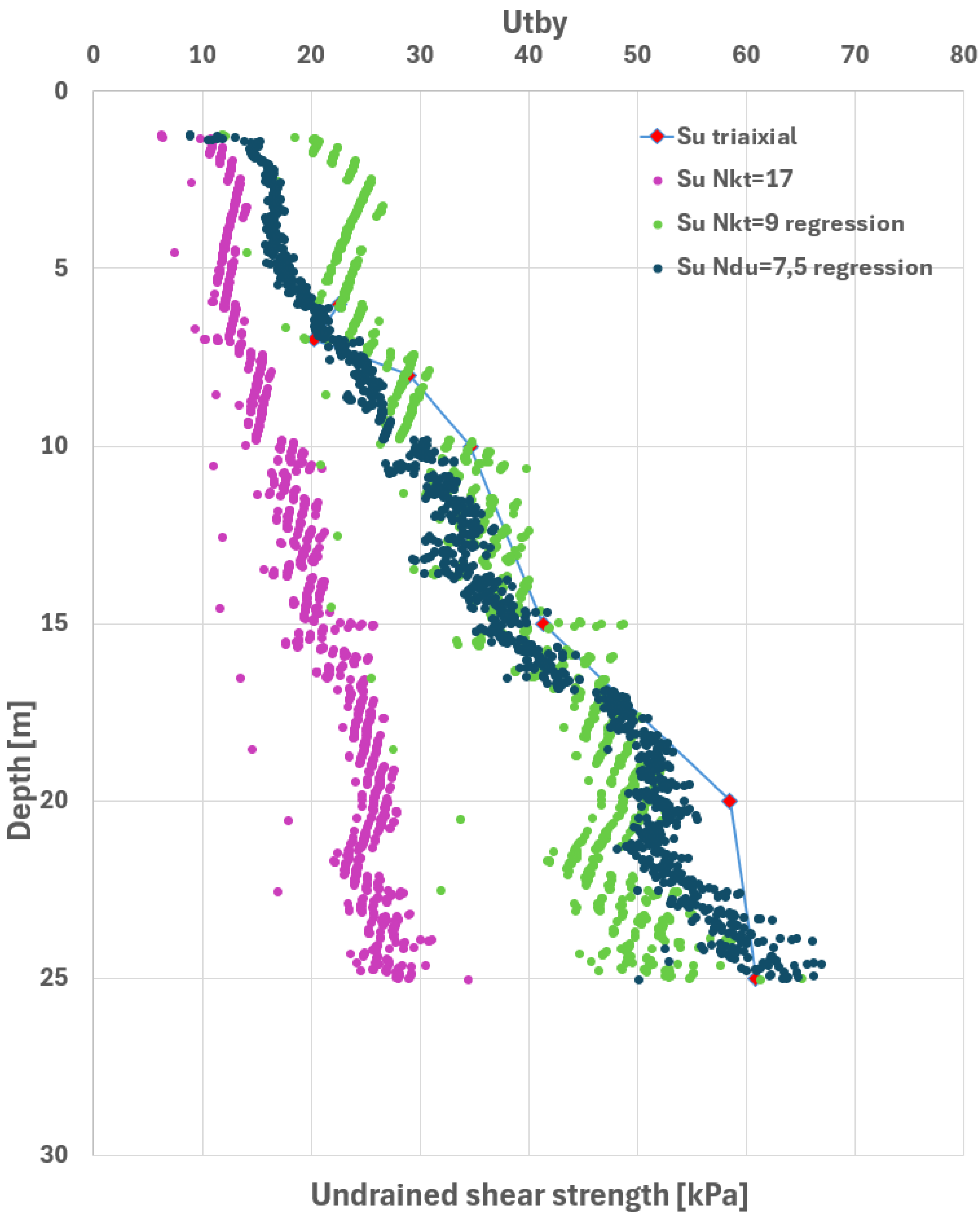

Figure 4.24: Undrained shear strength s_u driven by different cone factors for Utby

5 Discussion

5.1 Importance of soil classification

In order to identify the dominant soil behavior within the investigated sites, as well as to support the assumptions used in the calculation of different properties such as undrained shear strength s_u and cone factors, CPTu-based soil classification was carried out. The soil classification was performed using the normalized friction ratio F_r, normalized pore pressure ratio B_q, and soil behavior type index I_c on CPTu data from both the Kärra and Utby test sites. Robertson and Cabal describe this type of CPT-based soil classification as soil behavior type classification, because the classification is based on the in-situ response during cone penetration rather than directly on grain-size distribution [1].

The I_c index classification provided the most simple depth-dependent representation of the soil profile. Compared with the other two methods, the normalized friction ratio and normalized pore pressure ratio, the I_c index allowed a more direct visualization of changes in soil behavior with depth.

The normalized friction ratio F_r from the Kärra site indicated mostly clay and silt mixtures, from silty clay to clayey silt, for the majority of the soil profile. The normalized pore pressure ratio B_q indicated a dominant silty clay soil profile. The result from the SBT I_c index showed clay to silty clay, which follows almost the same trend as the normalized pore pressure ratio B_q, with some mixture of organic soil in the upper layer. This is considered reasonable since the upper part of natural clay deposits may contain organic material or more disturbed soil conditions which would have affected the sample quality.

The soil classification for the Utby test site showed similar results. The normalized friction ratio F_r method indicated mostly silty clay, but with some tendency towards clayey silt and silty sand. However, the normalized pore pressure ratio B_q indicated a more dominant and homogeneous clay to silty clay soil profile. The I_c index method indicated similar results to the pore pressure ratio B_q, with a mixture of organic soil in the upper layer. These results from soil classification indicate that the CPTu data hade high quality and therefore all different methods specify almost the same result in the both Utby and Kärra test site.

Generally, the soil classification supports the assumption that the main part of the analysis was performed within fine-grained soils. This is important, since the basic undrained shear strength s_u is calculated on the assumption that the soil profile is predominantly clay.

5.2 Evaluation of accuracy of derived cone factors

The undrained shear strength s_u obtained from the triaxial tests was used as a reference for the evaluation of the CPTu-based cone factors for the cone resistance and pore pressures at the u_2 position.
The results from both Kärra and Utby show a variation of undrained shear strength s_u with depth. This variation is expected in natural clay deposits and can reflect changes in water content, stress history, soil structure, and composition. However, the Utby test site is based on fewer triaxial samples and CPTu data from only one borehole. For the Kärra test site, the availability of four CPTu boreholes allowed a broader assessment of the variation in CPTu response. This made it possible to compare the calculated cone factors between boreholes and identify whether the results were consistent across the site. The assessment from Kärra test site indicates consistent results except for the few meters in the upper layer of the soil profile.

A central part of the analysis was the method used to connect CPTu data to triaxial test results. The triaxial tests represent discrete depths, while CPTu data provide almost continuous measurements with depth. The combination of these two data types requires careful matching between laboratory sample depth and CPTu depth. In this study, this was handled by selecting five CPTu points over a 100 mm interval corresponding approximately to the length of each triaxial sample. The similar approach was used by [5].
The results show that cone factors varies with depth. This is expected because CPTu parameters vary with depth, indicating that choosing a single value of cone factor, for instance the highest obtained value to cover weak layers or the mean value from the whole dataset, may not fully describe the relationship between CPTu data and undrained shear strength. Therefore, regression analysis was used as a complementary way to evaluate an overall trend between CPTu and triaxial-based s_u.

5.2.1 N_{kt}

Conducting a regression between q_{net} and s_u for the Kärra test site indicated that a value of approximately 9.5 can be chosen as a local value for N_{kt} in Figure 4.10. By repeating the same procedure for the Utby test site, a value of approximately 9 can be chosen as a local value for N_{kt} in Figure 4.23. These values are clearly lower than commonly assumed values used in preliminary CPTu interpretation. Robertson and Cabal report typical N_{kt} values in the range of approximately 10–18, while Mayne and Peuchen show that back-calculated values can vary significantly depending on soil type and reference strength mode [1, 4].

A recent study performed on Norwegian clays presented a similar type of evaluation, i.e. deriving cone factors from CAUC tests [2]. Therefore, it was of interest to compare how the Kärra and Utby results relate to the Norwegian clay database. Figure 5.1 shows the relationship between triaxial-based s_u and CPTu-based q_{net} for Norwegian clay together with the results from Kärra and Utby in Guthenburg.

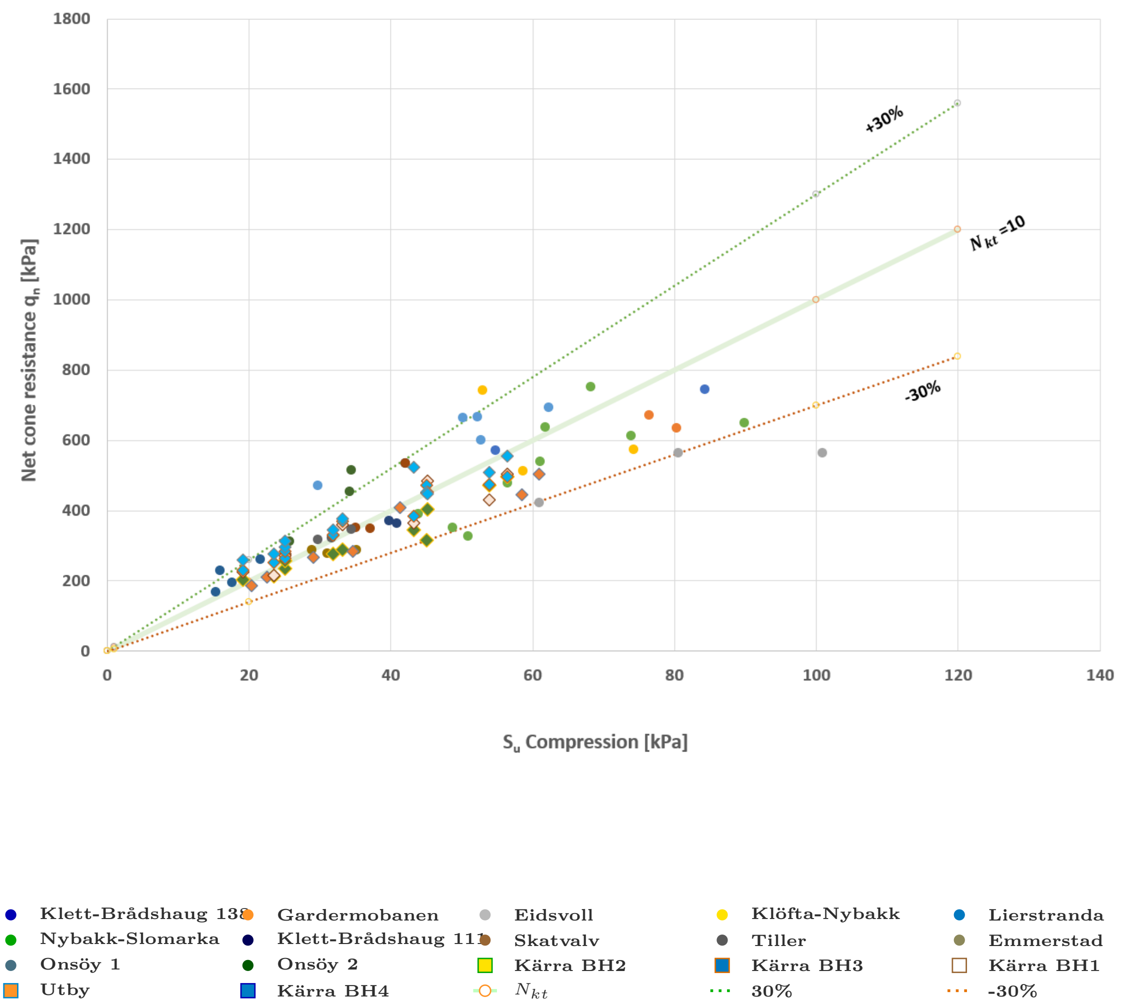

Figure 5.1: Net cone resistance q_{net} against undrained shear strength s_u from triaxial tests(Modified from [2]).

Figure 5.1 shows that the Kärra and Utby data follow the same general trend as the Norwegian clay data, however in Norwegian study the $N_{kt} = 9.37$ which is slightly lower than the $N_{kt} = 10$ by adding the Gothenburgs results to the data set, but the locally calibrated ratio between q_{net} and s_u is significantly lower than the general recommended value of $N_{kt} = 17$ used in the preliminary interpretation for clay without any laboratory calibration [2, 11]. This difference highlights the importance of site-specific calibration when laboratory reference data are available. However, it is important to note that the values obtained in this study are locally calibrated and should not be implemented as general correlations for other clay deposits.

5.2.2 $N_{\Delta u}$

The assessment of $N_{\Delta u}$ was carried out using the same general procedure as for the cone factor N_{kt}. However, instead of using net cone resistance q_{net}, the excess pore pressure Δu was divided by the triaxial-based undrained shear strength s_u. Paniagua et al. define the pore pressure factor $N_{\Delta u}$ as the ratio between excess pore pressure and laboratory-determined undrained shear strength, and the Norwegian study evaluates both q_{net} and Δu as CPTu parameters for estimating s_u in clay [2].

The results from Figure 4.9 and Figure 4.22 indicate that $N_{\Delta u}$ has better consistency and less scatter with triaxial-based undrained shear strength s_u compared with N_{kt} for the investigated dataset. This consistency with undrained shear strength from triaxial data is further illustrated by calculating the calibrated shear strength in Figures 4.11-4.14 for Kärra and Figure 4.24 for Utby, where the undrained shear strength derived from different methods and parameters, while it compared with triaxial results.

For $N_{\Delta u}$, the same analysis was also carried out together with data from the Norwegian study. The result is shown in Figure 4.2.

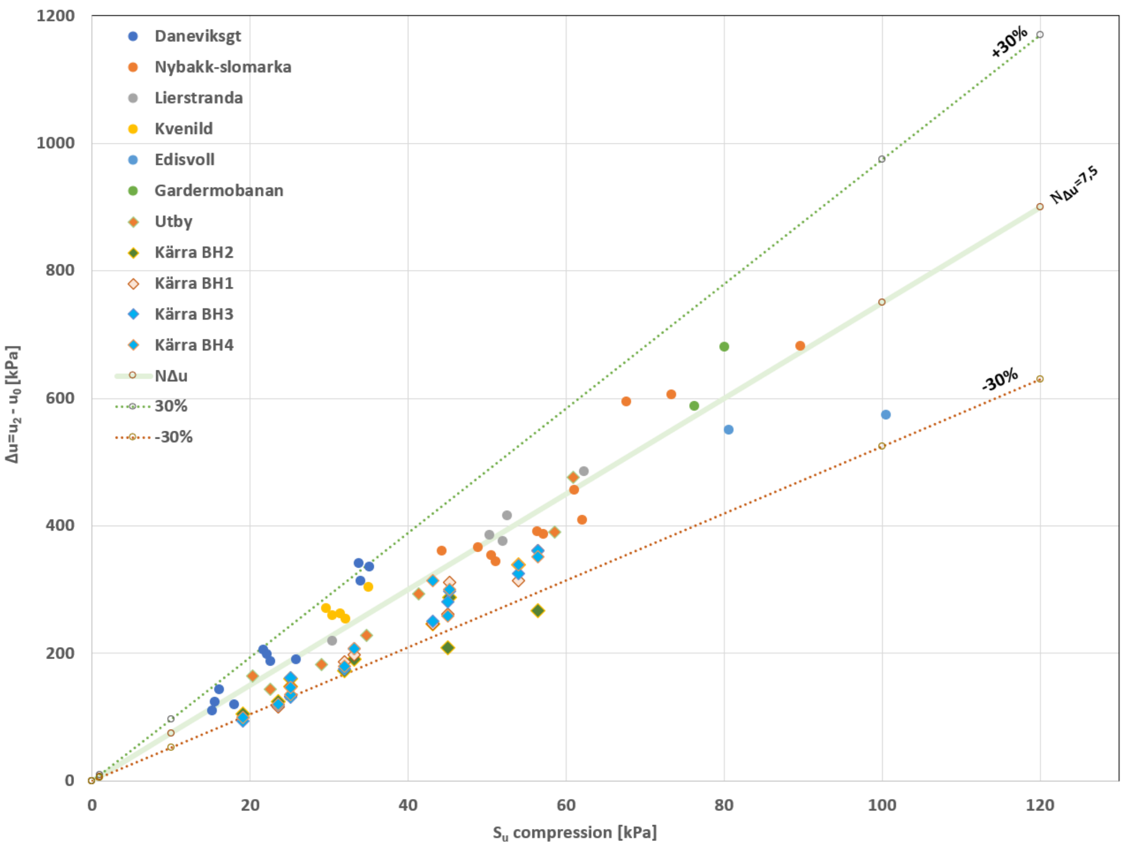

Figure 5.2: Excess pore water pressure Δu against undrained shear strength s_u from triaxial tests (Modified from [2]).

The results from the Norwegian study together with the Kärra and Utby test sites continue to follow the same general trend, as the Figure 5.2 shows the estimated $N_{\Delta u} = 7.5$ which is the same as the Norwegian study, while N_{kt} increased when the data from Gothenburg added to the dataset [2]. The comparison between the q_{net}–s_u relationship, and the Δu–s_u relationship indicates that the Δu shows less scatter and more consistency for the investigated data. This suggests that the pore pressure response may provide a useful basis for estimating undrained shear strength in the studied clay deposits. This agrees with previous CPTu studies showing that excess pore pressure can be an important parameter for estimating undrained shear strength in clay [2, 5]. However, as mentioned previously, these data are calibrated locally and should not be applied as general correlations without validation.

5.3 Evaluation of undrained shear strength from CPTu measurment

One of the main objectives of this study was to evaluated how CPTu parameters q_{net} and Δu are correlated to triaxial-based undrained shear strength s_u. Since the both cone factors N_{kt} and $N_{\Delta u}$ are normalized by the same triaxial undrained shear strength s_u, it will facilitate the evaluation of how the cone resistance and pore pressure response are correlated to the undrained shear strength.

The results of this study indicate that the calibrated undrained shear strength obtained from $N_{\Delta u}$ correlates better with the triaxial results than the undrained shear strength derived from N_{kt} for the investigated dataset. This became particularly clear after the assessment of different undrained shear strength profiles from test sites in Gothenburg, as shown in Figures 4.11-4.14 and 4.24. The use of data from different test sites made it possible to observe the spread between different CPTu soundings, and the results from different boreholes followed a similar trend. This strengthens the reliability of the observed correlation between pore pressure response and undrained shear strength.

The comparison between the Kärra and Utby test sites highlights the importance of data availability and site-specific conditions in CPTu interpretation. Even though the same analysis procedure was used for both sites, the result for Utby had weaker correlation to the triaxial data due to a more limited dataset. This supports the interpretation that site-specific calibration is important, but that the reliability of the calibration depends strongly on the amount and quality of available data. Previous CPTu studies also emphasize that empirical CPTu correlations may show scatter and should be interpreted as site-dependent correlation [2, 5].

Some uncertainty may arise from the calculation of CPTu-derived parameters. The net cone resistance q_{net} depends on the corrected cone resistance q_t and the total vertical stress σ_{v0}. The excess pore pressure Δu depends on the measured pore pressure u_2 and the assumed or interpreted in-situ pore pressure u_0. Therefore, assumptions related to unit weight, groundwater conditions and stress calculations can influence both N_{kt} and $N_{\Delta u}$.

Finally, the results are based on local calibration for the Kärra and Utby test sites. The calibrated values should therefore not be interpreted as general cone factors for all clay deposits. Instead, the results should be seen as site-specific relationships between CPTu parameters and triaxial-based undrained shear strength. This is consistent with previous studies, where CPTu correlations have been shown to depend on soil type, site conditions and the reference dataset used for calibration [2, 5].
Overall, the results show that the use of triaxial data as reference values has a strong influence on the interpretation of CPTu-based cone factors. The locally calibrated values of N_{kt} from Kärra and Utby were lower than the preliminary value based on general recommendations. This indicates that using a general value without calibration may overestimate or underestimate the undrained shear strength depending on the site conditions and the selected cone factor. The main outcome is therefore that site-specific calibration is important when CPTu is used to estimate undrained shear strength in clay. The comparison between N_{kt} and $N_{\Delta u}$ shows that both parameters provide useful information, but for this dataset the pore pressure-based approach gave a more consistent correlation with triaxial-based s_u. The results should therefore be viewed as a site-specific evaluation rather than a general recommendation for all clay deposits.

6 Conclusion and recommendation

This study investigated the CPTu cone factors N_{kt} and $N_{\Delta u}$ for two sites near Gothenburg, Kärra and Utby.

The CPTu-based soil classification showed that the investigated profile mainly consists of clay to silty clay. At the Kärra test site, the soil classification from CPTu boreholes showed a relatively consistent soil profile, with a trace of organic soil composition with clay in the upper layers. At Utby test site, the classification also indicated clay to silty clay as dominant soil type, and it was based only on one boreholes.

The reference undrained shear strength s_u, was evaluated from anisotropically consolidatd triaxial tests sheared in compression (CAUC) on STII piston samples at selected depths. Despite rather homogeneous conditions at each test site, the evaluated N_{kt} and $N_{\Delta u}$ values showed some variation in depth and with location of the CPTu borehole. Therefore, the N_{kt} or $N_{\Delta u}$ should not be treated as a single constant value without a site specific calibrating. A regression analysis provided a representative value for N_{kt} and $N_{\Delta u}$ for each site. The comparison between the Kärra and Utby test sites showed that the number of available CPTu boreholes and laboratory tests impacts the reliability of the interpretation. Although general literature values for N_{kt} and $N_{\Delta u}$ may be used as a first estimate the results from this study show that a local calibration can further optimize the design values for a more cost effective project. Therefore, CPTu-based interpretation of undrained shear strength should preferably be calibrated against site-specific laboratory data.

Finally, this study did not investigate the design framework in which these strength values will be used. Characteristic values for the undrained shear strength from geotechnical Site Investigation should be assessed by the engineer as they are not equal to the 'best fit'. Rather the characteristic values should reflect actual site conditions and any detrimental impact on the geotechnical design.

6.1 Recommendation

Based on the results and limitation of this study, the following recommendation are proposed:

Calibration with site-specific laboratory data is recommended. CPTu-based cone factors should be calibrated against high-quality laboratory tests, preferably triaxial test (CAUC/CAUE) or direct simple shear(DSS).

Both net cone resistance q_{net} and excess pore water pressure Δu should be evaluated when interpreting the undrained shear strength s_u from CPTu data. Since N_{kt} and $N_{\Delta u}$ are based on different CPTu parameters, they can provide complimentary information about the soil behavior.

When available use multiple CPTu boreholes, to improve the reliablity of the cone factors derived for a specific site.
Match CPTu data carefully with laboratory sample depth. The method of averaging CPTu data over 100 mm interval corresponding to the triaxial sample length is recommended for similar studies. This approach reduces the influence of single point fluctuations and give a more representative comparison between the laboratory data and filed data.

Include soil classification before conducting the analysis. CPTu-based soil classification recommended be carried out before evaluating the N_{kt} and $N_{\Delta u}$. This approach ensure that the interpreted cone factors are linked to the correct soil type and the assumption value for calculating different parameters is valid.

Future studies are recommended to include additional triaxial laboratory tests and CPTu data. This would enable a more comprehensive evaluation of lateral variability across the site, enhance the reliability of the analytical results, facilitate the identification of potential outliers within the dataset and reduce the uncertainty within the analysis.

Bibliography

[1] P. K. Robertson and K. L. Cabal, "Guide to Cone Penetration Testing." Gregg Drilling LLC, Seventh Edition, 2022.

[2] P. Paniagua, M. D'Ignazio, J.-S. L'Heureux, T. Lunne, and K. Karlsrud, "CPTU Correlations for Norwegian Clays: An Update," *AIMS Geosciences*, vol. 5, no. 2, pp. 82–103, 2019.

[3] R. Larsson, "CPT-sondering: Utrustning – utförande – utvärdering," SGI Information 15, Statens geotekniska institut, Linköping, Sweden, 2015.

[4] P. W. Mayne and J. Peuchen, "Evaluation of CPTU Nkt Cone Factor for Undrained Strength of Clays," in *Cone Penetration Testing 2018* (M. A. Hicks, F. Pisanò, and J. Peuchen, eds.), (Delft, The Netherlands), Delft University of Technology, 2018.

[5] J. Selänpää, B. Di Buò, M. Haikola, T. Länsivaara, and M. D'Ignazio, "Evaluation of Existing CPTu-Based Correlations for the Undrained Shear Strength of Soft Finnish Clays," in *Cone Penetration Testing 2018* (M. A. Hicks, F. Pisanò, and J. Peuchen, eds.), (Delft, The Netherlands), pp. 571–576, Delft University of Technology, 2018.

[6] V. Thakur *et al.*, "Landslide in sensitive clays – from research to implementation," in *Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering*, (Seoul), 2017.

[7] J.-S. L'Heureux *et al.*, "Landslides in sensitive clays: From geosciences to risk management," in *Landslides in Sensitive Clays*, Springer, 2013.

[8] Göta älvutredningen, "Skredrisker i Göta älvdalen i ett förändrat klimat: Slutrapport. Del 2 – kartläggning," tech. rep., Statens geotekniska institut (SGI), Linköping, 2012. GÄU 2009–2011.

[9] T. Lunne, P. K. Robertson, and J. J. M. Powell, *Cone Penetration Testing in Geotechnical Practice.* London, United Kingdom: Blackie Academic & Professional, 1997.

[10] R. Sandven, "Influence of Test Equipment and Procedures on Obtained Accuracy in CPTU," in *Proceedings of the 2nd International Symposium on Cone Penetration Testing*, (Huntington Beach, California, USA), 2010.

[11] R. Larsson, G. Sällfors, P.-E. Bengtsson, C. Alén, U. Bergdahl, and L. Eriksson, "Skjuvhållfasthet – utvärdering i kohesionsjord," SGI Information 3, Statens geotekniska institut, Linköping, Sweden, 2007.

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