Doctoral theses at NTNU, 2021:320

Annika Bihs

Investigation of a Coarse Silt Deposit by Varied Rate CPTU Testing

Doctoral thesis

NTNU
Norwegian University of Science and Technology
Thesis for the Degree of Philosophiae Doctor
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Preface

The present PhD study is carried out at the Geotechnical Engineering Group at the Department of Civil and Environmental Engineering at the Norwegian University of Science and Technology (NTNU), Trondheim, Norway. The work is presented as a collection of journal and conference papers complemented by a summary of the main findings. Professor Steinar Nordal (NTNU) was the main supervisor whereas Professor Mike Long (UCD) was the co-supervisor.

The study was financed from the Faculty of Engineering Science at NTNU. The evaluation committee consisted of Professor Paul W. Mayne as first opponent from Georgia Institute of Technology (USA), Professor Laura Tonni as second opponent from University of Bologna (Italy) and Professor Rao Martand Singh as administrator from NTNU.

Abstract

Well-explored methods for the interpretation of CPTU and laboratory tests for clay and sand materials exist, assuming undrained and drained conditions, respectively. The determination of design parameters in silt is not straightforward due to amongst other factors the presence of an unknown degree of partial drainage during CPTU testing which complicates the interpretation and influences the results. The geotechnical community has for a long time now attempted to establish an international acknowledged interpretation framework for intermediate soils such as silts, that often neither behave undrained nor drained but experience partial drainage during standard rate CPTU tests. In addition, silts are normally not homogeneous but have a rather complicated micro-layering that lead to a scattered and complex structure. The average engineering properties cannot be predicted without extensive testing.

Even though recent research draws more attention to the behaviour of these soils both in the field and in the laboratory, there still exist an absence of a comprehensive and basic engineering database of silts. The focus needs to be on the influence of the recognized partial drainage on the interpreted design parameters and methods needs to be developed on how to account for this. The aim of the present PhD study is to broaden the knowledge of silt behaviour both in the laboratory and in the field and to contribute to the improvement of a more confident interpretation framework. In particular, recommendations are needed on how to consider the influence of partial drainage on the determination of design geotechnical parameters.

To study the drainage behaviour of silt, an extensive field and laboratory research program was performed at the NTNU. The work was undertaken at the established Halsen-Stjørdal test site which is located north of Trondheim and consists of a 10 m thick natural, low plastic silt deposit. Alongside carrying out several standard CPTU tests, various investigations were conducted with penetration rates varying between 0.5 mm/s and 200 mm/s to cover the whole spectrum of drainage conditions. In addition, more than 40 dissipation tests were analyzed, that provide further insight into the development of pore pressures and drainage conditions at the site. To study the material behaviour of the silt, soil samples from the silt layer were taken and analyzed in the laboratory facilities at NTNU.

The laboratory analysis of the soil samples emphasize the difficulties of handling the material in the laboratory and confirm the importance of improving the methods for sampling and quality assessment and hence the confidence in the geotechnical design parameters. Recommendations are made on how to select consolidation and strength parameters in silt. Nevertheless, sample disturbance is likely to occur and may influence the results.

Varied rate CPTU tests show a strong rate dependency of the measured response on the penetration rate, typically with decreasing cone resistance and increasing negative excess pore pressure for increasing penetration rates. For many silts, the drainage conditions yield partially drained conditions during standard penetration. For high rate CPTU tests, the well-known N_{kt} approach reveals the most promising results for the undrained shear strength. For slow rate CPTU tests, the drained strength parameters are obtained using the NTH method. If standard penetration rate CPTU results are used for the interpretation, the undrained shear strength is overestimated whereas the drained friction angle is underestimated. Consolidation analysis of the measured dissipation data confirms the rate dependency on the established soil parameters. Dissipation data and consolidation analysis confirm drained behaviour after slow penetration tests whereas the faster tests show partial consolidated behaviour. If ignored, the presence of partial consolidation influences the interpreted coefficient of consolidation and needs to be accounted for to prohibit an underestimation of this parameter. To prevent misinterpretation of geotechnical design parameters, it is necessary to carry out additional varied rate tests in deposits where partial drainage is likely to occur.

The results and findings from the present PhD study on a natural silt deposit highlight the challenges when dealing with silty materials and broaden the database internationally. In particular, the research emphasizes the importance of recognizing and accounting for partial drainage when interpreting CPTU tests in silts. It is hoped that this study may contribute to improved practice for how to establish realistic strength and consolidation parameters for silts enabling safe and economical designs.

Acknowledgments

I would like to express my sincere gratitude to my two supervisors Professor Steinar Nordal (NTNU) and Professor Mike Long (UCD) who encouraged, guided, supported and inspired me through the years of this PhD. Thank you for all the meetings, discussions, help and proofreading the papers and manuscripts and answering all my questions. You gave me the confidence that I would be able to finish this work. I would not have been able to do this work without you.

A warm thank goes to the whole staff at the NTNU laboratory and field section. Especially I would like to thank Jan Jønland and Gunnar Winther for the endless hours we spend together in the laboratory and outside in the field. I would not have been able to carry out all the tests without your expertise, patience and support. Per Asbjørn Østensen, Frank Stæhli and Tage Westrum are thanked for helping me in designing and building the pore pressure cone device. This project would not have been possible without your creativity and helpfulness. Furthermore, I would like to thank the whole administration staff at IBM, NTNU. Marit Skjåk-Bræk and Maren Berg Grimstad are especially thanked for their support and helpfulness in all administrative matters. Thanks to Tone Måsøval Arntzen for helping me with patience throughout my several maternity breaks.

Many people accompanied me along the way and supported me during this time. Noel Boylan, Darren Ward and Mike Long are thanked for carrying out field work together with me both in Ireland and Norway. Furthermore, I would like to express my gratitude for using data from the Norwegian GeoTest Site project (NGTS) for the present research work.

I would like to thank all former and present PhD colleagues at the geotechnical division at NTNU. Thank you all for making this time memorable. My dearest friend and former office-mate Priscilla Paniagua is especially thanked for the fun times we had together at NTNU and for your support and encouragement in finishing this thesis. Tonje Eide Helle, I am grateful for the time we spend together at NTNU and all the walks in the forest we made over the last two years. You encouraged and supported me whenever things went difficult. Ivana Anušić, thank you so much for your support during finishing this PhD. The talks during our morning coffees made things so much easier! It is great that we are in the same office again!

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Contents

List of publications

List of papers appended in the thesis:

Paper 1 Bihs A, Gylland A, Long M, Nordal S and Paniagua P. 2018. "Effect of Piezocone Penetration Rate on the Classification of Intermediate Soils." In *Cone Penetration Testing 2018 (CPT18)*, vol.4, edited by Pisanò & Peuchen Hicks, 143-149. Delft University of Technology, The Netherlands: CRC Press.

Paper 2 Bihs A, Long M and Nordal S. 2020. "Geotechnical Characterization of Halsen-Stjørdal silt. Norway." *AIMS Geosciences 6,* no. 3 (September): 355-377. https://doi.org/10.3934/geosci.2020020

Paper 3 Bihs A, Long M and Nordal S. 2021. "Evaluation of Soil Strength from Variable Rate CPTU Tests in Silt." *Geotechnical Testing Journal*: (under review).

Paper 4 Bihs A, Long M, Paniagua P and Nordal S. 2021. "Consolidation Parameters in Silts from Varied CPTU Tests." (submitted).

Additional papers authored or co-authored by the candidate but not included in the thesis:

1. Bihs A, Long M, Marchetti D and Ward D. 2010. "Interpretation of CPTU and SDMT in Organic, Irish Soils." In *Second International Symposium of Cone Penetration Testing (CPT10),* vol. 2, edited by P. K. Robertson & P. W. Mayne, 257-264. Wisconsin, USA: Omnipress.

2. Paniagua P, Bihs A and Nordal S. 2011. "Interpretation of Cone Penetration Tests in Clay by Finite Element Simulations." In *9th Euroconference on Rock Physics and Geomechanics*. Trondheim, Norway.

3. Paniagua P, Bihs A and Nordal S. 2012. "Improved Finite Element Simulations for Interpretation of Cone Penetration Test Results." In *Nordic Geotechnical Meeting (NGM).* vol. 1, edited by Danish Geotechnical Society, 427-434. Copenhagen, Denmark: DGF.

4. Bihs A, Boylan N, Long M and Nordal S. 2012. "Interpretation of Consolidation Parameters from CPTU Results in Sensitive Clays." In *Geotechnical and Geophysical Site Characterization 4 (ISC4),* vol. 1, edited by R. Q. Coutinho and P. W. Mayne, 227-234. London, U.K.: Taylor and Francis.

5. Emdal A, Long M, Bihs A, Gylland A and Boylan N. 2012. "Characterization of Quick Clay at Dragvoll, Trondheim, Norway." *Geotechnical Journal of the SEAGS & AGSSEA 43,* no. 4 (December): 11-23.

6. Boylan N, Bihs A, Long M, Randolph MF and Nordal S. 2016. "Characterization of a Norwegian Quick Clay using Piezoball Penetrometer." In *Geotechnical and Geophysical Site Characterization 5 (ISC5),* vol. 1, edited by Acosta-Martinez & Kelly Lehane, 1193-1198. Sydney, Australia: Australian Geomechanics Society.

Declaration of authorship

The author of the thesis is named as the first author for all four publications appended to this thesis. The author of the thesis was responsible for preparing and writing the manuscripts, planning and performing the field and laboratory investigations as well as carrying out the analysis and interpretation of the data.

Mike Long, as co-supervisor, and Steinar Nordal as main supervisor advised and guided the first author on structuring the papers and discussing the results in all appended papers as well as proofreading all manuscripts.

Anders Gylland, as second author of Paper 1, helped the author in structuring the results and discussing the findings as well as proofreading the manuscript.

Priscilla Paniagua, as fifth author of Paper 1, was responsible for carrying out the laboratory rate study on Vassfjellet silt and assisted the first author in presenting and discussing the results for the manuscript. Furthermore, Priscilla Paniagua, as third author of Paper 4, provided data from the NGTS Halden silt site in Norway as well as valuable discussion and guidance on the interpretation of the results.

Declaration of contribution to the appended papers

The thesis author was responsible for planning and executing the field and laboratory work from the Halsen-Stjørdal test site. Furthermore, the thesis author was responsible for analyzing the data and writing the full manuscripts.

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Nomenclature

Latin letters

a	exponent number
B_q	pore pressure parameter [-]
CPTU	cone penetration test with pore pressure measurements
c'	effective cohesion [kPa]
CAUC	anisotropically consolidate undrained compression test
C_{cw}	virgin compression index
c_h	horizontal coefficient of consolidation [m^2/s]
CRS	constant rate of strain test
C_{rw}	recompression index
C_u	coefficient of uniformity
c_v	vertical coefficient of consolidation [m^2/s]
c_{v0}	vertical coefficient of consolidation at σ_{v0}` [m^2/s]
d	diameter of pushing probe [mm]
e	void ratio [-]
e_0	initial void ratio [-]
F_r	friction ratio [-]
f_s	measured friction [kPa]
G	shear modulus [MPa]
G_{50}	shear modulus at 50 % mobilized strength [MPa]
IP	plasticity index [%]
I_R	rigidity index [-]
k	permeability [cm/s]
K_0	coefficient of earth pressure at rest
m	modulus number
M	constrained oedometer modulus [MPa]
NCL	normal compression line
NGF	Norwegian Geotechnical Society
NGTS	Norwegian Geo-Test Sites
N_{ke}	effective cone resistance number
N_{kt}	cone resistance number based on cone resistance
$N_{kt,std}$	cone resistance number for standard penetration rate
$N_{kt,undrained}$	cone resistance number for undrained penetration rate
OC	overconsolidated
OCR	overconsolidation ratio [-]
p_c'	effective preconsolidation stress [kPa]
q_n	net cone resistance [MPa]
q_t	cone resistance [MPa]
Q_{ref}	reference normalized cone resistance [-]
$Q_{drained}$	drained normalized cone resistance [-]
Q_t	normalized cone resistance [-]
r	radius of pushing probe [mm]
U	normalized excess pore pressure [-]
u	pore pressure [kPa]
u_0	hydrostatic pore pressure [kPa]
u_2	pore pressure measured at the u_2 location [kPa]
u_{oct}	octahedral pore pressure component [kPa]
u_{shear}	shear pore pressure component [kPa]

SPM	stress path method
S_t	sensitivity [-]
s_u	undrained shear strength [kPa]
t	time [s]
T*	modified time factor [-]
t_{50}	time for 50 % of consolidation [s]
v	penetration rate [mm/s]
V	normalized penetration rate [-]
$\bar{V}_h$	normalized horizontal penetration rate [-]
ΔW_{oed}	cumulative work increment

Greek letters

β	plastification angle [°]
ε	strain [%]
ε_{vol}	volumetric strain [%]
σ_a	reference stress [kPa]
σ_r'	effective radial stress [kPa]
σ_v'	effective vertical stress [kPa]
σ_{v0}'	effective in-situ vertical stress [kPa]
τ	shear stress [kPa]
φ'	effective friction angle [°]
$\varphi_{drained}'$	effective friction angle for drained penetration rate [°]
φ_{std}'	effective friction angle for standard penetration rate [°]

1 Introduction

1.1 Background and motivation

The determination of geotechnical parameters from CPTU tests in silt is complex amongst other factors due to the unknown presence of partial drainage during a standard penetration rate of 20 mm/s. Furthermore, the soil structure of natural silt deposits is often affected by small layers and lenses of sand and clay which result in irregular soil conditions and complicate the interpretation of the rather scattered test results. Interpretation methods for clay and sand exist which are well established and based on the generally accepted assumption of undrained conditions in clay materials whereas a drained situation is adopted for sands (Lunne et al. 1997b). Unfortunately, up to now, little guidance is given to practicing engineers on how to interpret soil parameters from field investigation tests in silt deposits which often show a partially drained behaviour.

Establishing reliable geotechnical design parameters in silty soils from laboratory test results is complicated due to the layered structure of many silt deposits. Retrieving high quality soil samples for laboratory investigations is challenging and often not successful due to amongst others low cohesion and a complex soil structure, which is rather sensitive to disturbances (Long et al. 2010). Especially in loose, low plastic and coarse silt deposits, the handling of the material in the laboratory can be challenging. Thus, achieving high quality data in these soils is difficult. The lack of a standardized framework for the evaluation of sample quality for silts complicates the assessment of the determined soil data.

Several studies exist dealing with rate dependency of silts conducted under controlled conditions in the laboratory using calibration chambers or centrifuges (e.g. Silva and Bolton 2005; Schneider et al. 2007; Jaeger et al. 2010; Paniagua and Nordal 2015). But up to now only a few research projects exist on undisturbed silt deposits dealing with the rate dependency of the measured response due to partial drainage and analyzing geotechnical design parameters in these soils. In Norway, data from three different research test sites have been published dealing with the behaviour of silts during CPTU tests as well as the determination of sample quality and the interpretation of laboratory test results. Yet only the recent study carried out by Blaker et al. (2019) deals with the effect of carrying out varied rate tests. Sandven (2003) carried out an extensive field and laboratory study on a natural silt close to the present research site including standard CPTU and dissipation tests. Long et al. (2010) analyzed the behaviour of a silt deposit situated in the western part of Norway close to Bergen (Os). Various in-situ and laboratory tests were used including standard CPTU tests and the results of different soil samplers were investigated. Recently Blaker et al. (2019) and Blaker (2020) published a comprehensive study from the Norwegian Geo-Test Site (NGTS) south of Oslo in Halden. The test program included geophysical as well as geotechnical in-situ investigation methods and an intensive laboratory testing program. Various varied rate penetration CPTU tests supported by dissipation tests were conducted in order to study the rate dependency of the silt (Paniagua et al. 2016; Carroll and Paniagua 2018).

All studies carried out emphasize on the lack of a practicable framework for establishing soil parameters and determining sample quality in silty materials. In the absence of well-established correlations in silts, engineers tend to apply clay or sand based models which often reveal questionable results in silty materials (Andresen and Kolstad 1979; Lunne et al. 1997a). In addition, neglecting the presence of partially drainage may lead to an under- or overestimation of the geotechnical parameters resulting in possible unsafe design conditions (Long 2007; DeJong and Randolph 2012).

The lack of understanding of the material behaviour of silty soils and hence a consistent interpretation framework for both field CPTU and laboratory data motivated the present research study carried out at NTNU. Figure 1-1 shows drilling at the research test site, located in Halsen-Stjørdal, close to Trondheim.

Figure 1-1 Picture from the Halsen-Stjørdal test site.

One aim is to broaden the database of natural silt deposits. Furthermore, the identification of partial drainage and the influence on the interpretated soil parameters needs more attention in these soil conditions. Further research is warranted in these soils to broaden the database and increase the knowledge on these soils. A validation of the chosen methods applied to silt is necessary to increase the confidence of selecting proper design parameters in deposits where partial drainage occurs for both research and projects for geotechnical design.

1.2 Scope and objectives

The scope of the present PhD project is to increase the knowledge on the behaviour of natural silt deposits. The study focuses on the interpretation of field CPTU and laboratory tests on a silt site in Halsen-Stjørdal, Norway. The main results are related to the present research silt site. However, results from a NGTS silt site are used to validate some of the findings of this study. The objectives of the present PhD work are as follows:

- Plan and carry out an extensive field and laboratory test program to gain better understanding and insight into the behaviour of silty soils. (**Paper 1 - 4**)

- Increase the knowledge of silt behaviour during standard penetration CPTU and laboratory tests as well as on how to interpret the results in these materials. Furthermore, determine and discuss sample quality in silt deposits. (**Paper 1 & 2**)

- Investigate the rate dependency of the interpretated CPTU parameters in the field on the penetration rate and study the influence of partial drainage on established undrained and drained shear strength parameters. (**Paper 3**)

- Broaden the experiences on how to interpret consolidation parameters in silts from both field CPTU dissipation tests and laboratory test results. (**Paper 4**)

- Investigate the influence of penetration rate and hence partial consolidation in silt deposits on the interpretation of consolidation parameters. Validate the results by applying the findings from the Halsen-Stjørdal test site to data from a well investigated NGTS research silt site. (**Paper 4**)

- Use the results obtained to make recommendations and provide guidance for future research and consultancy projects carried out in silt deposits where partial drainage is present. (**Paper 2 - 4**)

1.3 Structure of the thesis

The thesis is prepared as a collection of three international journal papers and one conference article. After the introduction, Chapter 2 presents a brief state of the art review concerning special properties of silty soils and challenges concerning laboratory and field work in silts. In addition, a summary is made of national and international research silt sites available in the literature and the outcome of these. Chapter 3 describes the present silt test site, including a detailed field and laboratory plan. Furthermore, Chapter 4 deals with the results achieved from the laboratory investigations including a discussion on sample quality in silt (Paper 1 - 4). Chapter 5 presents the CPTU field test results focusing on the rate dependency of the measured parameters (Paper 1 - 3). Chapter 6 presents the results concerning strength from CPTU tests in silt and the rate dependency of the undrained and drained strength parameters (Paper 3). Finally, Chapter 7 describes the results obtained from the consolidation study by analyzing the dissipation tests carried out at the present test site and validates the findings using data from the Halden test site. The work has been summarized in Chapter 8 and 9 where the main findings and conclusions are presented. Chapter 10 deals with recommendations and ideas for future research work. The relevant conference and journal papers can be found in the Appendix of the thesis.

2 Silt

2.1 Introduction

Silt is a soil type located between the grain sizes of clay and sand. According to the Norwegian Geotechnical Society (NGF) a soil deposit is defined as SILT if more than 45 % of the grains are between 0.002 mm and 0.06 mm and less than 15 % is clay (< 0.002 mm). Since silts are by definition a combination of different fractions of sand and clay, the behaviour of a silt deposit is difficult to predict without intensive laboratory testing. In Northern Europe, deposits of silt are often affected by surface erosion or flow slides (Shaoli et al. 2002) (Figure 2-1).

Figure 2-1 Surface sliding in silt slopes caused by a period of heavy rain (Sandven 2003).

Even though a significantly large part of present day infrastructure and geotechnical problems relate to silty soils, little research has been carried out on this special soil behaviour and a great need of an accredited interpretation framework has been needed for a long time. The identification of the degree of drainage during testing as well as the influence of partial drainage on the interpretated geotechnical parameters are factors of great importance when dealing with silty soils.

2.2 Laboratory work in silt

Due to the layered, complex and fragile soil structure of many natural silt deposits, high quality soil samples are very difficult to achieve. Little research exist on the interpretation of sample quality in silty soils and a accepted interpretation framework is not available. There is a general accepted risk of increasing the density of loose silts or decreasing the density of dense silts during the sampling process. These factors have been accounted for in the Norwegian Classification system for sampling (NGF 2013). When designing a sampling program for a silt site, large diameter samplers are generally preferred since these show less influence of sample disturbance in the core of the sample probe (Sandven 2003). Studies showed that when using reconstituted silt samples, it is nearly impossible to reestablish the particular in-situ conditions (Shaoli et al. 2002).

Carrying out a representative laboratory test in silt is challenging due to the natural layered structure. The scattered data from the individual soil samples challenge the estimation of representative soil parameters for these deposits. Silty soils may behave both contractive and dilative, depending on the actual stress and strain level. Since a certain degree of sample disturbance is likely to occur in silts, this might influence the results from the laboratory investigations and needs to be accounted for. Soil sample examples from the present research site are presented in Figure 2-2 and Figure 2-3 representing typical natural silt conditions. The figures show cross sections and parts of whole soil samples taken from the silt layer. The pictures clearly indicate the layered structure of the silt deposit, illustrating horizontal as well as vertical layers and lenses of coarser sandy material and small pockets of clay. Furthermore, cracks and holes inside and along the samples are visible which complicate the handling of the soil samples in the laboratory and the determination of representative soil parameters in these soils.

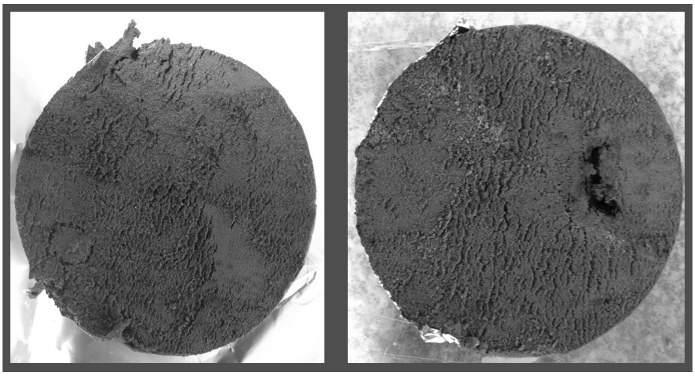

Figure 2-2 Cross section of soil samples taken in the silt layer at Halsen-Stjørdal.

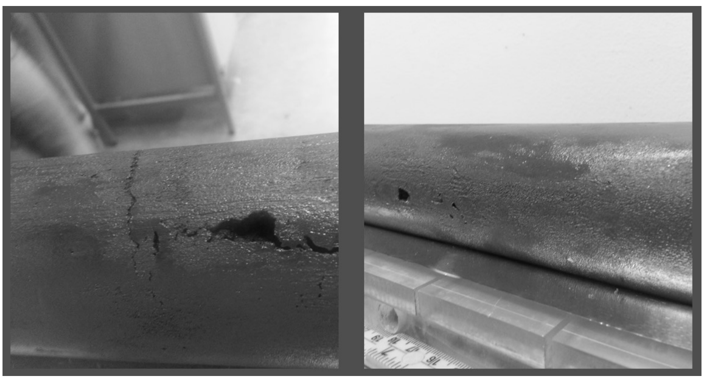

Figure 2-3 Sections of two soil samples from the silt layer at the Halsen-Stjørdal site.

2.3 CPTU in silt

The drainage conditions during a standard CPTU test are mainly dependent on the soil properties and the penetration rate used. Interpretation of the measured parameters has traditionally been conducted assuming either undrained or drained conditions. Drained conditions prevail for sand deposits whereas an undrained penetration is assumed for clay sites when the standard penetration rate of 20 mm/s is used.

When dealing with silty soils, engineers tend to apply interpretation methods made for either drained conditions (sands) or undrained conditions (clay). This assumption might work in situations where the standard CPTU test shows drained or undrained behaviour, and the geotechnical problem requires the interpretation of these parameters. However, this state and material behaviour is rarely the case for silts, even though grain size distributions might be close to the ones of clay or sand. For most of the cases, partial drainage occurs during a standard penetration test which complicates the interpretation of geotechnical design parameters and often the application of effective stress approaches is more appropriate (Senneset et al. 1989; Lunne et al. 1997b). During the process of analyzing CPTU data in silt deposits, assessing the degree of drainage that occurs under standard rate conditions is vital. Various varied rate penetration CPTU tests are needed in order to get an overview over the drainage condition at a specific site (Silva and Bolton 2005).

2.4 Overview over research sites on silt with varied rate CPTU tests

Only a few natural silt research sites have been investigated throughout the world. Campanella et al. (1981) and Finke et al. (2001) were some of the first ones addressing the problem of partial drainage in silt deposits. Nevertheless, their research is limited to only a few rate tests and the interpretation of geotechnical parameters from laboratory tests. The awareness and need of a standardized framework for silts increased over the last 10 to 15 years. Therefore, work from several research silt sites have been published recently (Schnaid et al. 2007; Kim et al. 2008; Tonni and Gottard 2019) and including dissipation tests after varied penetration rates (Suzuki et al. 2013; Holmsgaard et al. 2016; Garcia Martinez et al. 2016; Krage et al. 2016). Most of the project results published deal with the interpretation of soil parameters from both field and laboratory tests and investigate the rate dependency of these sites by conducting varied rate CPTU tests.

Table 2-1 shows a summary of the CPTU research test sites carried out on natural silt deposits published in literature and dealing with the rate dependency. The ones analyzing the consolidation behaviour by means of dissipation tests are noted. Even though three different silt sites have been extensively investigated in Norway over the last two decades, little focus has been on the interpreted parameters and their influence on the determination of geotechnical design values (Sandven 2003; Long et al. 2010; Blaker et al. 2019). Although the published results increase the knowledge and awareness of the importance of selecting careful geotechnical design parameters in silt and considering partial drainage, the geotechnical community is still lacking a standardized interpretation framework that allows practicing engineers a confident selection of soil design parameters. The purpose of the present research work was to help addressing some of these issues.

Table 2-1 Overview over research sites on silt with varied penetration tests.

Site name	Reference	Soil type	OCR (-)	IP (%)	I_R (-)	$V_{drained}$ (-)	$V_{undrained}$ (-)
McDonald's Farm, Canada	Campanella et al. (1981)	clayey silt				—	—
Opelika Test Site, U.S.	Finke et al. (2001)*	residual silt	—	—	—	—	—
Brazil	Schnaid et al. (2007)	silty tailing	NC	—	—	0.01 - 1	100
Indiana, U.S.	Kim et al. (2008)	clayey silt	—	—	—	0.05	10
Gingin/Bassendean, Australia	Suzuki et al. (2013)*	clayey, sandy silt	1-2	21	150	0.05	5
Dronninglund, Denmark	Holmsgaard et al. (2016)*	sandy silt	—	1.5-4.1	275	0.1 - 0.2	20 - 40
Po Valley, Italy	Garcia Martinez et al. (2016)*	clayey silt	1-2	0-8	130	1	30 - 50
Kornbloom B, U.S.	Krage et al. (2016)*	sandy silt	10-15	0-5	240	0.1 - 0.3	10 - 30
Halden, Norway	Blaker et al. (2019)*	clayey silt	1-1.3	6.6-9.3	126-147	—	—
Venice Lagoon, Italy	Tonni and Gottard (2019)*	clayey, sandy silt	1-2	5 - 15	—	—	—
Halsen-Stjørdal, Norway	present study *	clayey, sandy silt	1-2	0	200-250	0.2 - 0.3	40 - 50

* including dissipation tests

2.5 Section summary

- Silt deposits in Northern Europe are especially affected by surface erosion or flow slides.
- Little research has been carried out on natural silt deposits compared to sand or clay sites. Practicing engineers require a well-founded interpretation framework that allows a confident parameter selection for a safe design.
- Laboratory work in silty soils is complicated amongst others due to the layered and scattered structure and the presence of sample disturbance which impede the selection of representative soil parameters in these deposits.
- The interpretation of CPTU tests in silt deposits is complicated and not straightforward due to the presence of partial drainage during standard penetration.
- Applying undrained or drained interpretation methods in silt deposits may lead to over- or underestimation of the corresponding parameters.

- A few national and international research silt sites have been established over the last two decades dealing with the drainage behaviour of silts. Nevertheless, a standardized framework for testing and parameter determination is still missing, and conclusions are difficult to make. This highlights the complexity of the topic and the need to broaden the database of silts.

3 Test site Halsen-Stjørdal

3.1 Introduction

The research site Halsen-Stjørdal is situated in the Stjørdal valley about 35 km north-east of Trondheim in Norway. The ground conditions are characterized by a low plastic silt deposit, which is overlain by a stiff 4 m top layer. To ensure proper saturated pore pressure filters without losing saturation during CPTU testing, the top layer needed predrilling. The ground water table was at about 2.8 m depth.

3.2 Geology

Sveian (1995) studied the geological history of the area. During the Quaternary period, an enormous icecap covered Scandinavia. During the de-glaciation period, the icecap retreated and the Stjørdal valley was transformed into a deep, long fjord. Fine-grained particles such as silt and clay were transported by rivers of melt-water into the sea forming thick deposits of these sediments reaching 200–300 m of thickness over bedrock. This is typical for the Halsen-Stjørdal test site where the fine sediments are dominated by silt, with layers and pockets of clay and coarse sand forming a very irregular picture. Figure 3-1 shows a Quaternary map of the area, indicating that the test area consists mainly of river deposits.

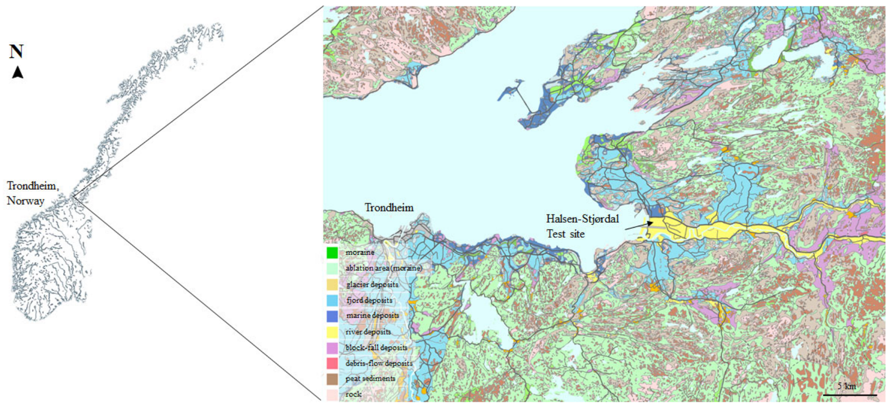

Figure 3-1 Quaternary map from NGU (2021).

However, there is no evidence that the Halsen-Stjørdal silt deposit has been overridden by any massive ice glacier since indications of glacial advances have only been found further up the Stjørdal valley. Geological studies carried out in the area indicate a normally to lightly overconsolidated (OC) soil deposit. Hence, one can expect no distinct pre-consolidation due to erosion of the soils in the area (Sveian 1995). The current site is about 500 m away from the well-known Halsen test site investigated by Sandven (2003).

3.3 Test Program

A field and laboratory testing program was assembled in order to characterize and examine the material behaviour of the silt deposit. CPTU tests were conducted following the international standard procedures according to ISO 22476-1 (ISO 2012). The tests were carried out down to a depth of 18 m by using a standard 35.7 mm friction cone and measuring the pore pressure at the u_2 location (Lunne et al. 1997b). Penetration rates were varied between 0.5 mm/s and 200 mm/s throughout the CPTU tests to study the drainage behaviour of the silt, covering drainage conditions varying from drained, partially drained to undrained. In addition, dissipation tests were performed at several predetermined depths of interest where the penetration was paused and the development of the pore pressure was measured over time. To avoid possible movement of the CPTU system during a dissipation test, the rods were clamped carefully during each test. The test site covers an area of about 11 m x 8 m and the different CPTU tests have been distributed evenly to characterize the rather irregular soil deposit. The test set up, including locations for the three boreholes, can be taken from Figure 3-2.

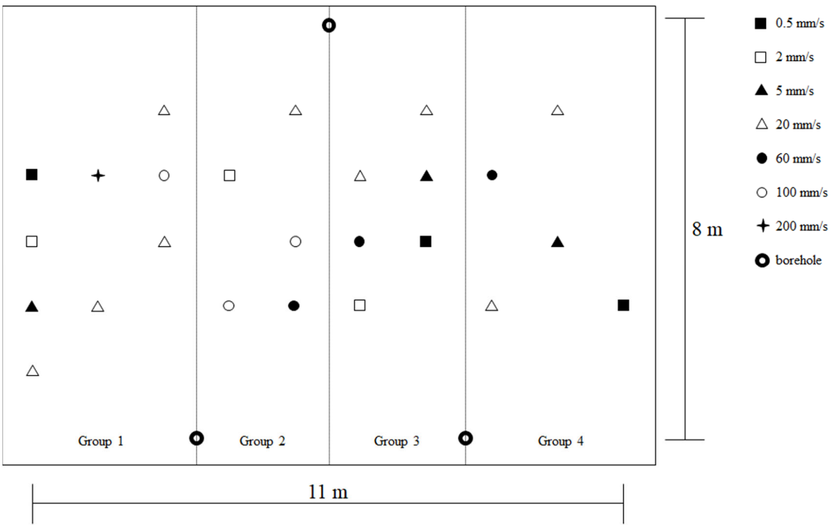

Figure 3-2 Test layout of varied CPTU tests and boreholes at Halsen-Stjørdal.

Furthermore, various samples were taken at 1 m intervals down to a depth of 13 m using thin walled 54 mm steel sample tubes in three different boreholes (Andresen and Kolstad 1979). The boreholes are spread over the test area to detect spacial variations of the soil deposit. All samples used for this project have been handled carefully during the sampling process and when taken back to the laboratory. To ensure the highest possible sample quality, all samples taken in the field were extruded in the laboratory within 24 hours of sampling time (Amundsen and Thakur 2018).

4 Laboratory results (Papers 1 - 4)

The laboratory investigation program was carried out at the facilities of the geotechnical division at NTNU, including basic index testing as well as oedometer and triaxial testing. The main findings and conclusions about the laboratory results from parts of Papers 1 to 4 are summarized in this section and can be find in the appended section of the thesis.

4.1 Index test results

Grain size distributions have been determined from the soil samples by using a hydrometer. Figure 4-1 shows a summation plot of some of the test results. In order to classify the soil type, the recommendations made by the Norwegian Geotechnical Society (NGF) have been used (NGF 2011). Most of the soil from the Halsen-Stjørdal test site consists of either sandy or clayey SILT with an average silt content of 55 %, with most of the particles falling into the coarse silt spectrum (e.g. 0.02 – 0.06 mm). The average soil grain density is about 2.66 g/cm^3 and the coefficient of uniformity (C_u) is 17.

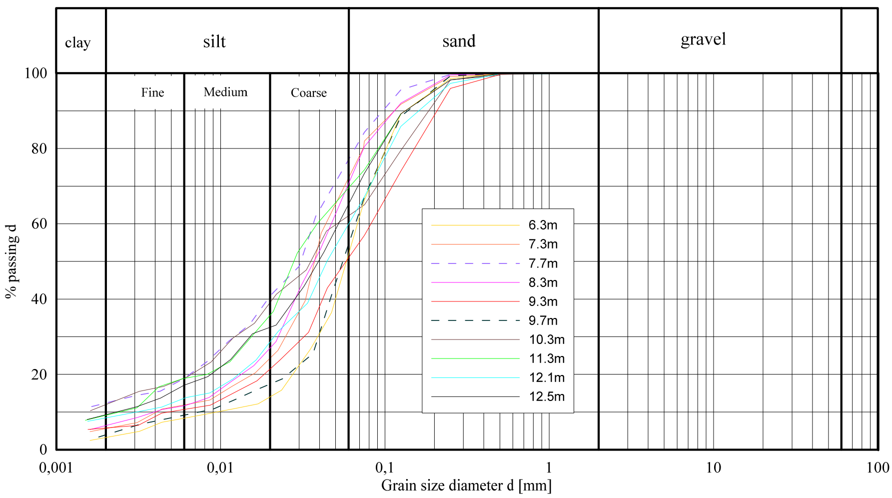

Figure 4-1 Summary of grain size distribution curves.

The basic material properties are summarized in Figure 4-2 indicating an upper and lower silt layer at 4 m and 8 m respectively, being normally to lightly overconsolidated. Due to the very low plastic and very coarse structure of the sediment, it was not possible to achieve results for the Atterberg limits. The natural water content does not vary significantly over depth with an average value of 25 %. Results from the bulk density measurements show an average value of 2.1 g/cm^3. Furthermore, sensitivity measurements (S_t) from the falling cone test have been included, indicating a medium sensitive silt with increasing values with depth (NGF 2011).

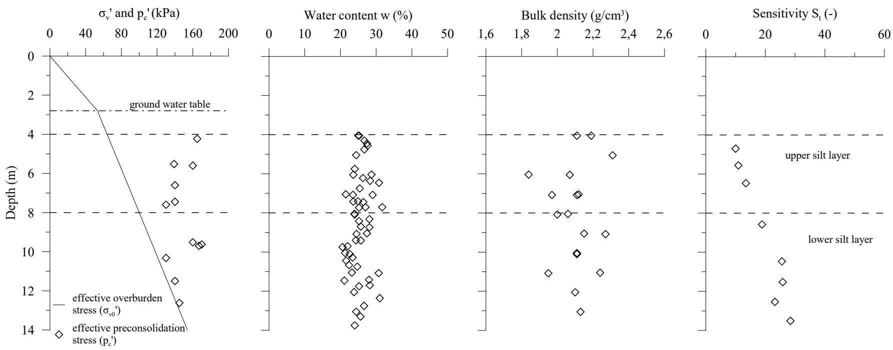

Figure 4-2 Basic soil parameters.

4.2 Oedometer tests

A series of oedometer tests were conducted on 2 cm thick specimens to study the vertical one dimensional compression behaviour of the silty material using the standard procedure proposed by Sandbaekken et al. (1986). Constant rate of strain (CRS) tests with axial strain rates varying between 2%/h and 5%/h were carried out on samples taken from depths between 4 - 14 m. The pore pressure was measured at the bottom of the specimen allowing for a single-sided drainage.

The interpretation of 1D compression curves for silty soils is difficult and up to now no international accredited framework exist. Figure 4-3 shows the results for the upper and lower silt layer respectively. The log vertical effective stress (σ_v\`) versus void ratio (e) plots show a typical behavior for silt materials (Figure 4-3 (A & D)). The graphs are of rounded nature and no distinct preconsolidation stress can be identified which might be partly due to the silty nature of the material itself and partly due to the influence of sample disturbance (Boone 2010; Long et al. 2010). The relative flat σ_v\` versus e plots in the upper silt layer show little change in void ratio during further compression which is an indication for a possible experienced densification. Furthermore, the curves continue parallel with increased $\sigma_v$\`, not converging to a unique Normal Compression Line (NCL) which is a typical behaviour for many silts. This phenomenon has been discovered by several researchers working with silts, suggesting that complex factors govern the non-convergent behaviour of silty soils (Cola and Simonini 2002; Donohue and Long 2010; Shipton and Coop 2012).

In order to calculate modulus numbers (m) and constrained modulus (M) values for the silt, the tangent concept has been applied to the data set which defines M as the ratio of $\Delta\sigma_v$\` over strain ($\Delta\varepsilon$). M can be expressed with the following general equation by means of m, the reference stress (σ_a) and an exponent number (a) which varies according to Janbu (1963) with soil type:

$$M = m\sigma_a \left[\frac{\sigma`}{\sigma_a}\right]^{1-a} \tag{2}$$

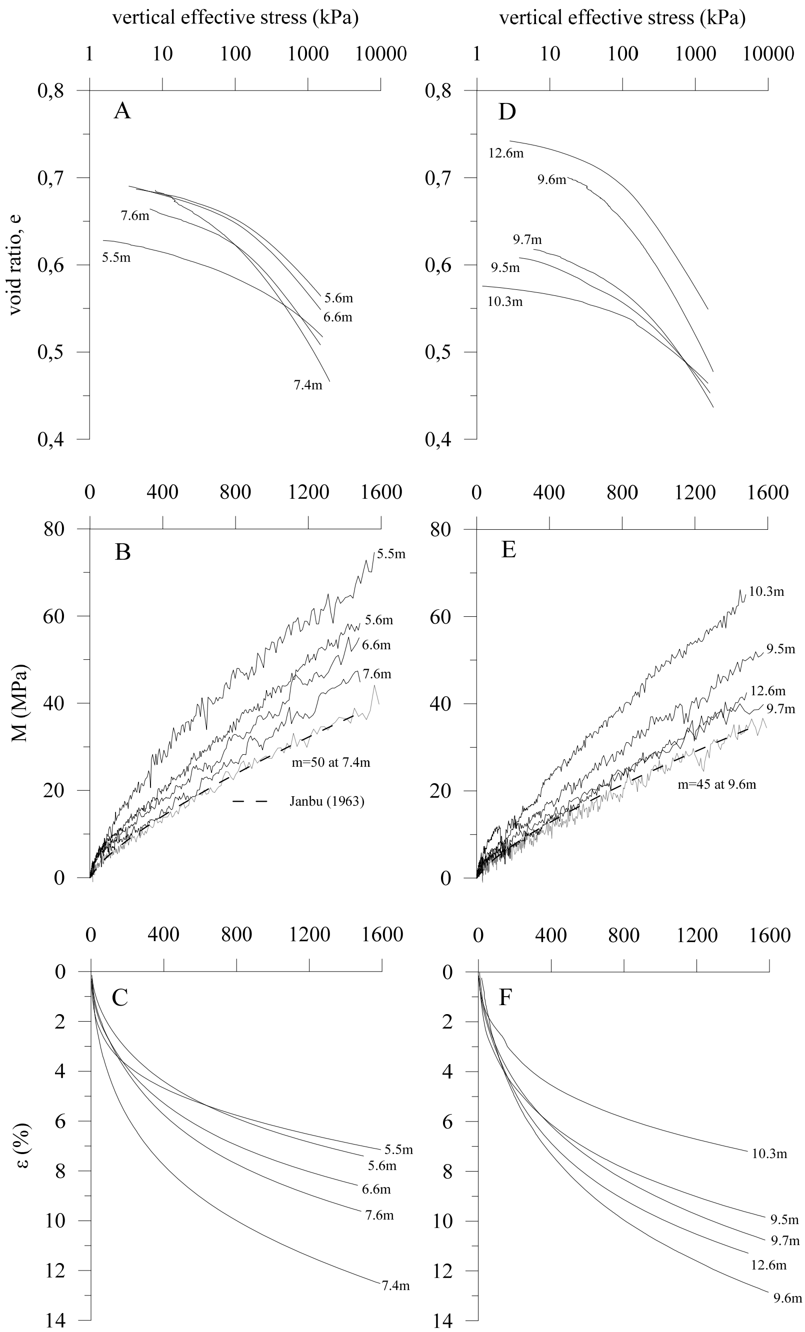

Figure 4-3 Oedometer test results: σ_v\` versus e, M and ε: (A-C) 4 - 8 m & (D-F) 9 - 14 m.

For the present test site, an exponent number a of 0.25 gave the best fit to the measured data set, resulting in a stress exponent of 0.75. Figure 4-3 (B & E) shows a steady increase of constrained modulus (M) with increasing σ_v` and no constant behaviour in the lower stress ranges or reduction around the preconsolidation stress (p_c’) as would be seen for clay soils. Janbu (1985) stated that this observed behaviour is characteristic for silty soils. The results obtained agree well with findings from other silt sites in Norway using the proposed method and confirm the gradual increase of M with increasing σ_v` of silty soils under compression (Sandven 2003; Long et al. 2010; Blaker et al. 2019). Modulus numbers for the present silt vary in the range of 45 to 110 with relatively low porosities between 36 % and 43 % which is characteristic for Norwegian silts and represent the lower bound of the silt range suggested by Janbu (1963).

Existing methods to interpret p_c’ often do not work in silty materials due to sample disturbance, the particular compression behaviour and the characteristic shapes of the curves as mentioned above (Casagrande 1936). Janbu (1963) claims that the constrained modulus curves cannot be used due to the lack of a significant change of behaviour in the preconsolidation stress region as for clayey materials. Recent studies carried out reviewed the application of existing methods used for clay materials to silty soils (Grozic et al. 2003; Boone 2010). Since a reliable method for determining p_c’ in silts is not available, the method proposed by Becker et al. (1987) based on the work criteria has been used for the present study, even though originally invented for clay soils, often showing a change of behaviour around p_c’ which is often not the case in silty soils (Figure 4-3). Figure 4-4 presents results of OCR with depth, yielding values between 1 - 2 which is characteristic for a normally to lightly overconsolidated material and showing a decreasing trend with depth. The results facilitate the geological history of the location and coincide well with previous studies carried out in the area (Sandven 2003). Figure 4-5 shows the oedometer test results for the silt layer in the low stress region, including the obtained results for p_c` from Becker et al. (1987).

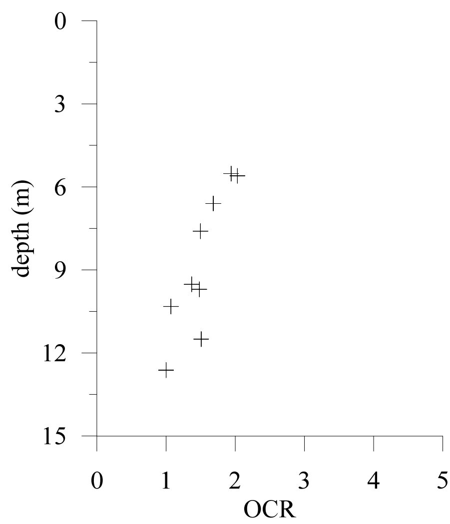

Figure 4-4 OCR for Halsen-Stjørdal after Becker et al. (1987).

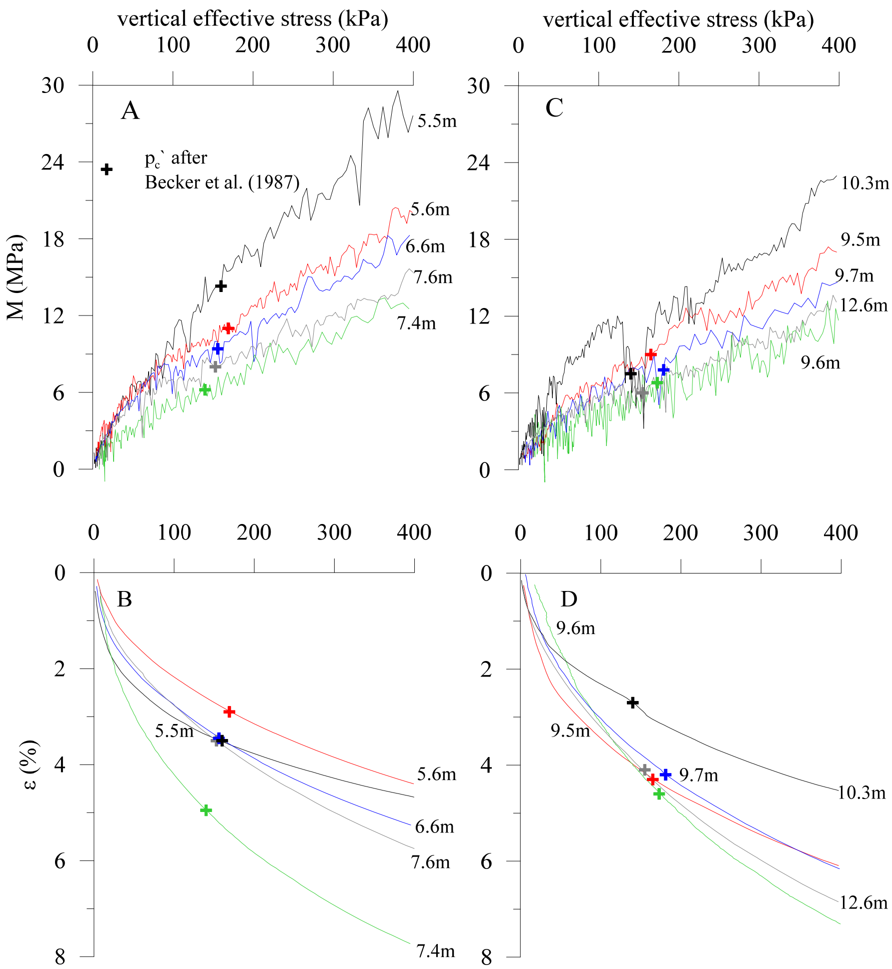

Figure 4-5 Oedometer test results: σ_v` versus M and ε: (A-B) 4 - 8 m & (C-D) 9 - 14 m including $p_c$`after Becker et al. (1987).

Apart from the sample from 10.3 m depth, there is no evidence of any change of behaviour around p_c` which supports the fact that the preconsolidation stress is blurred due to sample disturbance. Even though the method proposed by Becker et al. (1987) could be successfully applied to the present study, further research on its use in silts is warranted.

Finally, vertical coefficients of consolidation (c_v) were established using the method proposed by Janbu (1963). Figure 4-6 (A) presents two examples from 7 m and 9 m respectively, showing the development of c_v with σ_v`. Due to the rather scattered results, design values for $c_v$ have been taken at vertical effective in-situ stress ($\sigma_{v0}$`) leading to a c_{v0} profile with depths shown in Figure 4-6 (B). The results confirm the irregular structure of the silt deposit. Values of c_{v0} for the upper silt layer are scattered and vary between 80 m^2/year and 750 m^2/year whereas results of c_{v0} for the lower silt layer are more consistent between 80 m^2/year and 300 m^2/year. Studies carried out by Sandven (2003) showed similar results, emphasizing on the influence of sample disturbance and irregular layering on the laboratory test results.

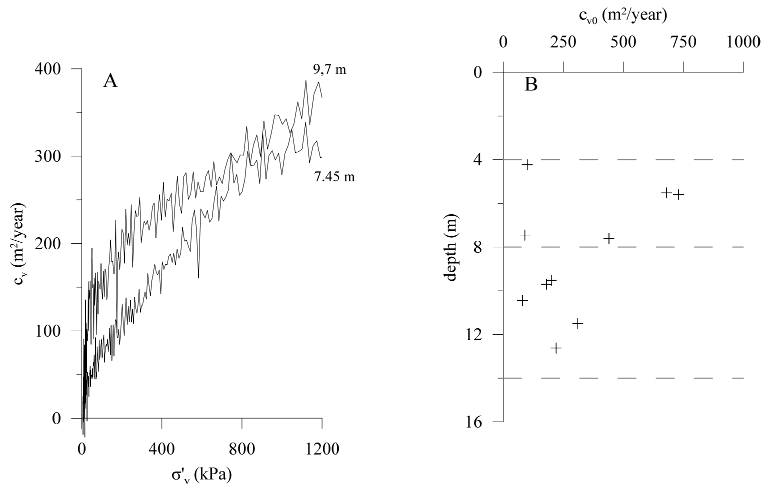

Figure 4-6 Coefficient of consolidation from CRS tests: A) example from 7 and 9 m depth & B) variation of c_{v0} at σ_{v0}` with depth.

4.3 Triaxial tests

Anisotropically consolidate undrained compression tests (CAUC) have been conducted on various samples from the research test site where the standard test procedures proposed by Berre (1982) have been adopted. In order to consolidate the samples back to the in-situ effective stress level, a coefficient of earth pressure at rest (K_0) of 0.5 has been used which is in accordance with work carried out by other researchers close to the present research site (Sandven 2003). Volumetric strains between 1.1 % and 4.5 % were recorded during the consolidation process. During the shear phase a strain rate of 4 %/hour was used and for most of tests a backpressure of 200 kPa has been applied.

Handling the silty material in the laboratory during preparation and building into the apparatus was challenging. Lateral displacements occurred at times due to the self-weight of the specimen and it was difficult to keep the sample in a vertical position during trimming due to the rather coarse and fragile structure. Due to the mentioned difficulties, it cannot be precluded that densification of the samples might have occurred during preparations. For future projects in silts it is recommended to try taking 75 mm samples instead of 54 mm and use an approach where the sample is directly extruded into the triaxial membrane as suggested by Wijewickreme and Sanin (2006). By using a larger sample, the cross-sectional area is lager which is welcome especially for non-homogenous soil deposits leading to more representative test results. Figure 4-7 shows the test results in terms of shear stress (τ) and pore pressure (Δu) vs. axial strain (ε) or effective radial stress (σ_r'). Figure 4-7 (A-C) represents results for the upper silt layer whereas Figure 4-7 (D-F) shows results for the lower silt layer. All samples showed "barrel" shape failure and no distinct shear bands occurred.

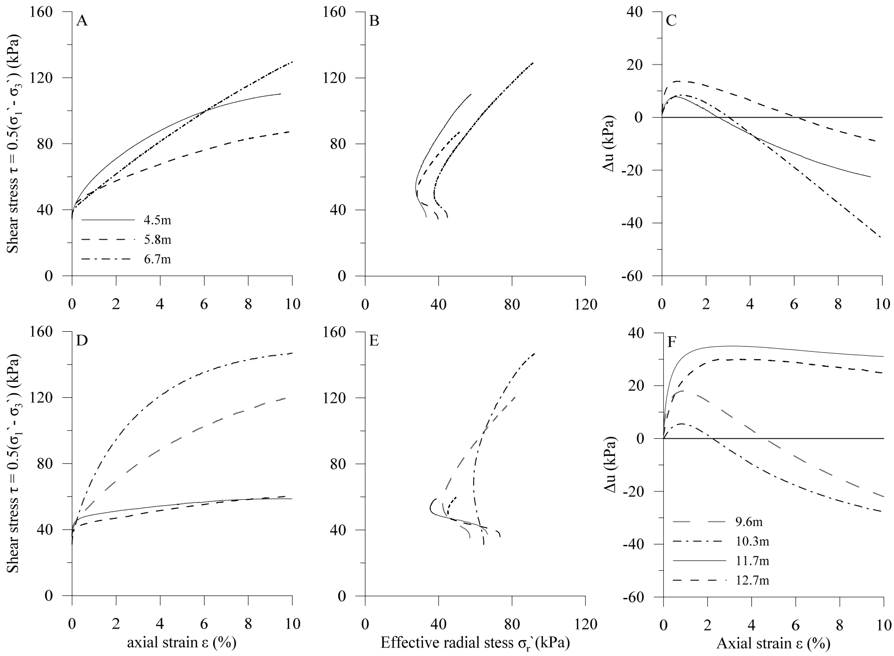

Figure 4-7 Triaxial test results: (A-C) 4 - 8 m & (D-F) 9 - 14 m.

A strong dilative behaviour can be observed for the samples between 4 m and 10 m depth with increasing τ and ε without reaching a defined maximum shear. The stress paths reveal some contraction upon shearing followed by dilation with increased effective stress level continuing along a clear failure line. The pore pressures increase up to a strain level of about 1% for all samples. At higher strains pore pressures decrease and dilate into negative values. For samples below 10 m depth, a different pattern can be seen, showing contractive behaviour with a more distinct maximum shear strength and positive pore pressures in the beginning of the test, before is tends slightly to dilate at larger strains due to less coarse material and higher clay contents at deeper depths.

Values of large strain effective friction angle (φ') and cohesion (c') have been established from the triaxial test results, yielding a range of values due to the varied nature of the silt found at the test site. Lower and upper bound values for φ' have been found between 34.2° and 38.7° respectively with an average of 36.9° and cohesion value of 7 kPa. The findings from the present silt site correlate well with results from similar silt sites in Norway. Long et al. (2010) reported a friction angle of 35° for the Os silt south of Bergen whereas Blaker et al. (2019) concluded a value of 36° for the Halden silt in the Oslo region.

Due to the dilative nature of silty material, shear stresses increase during shear without reaching a clear maximum value which conversely leads to the absence of a unique shear strength value. This fact might be one of the significant differences between clay and silt when it comes to triaxial compression (Wang et al. 1982). Although many researchers doubt the use of s_u for partially drained materials, this parameter is still frequently applied by many engineers and needs therefore some attention as well (Long et al. 2010). Figure 4-8 shows shear strength plots for the Halsen-Stjørdal test site. In order to establish a site specific s_u-profile, it is common practice in Norway to apply the SHANSEP method by using s_u / σ_v' =S(OCR)^m. Using a m-value of 0.8 and S_{silt} = 0.2-0.3 as proposed by Ladd (1991) and upper and lower bound values for OCR, limits for s_u / σ_v' of 0.3 - 0.52 for the present silt site were established. Results from the Swedish fall cone test are varied but most of the results plot close to the proposed s_u / σ_v' limits.

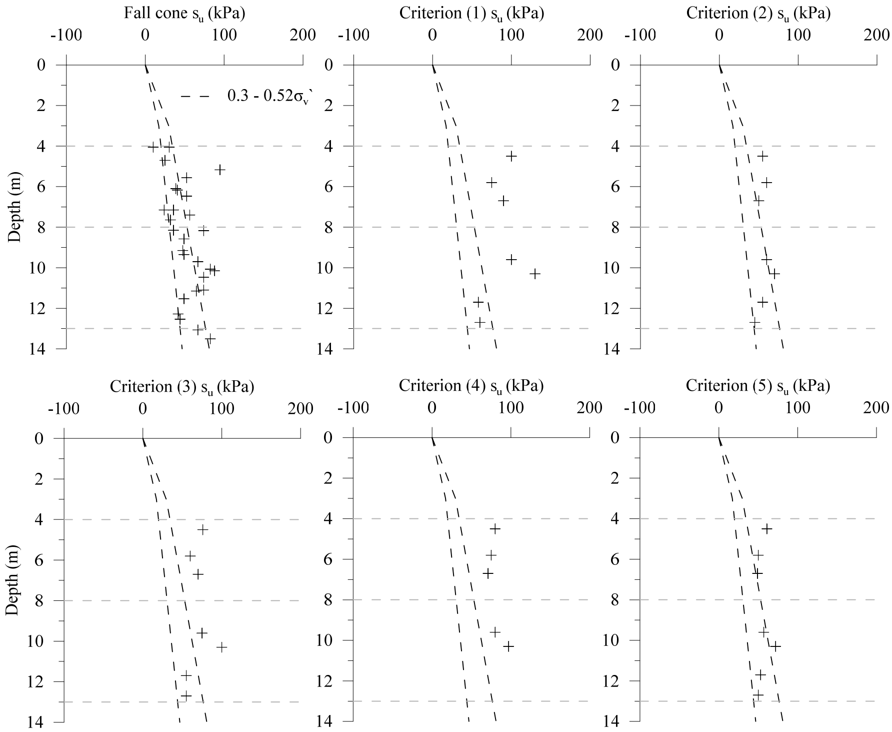

Figure 4-8 Undrained shear strength derived from falling cone and CAUC tests.

To analyze the CAUC data with respect to the undrained shear strength, the five different criteria presented by Brandon et al. (2006) have been applied to the data set (Figure 4-8):

(1) at peak deviatoric stress
(2) at ε = 1%
(3) at reaching the Mohr-Coulomb line
(4) at Δu = 0
(5) at maximum pore pressure (u_{max})

Currently there exist little guidance in literature which criteria to use for silty materials. The dilative behaviour of low plastic silts complicates the interpretation and makes it difficult to define a general failure criteria. For the dilative samples between 4 m and 10 m, criteria (1) and (3) did not work due to unrealistic high s_u values. Even though the results from criterion (4) showed less scattered results than criteria (1) and (3), it revealed high s_u values as well since the conditions were met at relatively high strains. The most consistent and realistic results could be achieved by applying criteria (2) or (5) since the limit strain has been chosen at 1% ensuring no dilation which coincides for most of the test results with the strain at maximum pore pressure (criterion (5)). The results for these two criteria follow the SHANSEP limits and reveal least scattered results. Researchers working with silty materials have reported the successful application of criterion (5) as being the most promising one since it is on the conservative side being well below the fully mobilized failure and ensuring no dilative pore pressures (Brandon et al. 2006; Long et al. 2010). For the more contractive samples below 10 m the s_u criteria work in a more consistent way (probably also due to the higher clay content). Except for criterion (4) which cannot be applied due to contractive pore pressure development, all results plot well in the s_u / σ_v' limits which underlines the application for the models in contractive soils.

4.4 Sample quality

It is of high importance to consider sample quality for both practicing engineers and in research since the results of the laboratory tests will be used directly for design and to establish new approaches and correlations. Two internationally well accepted methods are used based either on volumetric strain (ε_{vol}) (Andresen and Kolstad 1979) or on the change of void ratio normalized by the in-situ void ratio e/e_0 (Lunne et al. 1997a). For the evaluation of sample quality in Norway the latter method is recommended (NGF 2013).

The above described methods are based on studies of marine clays (sampling depth 0 - 25 m) with an overconsolidation ratio (OCR) varying between 1 - 4 and plasticity index (IP) between 6% and 43%. Lunne et al. (2006) stated that care must be taken when applying these methods to soils that fall outside the mentioned ranges. Especially silts may suffer from densification during shearing and sampling, indicating an unrealistic low void ratio or change of volumetric strain upon recompression to in-situ stresses, leading to an apparent high sample quality, even though being highly disturbed (Long et al. 2005; Blaker et al. 2019). Carroll and Long (2017) stated that applying these methods to silty materials is challenging and will often result in misleading conclusions and may not reflect the true quality of the specimen. Up to now no well-established framework exists to assess the sample quality in silts. Recent studies have shown that the use of strain energy and compression ratios can be a useful tool in intermediate, low plastic soils. DeJong et al. (2018) established a method based on the concept developed by Becker et al. (1987), defining the work as the energy necessary to compress the soil to a given stress state. Equation 1 defines the work per unit volume for a given load increment, where σ_i' and σ_{i+1}' are effective stresses and ε_i and ε_{i+1} strains at the beginning (i) and at the end (i+1) of the current load increment:

$$\Delta W_{oed} = \left[\frac{\sigma_i^{'} + \sigma_{i+1}^{'}}{2}\right](\epsilon_{i+1} - \varepsilon_i) \tag{1}$$

The cumulative work increments (ΔW_{oed}) for loading to a certain stress level (σ_v\`) can be plotted against $\sigma_v$\` and be used to interpret p_c' from an oedometer test. Based on this concept, strain energy compression indices were established for both recompression (C_{rw}) and virgin compression (C_{cw}). C_{rw} represents the initial recompression strain energy index, which is defined from seating stress to in-situ condition. Since the oedometer tests in the present study have no unloading reloading loop, C_{cw} has been evaluated from the stress interval between (2.5 - 5) · p_c` (DeJong et al. 2018). Furthermore, generating the ratio of strain energy compression indices (C_{rw}/C_{cw}) normalizes the influence of plasticity and is independent of the in-situ stress and OCR. Therefore, this ratio becomes a useful indication of sample disturbance for a wide range of soil types and in-situ conditions. The soil database used to establish the sample quality criteria ranges of soil mixtures with IP between 0 % and 31 % and maximum σ_v` of 1000 kPa.

The data set from the present study has been applied to the clay-based volume change criteria as well as to the strain energy and compression ratio concept using results from the oedometer tests. Figure 4-9 shows the sample quality assessment for the clay-based volume change criteria. The void ratio defines half of the samples as good to fair and the other half as poor whereas the volumetric strain defines all samples as poor quality. None of them are characterized as very poor. A decreasing sample quality with depth can be seen which supports findings from other researchers who report that the approach using void ratio is effective stress dependent (Krage et al. 2016).

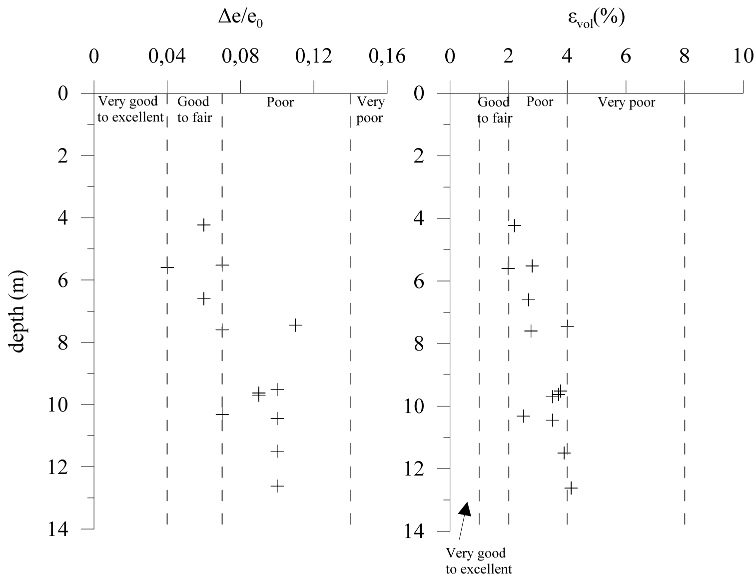

Figure 4-9 Sample quality assessment clay-based volume change criteria.

Figure 4-10 (A) shows the results for strain energy and compression ratios. Four samples are of moderate quality and the remaining ones are of low quality. Since the method is independent of the present effective stress level it also reveals reasonable results for deeper samples. Figure 4-10 (B) shows the combination of the void ratio and the compression ratio approach. Even though most of the samples are defined as poor quality for the present material, none of them is defined as very poor. Bearing in mind that high quality samples in silt are difficult to obtain, the achieved sample quality is acceptable for this type of material. Although, the sample quality could be increased by extruding the samples directly in the field. Thereby any influence on the sample quality due to transport, temperature changes, vibrations or shock loads can be eliminated (Amundsen 2018).

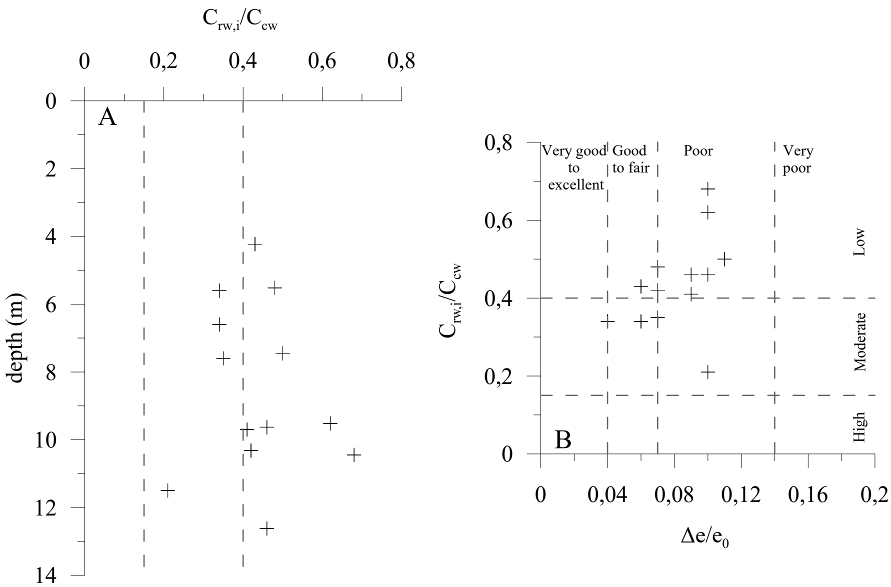

Figure 4-10 Sample quality assessment: A) compression ratio criteria & B) combination of void ratio and compression ratio approach.

4.5 Section summary

- The results from the index tests revealed an either sandy or clayey SILT with most of the particles falling into the coarse silt spectrum. Furthermore, the results show a medium sensitive material with a low plasticity. The material is different from other sites studied in Norway and provides therefore useful information to the existing database.
- No well-founded framework currently exists to assess the sample quality in silty soils. A recent developed method by DeJong et al. (2018) using strain energy and compression ratios seems promising and shows quality results consistent with the void ratio criteria, revealing modest but reasonable sample quality, given the difficult soil conditions. Nevertheless, sample quality is a significant challenge in these materials.
- Due to the more confined procedure using a steel ring to support the soil and the smaller size of specimen, handling the material during preparation of the sample prior to oedometer testing was not as challenging as for the triaxial tests.
- The interpretation of 1D compression tests is difficult in silts and currently no international recognized framework exists to deduce deformation parameters.
- The log σ_v' versus e plots are of rounded shape and no distinct p_c' can be identified. The flat curves in the upper samples indicate the presence of densification. No unique NCL exist which is typical for silts.

- The constrained modulus curves show no change of behaviour around p_c'. These factors complicate the interpretation of oedometer tests in silt which is caused by sample disturbance.
- The method proposed by Becker et al. (1987) to determine p_c' showed promising results for the present silt site with OCR values between 1 and 2, supporting the geological history of the location. Nevertheless, more research is warranted on the application of this method in silt materials.
- The procedure for preparing the samples for CAUC tests lead to challenges during handling the material in the laboratory and it cannot be precluded that densification of the samples might have occurred during preparation.
- The results from the CAUC tests showed a strong dilative behaviour for the samples between 4 m and 10 m depth, developing dilative pore pressures at higher strains. The deeper samples (increased clay content below 10 m depth) showed a more contractive behaviour, developing positive pore pressures at the beginning of the test before tending to slightly dilate at large strains.
- An average effective friction angle of 36.9 ° and effective cohesion of 7 kPa were determined for the present test site which is consistent with values found by others studying similar silt sites.
- The determination of s_u in silt is complicated and little guidance exist up to now in the literature which criteria to use for silty materials.
- Due to the dilative nature of the material, shear stress increases during shear without a clear maximum value. According to Wang et al. (1982) this might be one of the significant differences between clay and silt when it comes to triaxial compression.
- Selecting design values for s_u at ε = 1% or at u_{max} revealed the most promising results, with values following the established s_u / σ_v' limits and being less scattered. The latter criterion has been found to be the most promising one in literature, revealing conservative results by being well below the fully mobilized failure and ensuring no dilative pore pressures (Brandon et al. 2006; Long et al. 2010).
- Given the fact that possible densification took place during sampling, one must be careful when choosing parameters for geotechnical design purposes and test results need to be treated with caution. Choosing design strength values on the conservative side is appropriate for these soil conditions.

5 Variable rate CPTU results (Paper 1 - 3)

All CPTU tests have been carried out at the Halsen-Stjørdal test site. The following chapter summarizes the main findings and conclusions from parts of Paper 1, 2 and 3. The papers can be found in the appended section of the thesis.

5.1 Standard rate tests

5.1.1 CPTU results

Various standard rate penetration tests (20 mm/s) have been carried out at the test site to examine the silt deposit. Figure 5-1 shows results from a few standard CPTU tests in terms of measured cone resistance (q_t) corrected for out of balance pore water pressure effects, pore pressure at the u_2 location (u_2) and sleeve friction (f_s) down to a depth of 18 m. In addition, the ratio of measured excess pore pressure to net cone resistance (q_t - σ_{v0}) defined as the pore pressure parameter (B_q) has been calculated from the measured parameters and can be studied in Figure 5-1 (Wroth 1984). Furthermore, the hydrostatic pore pressure line has been indicated in the u_2 - plot, as well as the borders for the upper and lower silt layer as defined earlier.

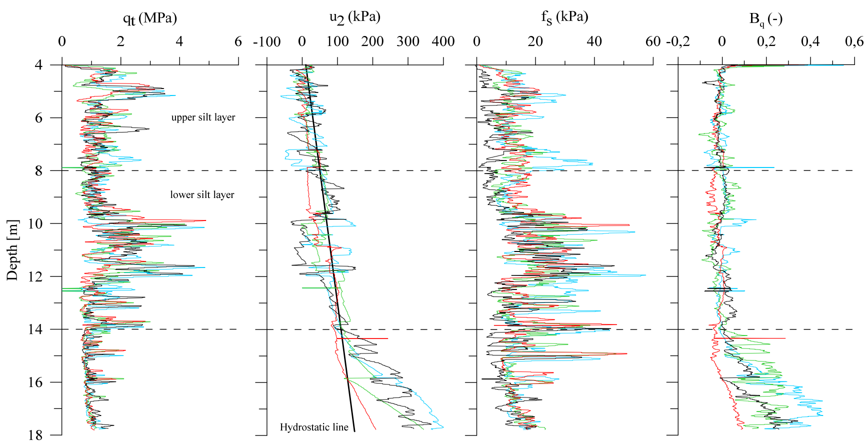

Figure 5-1 Standard CPTU results from Halsen-Stjørdal.

In general, the measurements reveal a stratified soil deposit with small layers and lenses which are picked up rather precisely by the readings, verifying the good quality of the CPTU tests. The test results confirm the previous laboratory results from this study. The silt layer between 4 m and 14 m is indicated by rather constant cone resistance readings for all tests with an average value of 1.5 MPa and some peak values between 10 m and 12 m depth, representing typical values for a loose to medium stiff silt. Pore pressure measurements are somewhat more scattered reflecting the natural variability of the soil deposit and showing partly negative readings. The results from the sleeve friction measurements reveal an increasing trend with depth although being rather scattered. Even though Lunne et al. (1997b) stated that measurements of f_s are less reliable and should be used with care, the present results confirm the rather irregular, silty nature of the silt deposit. B_q values for the silt layer (4 - 14 m) are low and even negative due to the low u_2 readings. In deposits where B_q values are lower than 0.3 to 0.4, care has to be taken when using this parameter for further analysis (Senneset et al. 1988). Due to the higher clay content and lower silt content below 14 m depth, cone resistance readings become steadier with increasing u_2 and decreasing f_s values, confirming the results from the laboratory study.

5.1.2 Soil behaviour charts

Derived values deduced from the measured CPTU results can be used to describe the soil behaviour using soil behaviour charts. These charts are meant as a guidance to define soil behaviour rather than a definition of soil type or grain size distribution. Three different types have been chosen for the present analysis, representing the most widely used soil behaviour charts in Norway and worldwide. The soil behaviour chart established by Senneset et al. (1989) used B_q and q_t. Whereas the well-known solution proposed by Robertson et al. (1986) and later revised by Robertson (1990) is based on a combination of either normalized cone resistance ($Q_t = (q_t - \sigma_{v0})/\sigma_{v0}' = q_n/\sigma_{v0}'$) and B_q or Q_t and friction ratio ($F_r = (f_t/(q_t - \sigma_{v0})$). Lately, Schneider et al. (2008) focused on detecting the behaviour of intermediate soils and based his solution on a combination of Q_t and normalized excess pore pressure ($\Delta u_2/\sigma_{v0}'$). Figure 5-2 shows CPTU results from the test site plotted into the three different behaviour charts, separating the upper and lower silt layer respectively.

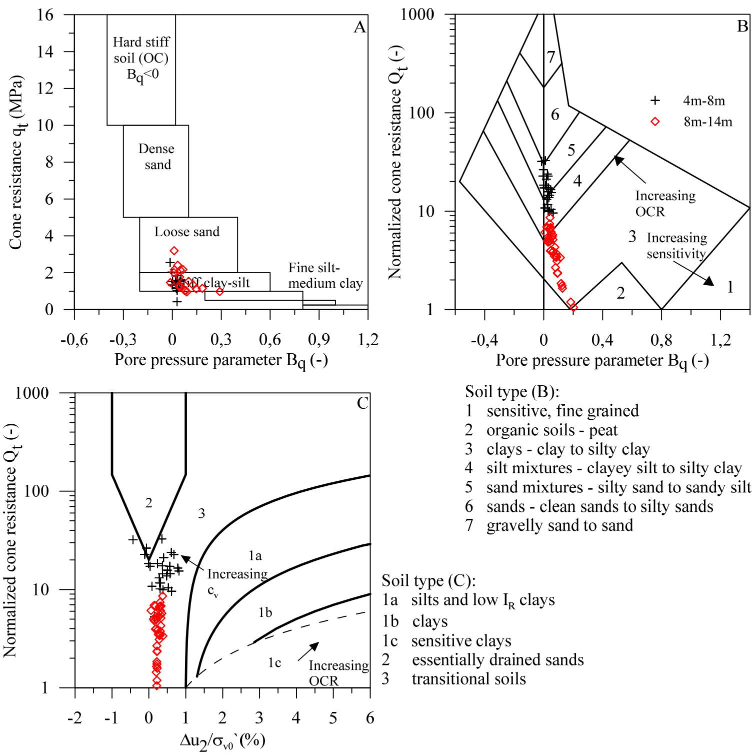

Figure 5-2 Halsen-Stjørdal data plotted into SBC charts according to: A) Senneset et al. (1989), B) Robertson (1990) & C) Schneider et al. (2008).

The solution proposed by Senneset et al. (1989) suggests defining the soil behaviour as stiff clay to silt without distinguishing significantly between the two silt layers. Furthermore, the chart by Robertson (1990) defines the upper silt layer behaving as a silt and sand mixture (zone 4 and 5) whereas the lower silt layer is mainly characterized by a clay to silty clay soil behaviour (zone 3). Finally, the approach by Schneider et al. (2008) defines the CPTU results for both silt layers as a transitional soil behaviour (zone 3). Even though the presented solutions show promising results in detecting the present silt layer behaving as a silty material, the solution presented by Schneider et al. (2008) identifies the whole silt layer consistently as intermediate soil behaviour.

5.2 Variable rate tests

5.2.1 CPTU results versus depth

To study the drainage behaviour of the investigated silt deposit, variable rate tests have been conducted. The penetration rate has been varied between 0.5 mm/s and 200 mm/s but kept constant throughout every single test. By varying the penetration rate over a wide range, the test program aimed to cover all penetration conditions from drained to partially drained and finally undrained. To study the drainage behaviour of the silt, the focus will be on the 10 m thick layer between 4 m and 14 m for the following rate studies of this thesis. Figure 5-3 shows the results for some of the varied rate CPTU tests with depth in the silt layer, covering a wide range of different rates.

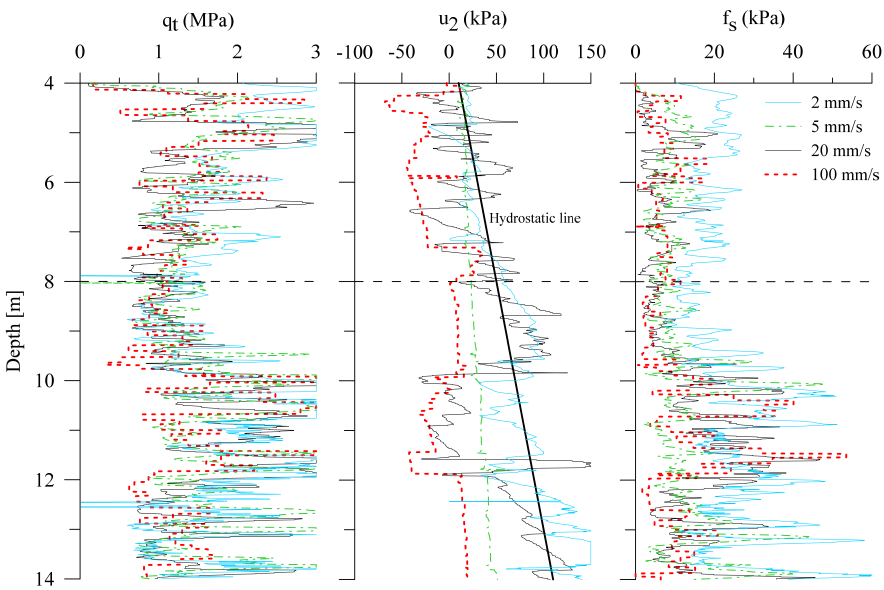

Figure 5-3 CPTU results from variable rate tests at Halsen-Stjørdal in the silt layer.

The readings reflect the very layered structure of the silt deposit and confirm the results from the standard rate tests from Chapter 5.1. Cone resistance readings reveal higher values for lower penetration rates and decreasing values for higher rates. In contrast to the pore pressure readings which seem more scattered, indicating the lowest values for the fast rates, resulting in negative values for some tests. Results close to the hydrostatic pore pressure line can be seen for very slow tests. Friction readings are in general very scattered throughout every single test, revealing less reliable results as mentioned in Chapter 5.1.

5.2.2 CPTU results versus penetration rate

Due to the scattered structure of the silt deposit and in order to being able to compare a wider range of tests and to make an assessment about the drainage condition it was decided to plot the results for each CPTU test (averaged over the silt layer) over penetration rate. Due to operational problems of the pushing system, it was complicated to keep the penetration rate constant for some very slow penetration tests. Therefore the interpretation intervals have been divided into smaller segments with an average constant penetration rate varying between 0.05 and 1 mm/s. Figure 5-4 presents the averaged results for all CPTU tests taken at the site in terms of q_t, excess pore pressure (Δu_2) and f_s over penetration rate (v). The upper silt layer (4 m - 8 m) is shown in Figure 5-4 (A-C) whereas the lower silt layer (8 m - 14 m) is presented in Figure 5-4 (D-F). Furthermore, the standard penetration rate of 20 mm/s is marked in all plots.

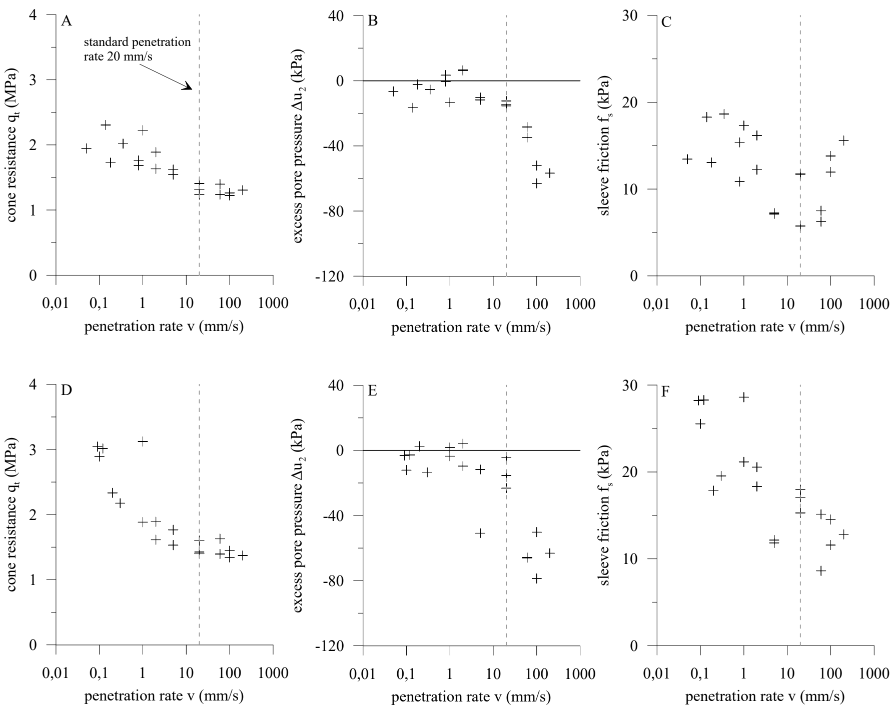

Figure 5-4 Varied rate CPTU results averaged for the upper silt layer (4 m - 8 m): A) q_t, B) Δu_2 and C) f_s versus penetration rate & for the lower silt layer (8 m - 14 m): D) q_t, E) Δu_2 and F) f_s versus penetration rate.

The upper silt layer shows a strong rate dependency for both q_t and Δu_2 results. Readings of the cone resistance are indicating the highest values for very slow penetration rates at around 2.4 MPa and decreasing with increasing v. For high penetration rates, q_t values become steady at about 1.1 MPa. The results for Δu_2 indicate low values for slow penetration rates around the hydrostatic level (converging to zero) and developing increasing negative values for high penetration rates. Similar findings are reported by Schneider et al. (2007) and Krage et al. (2016) working with silty material similar to the silt in the present study. Sleeve friction readings are scattered and widespread, not indicating any trend with penetration rate, maybe indicating slightly higher values at low rates. Several researchers working with rate effects in intermediate soils have reported a similar trend of higher f_s readings with decreased v (Kim et al. 2008; Krage et al. 2016). Due to the uncertainties concerning the repeatability of f_s, this parameter will not be considered any further in this study. Results for the lower silt layer reveal similar trends as reported for the upper silt layer. Nevertheless, indicating higher q_t readings of maximum 3.1 MPa and minimum 1.3 MPa. Sleeve friction readings are higher as well and not being as scattered as for the upper silt layer.

5.2.3 CPTU results versus normalized penetration rate

Finnie and Randolph (1994) and later House et al. (2001) introduced a normalized velocity (V) which makes it possible to compare various rate tests from different sites (e.g. soil types) and penetrometer types to one another. Furthermore, the degree of drainage of a deposit can be assessed by defining upper and lower values for V corresponding to an undrained and drained limit, respectively. Equation 2 relates v, the diameter of the pushing probe (d) and c_v in the following way:

$$V = \frac{v}{v_{ref}} = \frac{v}{d/t_{ref}} = \frac{v}{d/\left(d^2/c_v\right)} = \frac{v \cdot d}{c_v} \tag{2}$$

Figure 5-5 presents the results in terms of Q_t and normalized excess pore pressure versus V as suggested by Schneider et al. (2007) for the upper and lower silt layer accordingly. Due to pragmatic reasons, averaged values for c_v (685 m^2/year up to 826 m^2/year) have been used from existing CRS tests close to the in-situ stress level in the silt layer of interest. In order to fit the data into a function, an hyperbolic equation as proposed by House et al. (2001) and Chung et al. (2006) has been applied to the data set. The results presented in Figure 5-5 confirm the findings from Chapter 5.2.2 indicating a strong rate dependency for both measured parameters q_t and u_2. Both silt layers show maximum magnitudes for the slowest penetration rate, which can be explained by an increased consolidation process taking place ahead of the advancing cone. Increasing the penetration rate leads to decreasing Q_t values until reaching a steady state for fully undrained penetration. The development of $\Delta u_2/\sigma_{v0}'$ illustrates an opposite trend, converging to zero for low penetration rates. This corresponds to a fully drained soil response where all excess pore pressure is dissipated, and the sensors measure hydrostatic pore pressure values. During an increase of penetration rate, high negative excess pore pressure values are observed for both silt layers. These findings are characteristic for a coarse, non-plastic silt deposit. Researchers working with silts comparable to the one studied, report similar findings both in the laboratory and in the field where the rate dependent shear induced pore pressures have a significant effect on $\Delta u_2/\sigma_{v0}$' especially for high rates (Schneider et al. 2007; Krage et al. 2016). Even though Q_t readings reach a constant value for high penetration rates, $\Delta u_2/\sigma_{v0}$' measurements continue to increase for increasing rates, indicating a stronger dilation for high velocities. The observations may be explained assuming two different zones of soil surrounding the pushing device. The cone tip is influenced by a zone of compression whereas dilation dominates in a shear zone further out and behind the cone (Paniagua 2014; Krage et al. 2016).

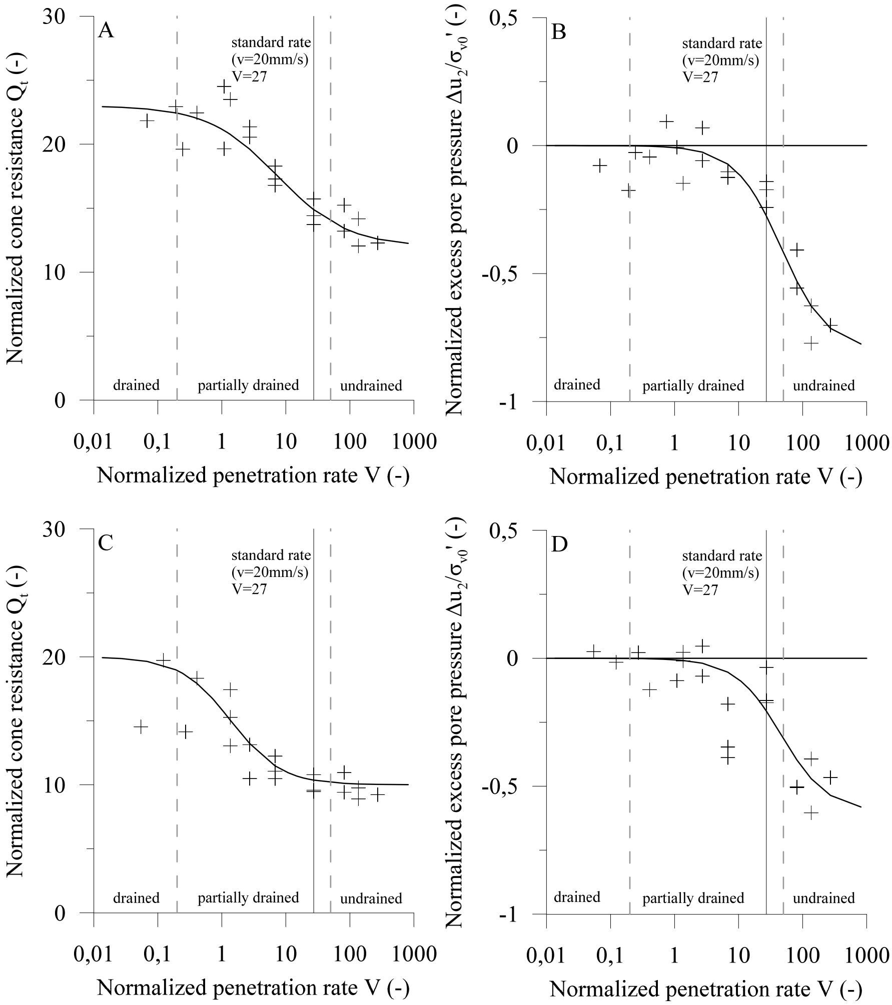

Figure 5-5 Varied rate test results for the upper silt layer (4 m - 8 m): A) Q_t and B) $\Delta u_2/\sigma_{v0}$' versus normalized penetration rate & for the lower silt layer (8 m - 14 m): C) Q_t and D) $\Delta u_2/\sigma_{v0}$' versus normalized penetration rate.

Figure 5-5 offers valuable information on transition points for undrained, drained and partially drained penetration conditions. An upper bound value for V at the present test site can be set to 40 - 50 corresponding to a change from where the penetration moves from a partially drained state into undrained conditions. On the other hand, the penetration changes to drained conditions for V < 0.2 - 0.3. It can be concluded that partially drained conditions can be expected for the present silt when pushing the CPTU cone at standard penetration rate. The presented values for the Halsen-Stjørdal site fit well into reported intervals from various research sites dealing with rate dependency in intermediate soils (Kim et al. 2008; Suzuki et al. 2013; Holmsgaard et al. 2016; Krage et al. 2016; Garcia Martinez et al. 2018). Nevertheless, the variations in reported values are partly dependent on the type of test (field versus centrifuge) and on the choice of a representative coefficient of consolidation (Finnie and Randolph 1994; Randolph and Hope 2004; DeJong and Randolph 2012).

By defining a normalized reference cone resistance (Q_{ref}) corresponding to the minimum undrained resistance, the ratio of drained to undrained cone resistance can be established. Results from the present silt site reveal values between 1.8 and 1.9 for the upper and lower silt layer respectively which correspond well with findings from other silt sites, reporting average values between 1.3 and 3 (Suzuki et al. 2013; Garcia Martinez et al. 2016; Krage et al. 2016).

Schnaid et al. (2020) suggested a new approach for evaluating the drainage situation of a soil deposit and being at the same time independent of the coefficient of consolidation. The new normalized penetration rate $\overline{V}_h$ is based on a combination of penetration rate, probe diameter, rigidity index (I_R) and time for 50 % of consolidation (t_{50}) and avoids thereby the problems in establishing a reliable value for the coefficient of consolidation in partial consolidated soil deposits. The present data set for the silt layer has been applied to the recommended method, yielding reliable results for the silt deposit, and revealing drained and undrained penetration limits corresponding to the results presented above.

5.3 Section summary

- The results from the standard CPTU tests confirm the findings from the laboratory investigations, revealing a rather irregular silt deposit between 4 m and 14 m. Thin layers can easily be identified, confirming the good quality of the tests.
- Applying the measured CPTU data to existing soil behaviour charts indicated a silty, intermediate soil deposit for the present silt layer. The soil behaviour chart proposed by Schneider et al. (2008) gave the most promising and consistent results in identifying the silt layer as intermediate soil behaviour and verified the findings from the laboratory.
- The rate study of the Halsen-Stjørdal silt layer revealed a strong rate dependency of the measured q_t and Δu_2 values. High q_t values are recorded for slow penetration rates with values decreasing for increasing rate of penetration. Results of Δu_2 values show an opposite trend with low values around zero for low rates of penetration and increasing negative excess pore pressures with increasing rate of penetration.
- Readings of f_s are scattered and less reliable without any trend with penetration rate. Therefore, this parameter will not be considered any further in this thesis.

- Introducing a normalized penetration rate (V) revealed borders for drained, partially drained and undrained penetration conditions. For $V < 0.2 - 0.3$ the penetration is drained whereas the penetration is mainly undrained for $V > 40 - 50$. This results in a partially drained penetration condition when pushing the cone with the standard penetration rate at the present test site.
- Deduced values of $Q_{drained}$ / Q_{ref} revealed an average of 1.8 which is within the range reported by others studying the penetration behaviour of silts.
- Schnaid et al. (2020) proposed a new approach to assess the degree of drainage by avoiding the use of c_v or c_h and introducing a new normalized penetration rate $\bar{V}_h$. Applying the method to the present data set revealed drained and undrained limits in accordance with previous results using V.

6 Strength from variable rate CPTU tests (Paper 3)

The following chapter summarizes the main findings and conclusions from parts of Paper 3 which can be found in the appended section of the thesis.

6.1 Undrained shear strength

The use of the undrained shear strength parameter in soils with B_q values lower than 0.4 is questionable and effective stress approaches are preferred in these materials (Senneset et al. 1982; Long et al. 2010). Nevertheless, this parameter needs attention in those soils as well since it is often of interest in practical problems to investigate undrained conditions. Various approaches exist in the literature on how to establish s_u values for clayey soils mainly based on the concept of bearing capacity factors and recommendation is given on how to select the values ensuring a safe design. Nevertheless, this is not the case for silty soils. Little guidance is provided in the literature on how to select appropriate undrained strength parameters in soils that experience partially drained behaviour.

Due to absence of an interpretation framework for silts, the most frequently used correlations for clays have been applied to the existing data. The models are based on bearing capacity formulations and use a combination of cone factors and reference values for s_u from triaxial test results (Lunne et al. 1997b). To establish a s_u - profile for the present silt, the approach based on q_t (N_{kt}) and a combination of q_t and u_2 (N_{ke}) have been used. Figure 6-1 (A) presents the results with depth for a standard penetration test including the reference values from the triaxial tests for the silt layer, revealing N_{kt} and N_{ke} values of 17 and 12 respectively.

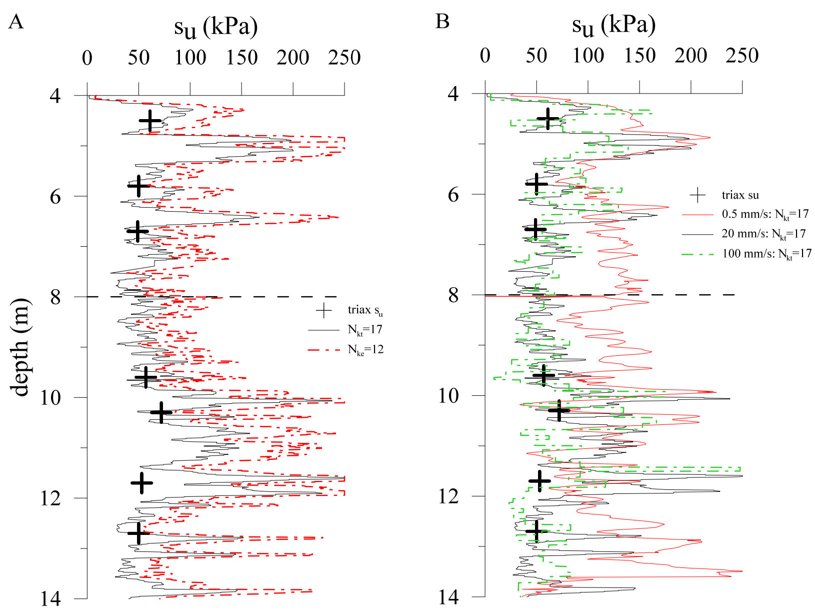

Figure 6-1 Undrained shear strength for the silt layer from CPTU results (being partly drained): A) standard rate (N_{kt} and N_{ke}) & B) varied rate (N_{kt})

The rather strong pore pressure dependency of the N_{ke} approach leads to a less reliable method for the present site. Therefore, the cone resistance approach seems to reveal the most promising results, being well inside the interval of reported N_{kt} values from several Norwegian silt sites (Senneset et al. 1988). Sandven (2003) carried out investigations on a silt site close to the present one, reporting values of 15 for Stjørdal silt whereas Long et al. (2010) stated values of 11 for N_{kt} for the Os silt site close to Bergen, Norway. Blaker et al. (2019) presented results on a silt site south of Oslo, indicating N_{kt} values of 15 for the Halden silt. Carroll (2013) and Long (2007) carried out research on several Irish silt sites reporting N_{kt} values between 15 and 18.

Furthermore, the N_{kt} approach has been applied to varied rate test results using the same N_{kt} factor as for the standard rate test and presented in Figure 6-1 (B) together with the results from the triaxial test results. Results for very slow penetration rates tend to overestimate s_u whereas the values for the fast test may even tend to underestimate it, reflecting the rate dependency of q_t on the results. To study the rate dependency of N_{kt}, the approach has been applied to varies rates (Figure 6-2 (A)) and the results for all rate tests have been plotted in Figure 6-2 (B) over penetration rate.

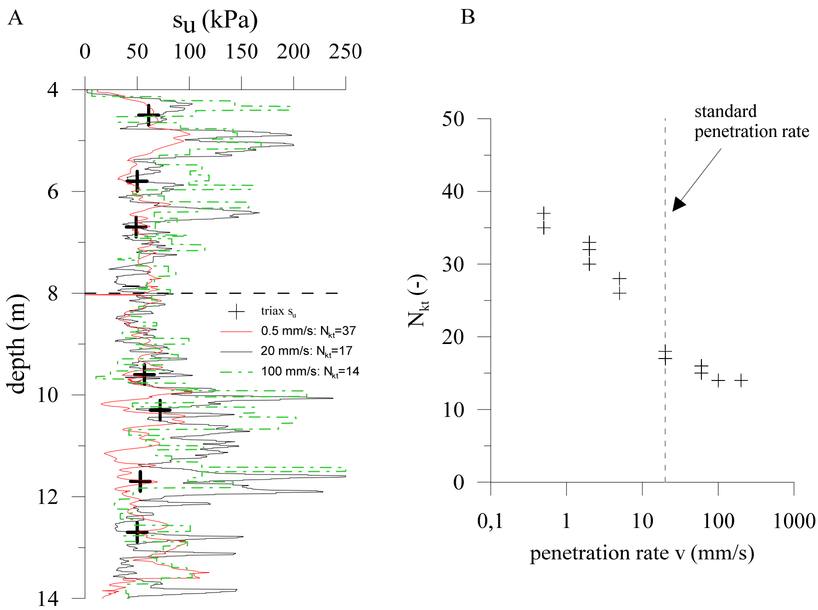

Figure 6-2 Undrained shear strength for the silt layer from CPTU results: A) varied penetration rate (varied N_{kt}) over depth & B) variation in N_{kt} over penetration rate v.

The results indicate high N_{kt} values of 37 for very slow penetration rates whereas low N_{kt} values of 14 are seen for high rates. Researchers already reported the risk of overestimating s_u for silts where partial drainage occurs (Schnaid et al. 2007; Kim et al. 2008). By establishing the ratio of N_{kt_std} to $N_{kt\_undrained}$ a statement can be made about the over- or underestimation of these parameters. The present test site revealed a factor of 1.2 leading to an overestimation of about 20 % of the undrained shear strength if partially drainage is not considered, and shear strength values would be determined using standard penetration rate results only.

The undrained shear strength study at the present silt site highlighted the importance of considering the degree of drainage during a standard penetration test in silt and intermediate soils and account for this when establishing design parameters (Schnaid et al. 2004). To prevent a potential overestimation of the undrained shear strength, it is highly recommended to carry out at least a few fast rate tests to reproduce the undrained conditions. Thereby it is possible to establish site-specific factors, make assumptions about the drainage conditions and adjust the final design values accordingly.

6.2 Drained shear strength

Various approaches have been published in the literature regarding the determination of the peak friction angle (φ') of a soil deposit. For the current project, three different approaches have been applied to the existing data set. Robertson and Campanella (1983) established an empirical approach for normally consolidated (NC) sands based on q_t and σ_{vo}'. Later Kulhawy and Mayne (1990) suggested an alternative expression to account for nonlinear normalization of q_t with stress level. Finke et al. (2001) applied the method successfully to silty soil deposits. The last method (NTH method) that will be used is based on a bearing capacity formulation according to Janbu and Senneset (1974), and is applicable for a wide range of soil types (Senneset et al. 1982; Senneset et al. 1989; Sandven 1990). Therein the plastification angle (β) represents an important parameter when applying the NTH approach, defining the size of the generated failure zone around the advancing cone. In order to make a qualified assessment of β, Sandven (1990) carried out a sensitivity study and developed recommendations relating β to soil type and q_n.

Figure 6-3 (A) shows the results for the silt layer applying the three methods explained above by using a drained (slow rate) CPTU test and including the triaxial test results from the laboratory investigations. The NTH method revealed the most consistent results. The other two methods proposed by Robertson and Campanella (1983) and Kulhawy and Mayne (1990) underestimated φ' significantly especially for the upper silt layer. Therefore, the focus will be on the NTH method for further rate dependency analysis. When applying the NTH method, the correct choice of β is of importance. To study the influence of this parameter on the results, a sensitivity study has been carried out by varying β between 10° and 30°. Figure 6-3 (B) shows the results plotted together with results from the triaxial tests. The best fit with the CAUC test data could be achieved by using a β value of 20° which is in accordance with studies carried out by Sandven (1990) close to the present research site.

To study the rate dependency of the friction angle, the NTH method has been applied to all varied rate CPTU tests. Figure 6-4 (A) shows the results over depth whereas Figure 6-4 (B) presents the results in terms of φ' versus v. As expected from previous results a strong rate dependency can be seen from the presented results, indicating a significant underestimation of the friction angle when using standard rate tests compared to the results for a drained penetration test (slow penetration rate). Rate effects in natural, low plastic silts have been reported by various researchers, recognizing the dependency of the friction angle on the given penetration rate (Holmsgaard et al. 2016; Krage et al. 2016). To prevent an underestimation of φ' it is recommended to carry out additional slow CPTU tests, to achieve drained conditions for the relevant test site. Establishing site specific factors ($\varphi_{drained}'$/ φ_{std}') may result in more realistic φ'-profiles when partial drainage is present.

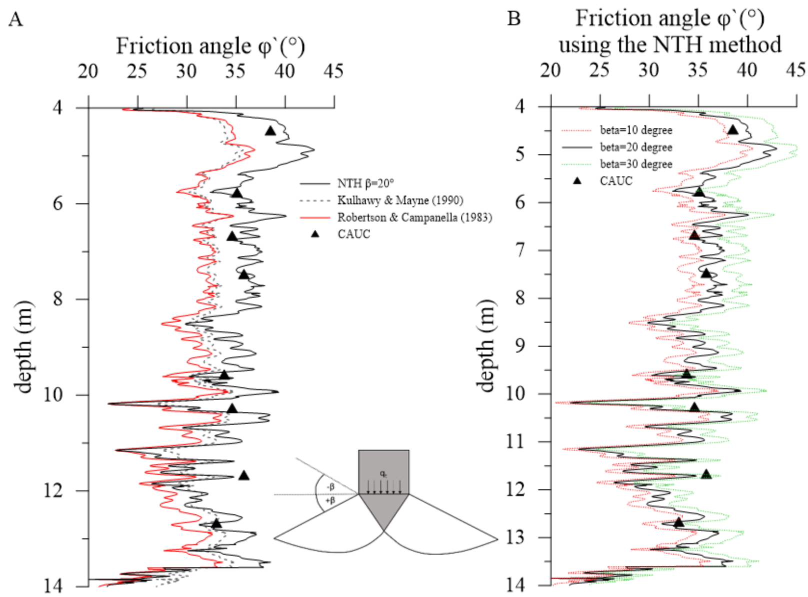

Figure 6-3 A) φ for the silt layer from a slow CPTU test applying several methods & B) φ from the NTH method with varying angles of beta.

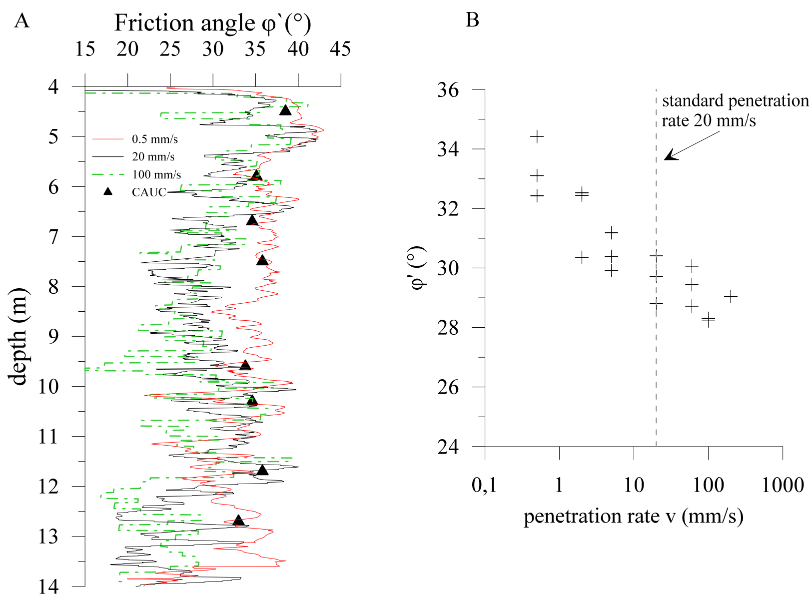

Figure 6-4 Friction angle from varied CPTU tests applying the NTH method for the silt layer (4 m - 14 m): A) over depth for three different rates including CAUC tests & B) over penetration rate for several varied rate tests.

Ouyang and Mayne (2020) investigated the applicability of the NTH method by making use of a database of various soil types, covering mainly clays and fine-grained soils with low permeability and IP between 5 % to 41 %. Natural low plastic silts, dense dilative and highly OC soils were not part of the study. The researchers concluded that the NTH method can be successfully applied and resulted in realistic φ'- profiles, verified by laboratory test results. Although being frequently used in Norway, the NTH method has not gained as much attention internationally as it deserves (Sandven 2003; Bradshaw et al. 2012; Carroll 2013; Ouyang and Mayne 2017; Blaker et al. 2019). The method revealed valuable results for the present silt site and should be considered more often by researchers and practicing engineers when dealing with partial drained soils.

6.3 Section summary

- Even though an effective stress approach is preferred in soils experiencing partially drainage, the issue of s_u needs attention as well since it is often of interest in practical problems to investigate the limiting case of undrained conditions. Furthermore, s_u is frequently used in design methods and parametric studies. Care should be taken in using undrained strength for silts, in particular for excavations, since drainage develops fast and makes the design non-conservative.
- The N_{kt} approach is the most suitable method for the present silt. This facts has already been concluded by others working with silts (Sandven 2003; Long et al. 2010; Blaker et al. 2019). The reported values fall within the range reported in the literature.
- The results revealed an overestimation of the undrained shear strength of about 20 % if partially drainage is not considered, and standard penetration rate tests are solely used and interpreted as if undrained.
- When dealing with silt projects it is recommended to conduct fast penetration tests in addition to standard rate tests to make assumptions about the drainage situation and prevent an overestimation of the undrained shear strength.
- The NTH method revealed the most promising results when establishing a φ'-profile for the current silt site by applying a plastification angle of 20°.
- The rate study carried out revealed a strong rate dependency of φ' on the results, leading to an underestimation of φ' when using a standard penetration test compared to the drained solution.
- For projects where partial drainage is present during standard rate CPTU tests, it is recommended to conduct additional slow rate tests and to establish site specific factors that allow for a more realist φ' profile of a given site.

7 Dissipation test results (Paper 4)

Dissipation tests have been carried out for all CPTU tests at several predetermined depth of interest covering all rates of penetration. All tests have been invariably conducted at the Halsen-Stjørdal test site. The following chapter summarizes the main findings and conclusions from parts of Paper 4 focusing on the interpretation of consolidation parameters from dissipation CPTU tests. The paper can be found in the appended section of the thesis.

7.1 Introduction

The interpretation of dissipation tests is complicated due to amongst others the complexity of the initial pore pressure distribution, soil disturbance and soil anisotropy (Lunne et al. 1997b). Dissipation takes part in a partially remolded soil volume around the tip, caused by the penetrating cone. In cohesive soils large pore pressures are generated due to the cavity expansion generated by the cone, causing a reduction in effective stress (Campanella et al. 1981). When the penetration process is stopped, the excess pore pressures dissipate over time until reaching equilibrium. By measuring the decay of pore pressure at the u_2 position, radial consolidation dominates the dissipation process, expressed by the horizontal coefficient of consolidation (c_h).

Monotonic or standard dissipation behaviour can be expected in normally consolidated soils. However, this is often not the case for intermediate soils such as silts or high OC soil deposits. Dilation in silt during penetration may even create negative excess pore pressure and the classical dissipation curves may not be found. This complicates the interpretation of the dissipation process. In order to classify the different dissipation behaviour of various soils, Sully et al. (1999) categorized them into different types according to Figure 7-1 for pore pressure measurements at the u_2 position. Type II and V show a monotonic decay of pore pressure over time whereas Type III and IV represent a dilatory or non-standard dissipation behaviour (Mayne 2001).

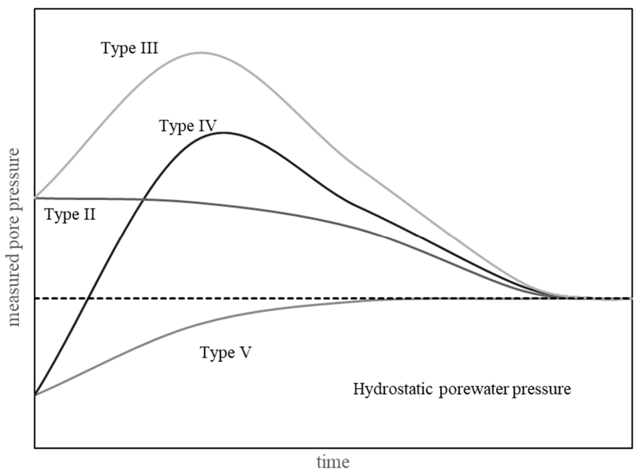

Figure 7-1 Definition of different types of dissipation response for measurements at the u_2 position after Sully et al. (1999).

Factors influencing the non-standard dissipation behaviour are diverse. Pore pressure redistribution upon a stop of penetration is one of them, where a pore pressure gradient towards the pore pressure location causes a drainage flow from the tip to the shaft (Burns and Mayne 1998; Sully et al. 1999; Chai et al. 2004). Furthermore, soil permeability as well as strength and stiffness of the surrounding soil and hence the magnitude of the pore pressure gradient are influencing the rate of drainage and the dissipation behaviour. Due to shear induced dilatancy, excess pore pressures close to shoulder and cone shaft will be lower than in the surrounding soil causing a temporary rise of pore pressure upon decay (Chai et al. (2004). The measured pore pressure during a CPTU test is a combination of hydrostatic pore pressure (u_0), change in normal stress or mean total stress (u_{shear}) and change in u_{shear} (Wroth 1984; Mayne and Bachus 1988). It is generally accepted, that shear induced pore pressures (u_{shear}) play a significant role on the development of non-standard dissipation curves in silty soils. Although poorly saturated pore pressure filters can cause this behaviour, it is generally accepted that this is unlikely a significant reason for this behaviour for silty soils.

7.2 Interpretation models

Teh and Houlsby (1991) established the most widely used interpretation method for soils in undrained dissipation conditions (e.g. standard dissipation behaviour) as seen in soft clays by idealizing the initial pore pressure distribution with the help of large strain finite element analysis in combination with the stress path method (SPM). To make an estimate of c_h, a modified time factor (T*) is introduced, depending on the degree of consolidation, the cone radius (r) and rigidity index. Plotting the dissipation test results by means of normalized excess pore pressure ($U=\Delta u_t/\Delta u_{initial}$) over time (t), the time associated with 50 % dissipation (t_{50}) can be selected and the following equation can be used to yield c_h (Houlsby and Teh 1988; Teh and Houlsby 1991)

$$T^* = \frac{c_h \cdot t}{r^2 \cdot \sqrt{I_R}} \tag{3}$$

In order to evaluate t_{50} for dissipation tests which show a non-standard dissipation behaviour, the initial pore pressure dissipation curves need to be adjusted to fit to existing theories. Burns and Mayne (1998) proposed a solution based on a cavity expansion - critical state model formulation and by using OCR, I_R and φ as input parameters. Furthermore, Sully et al. (1999) introduced amongst others the square root of time plot and the logarithm of time plot correction method in order to fit the measured data to existing theories. However, both correction methods do neglect the initial part of the measured dissipation curves. Chai et al. (2012) suggested a more recent approach, using cavity expansion simulations to adjust t_{50} for non-standard dissipation behaviour. In order to avoid an underestimation of c_h from partially drained dissipation tests, DeJong and Randolph (2012) came up with a modified Teh and Houlsby (1991) approach by accounting for different drainage conditions for modified $t_{50} < 100$ s.

It is important to bear in mind that even though more recent developed approaches account for non-standard dissipation behaviour and partially drained conditions, they do not aim to provide a perfect solution to the complex problem. The interpretation of dissipation tests is difficult and the presented solutions should rather be treated as a given pragmatic framework for practicing engineers as a possibility of adjusting the design parameters according to a specific problem (Robertson et al. 1992).

7.3 Rigidity Index

The determination of consolidation parameters from dissipation test results requires an estimate of the rigidity index of the relevant test site. The size of the plastic failure zone generated during a CPTU test can be related to I_R, representing the ratio of shear modulus (G) to s_u. An increase in I_R (plastic failure zone) will directly influence the pore pressure generation and thereby c_h (Teh and Houlsby 1991; DeJong and Randolph 2012). It is of great importance to establish a careful I_R profile which needs to reflect the actual stress distribution around the penetration cone as realistic as possible (Schnaid et al. 1997). Several approaches exist on how to establish I_R nevertheless, use of high quality laboratory data is preferred. Krage et al. (2014) stated that G is mostly taken at 50 % mobilized strength (G_{50}), representing the average soil response of the affected soil volume around the cone.

Most of the existing approaches to establish I_R are clay-based and reveal often unrealistic results for deposits dealing with intermediate soil behaviour. Several methods have been applied to the existing project. The proposed solution by Kulhawy and Mayne (1990) and later reformulated by Mayne (2001) as well as the approach by Agaiby and Mayne (2018) resulted in unreliable low I_R values. Krage et al. (2016) suggested two different approaches (Method A and B) based on a combination of laboratory data and seismic CPTU data respectively. The results indicated values of I_R between 100 and 300 with an increased tendency with depth where results from Method A fall in between the results from Method B and the laboratory. Carroll and Paniagua (2018) confirmed the successful application of the proposed methods by Krage et al. (2014) on data from a natural silt deposit. For the following consolidation analysis, an I_R of 200 has been used for the upper silt layer, whereas a value of 250 has been applied to the lower silt layer. Dissipation studies published on natural, low plastic and low OC silt deposits confirm the results from this study, with I_R values falling in between the intervals used by others (Suzuki et al. 2013; Holmsgaard et al. 2016; Garcia Martinez et al. 2018; Blaker et al. 2019).

7.4 Dissipation test results at Halsen-Stjørdal

The dissipation tests carried out at the site have been presented in Figure 7-2 to Figure 7-4, sorted by dissipation depth location, penetration rate and type of dissipation test behaviour, respectively.

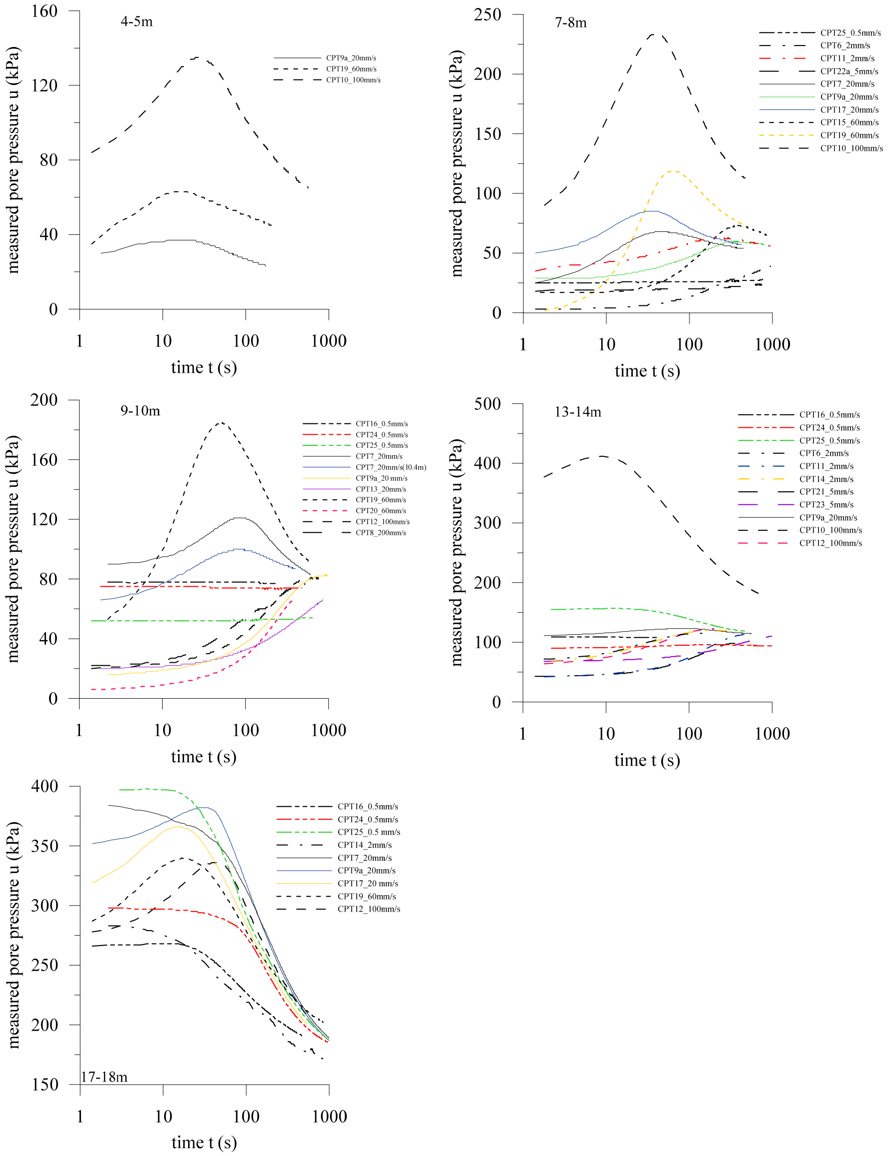

Figure 7-2 Dissipation test results sorted by location.

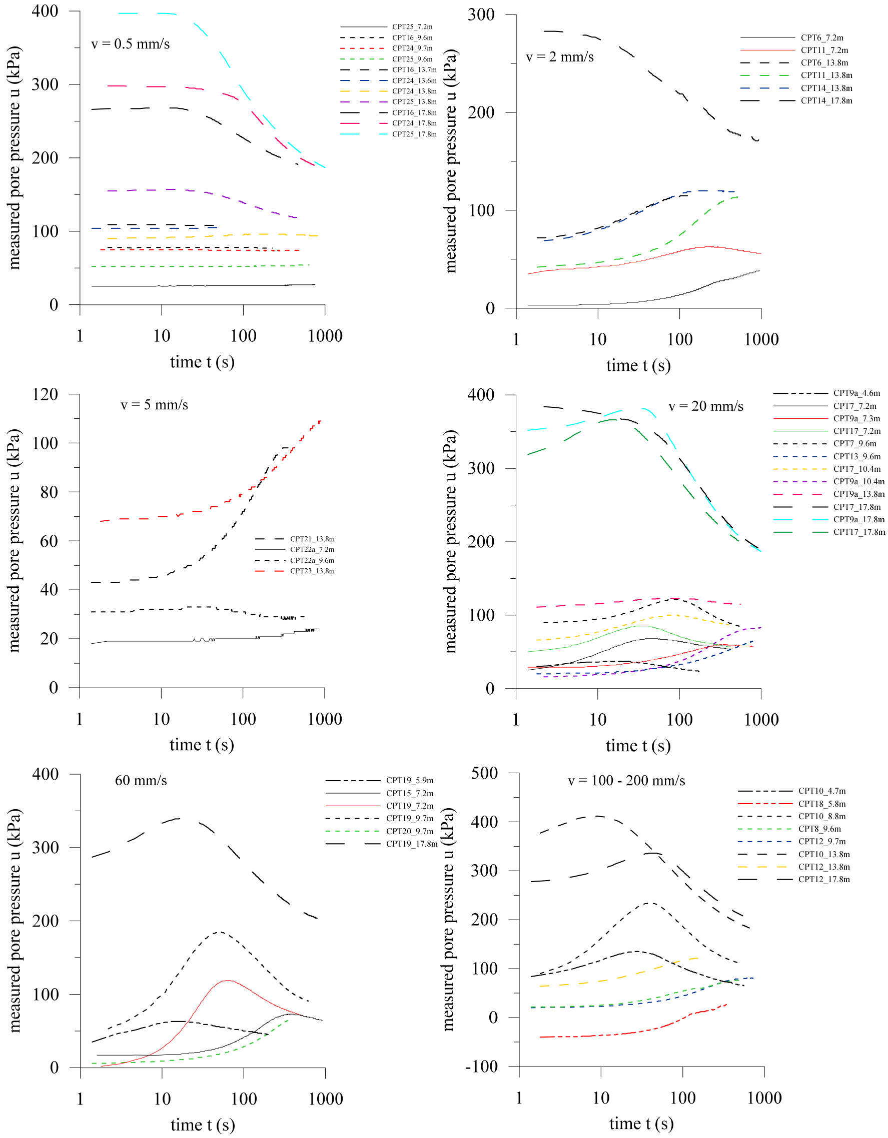

Figure 7-3 Dissipation test results sorted by penetration rate.

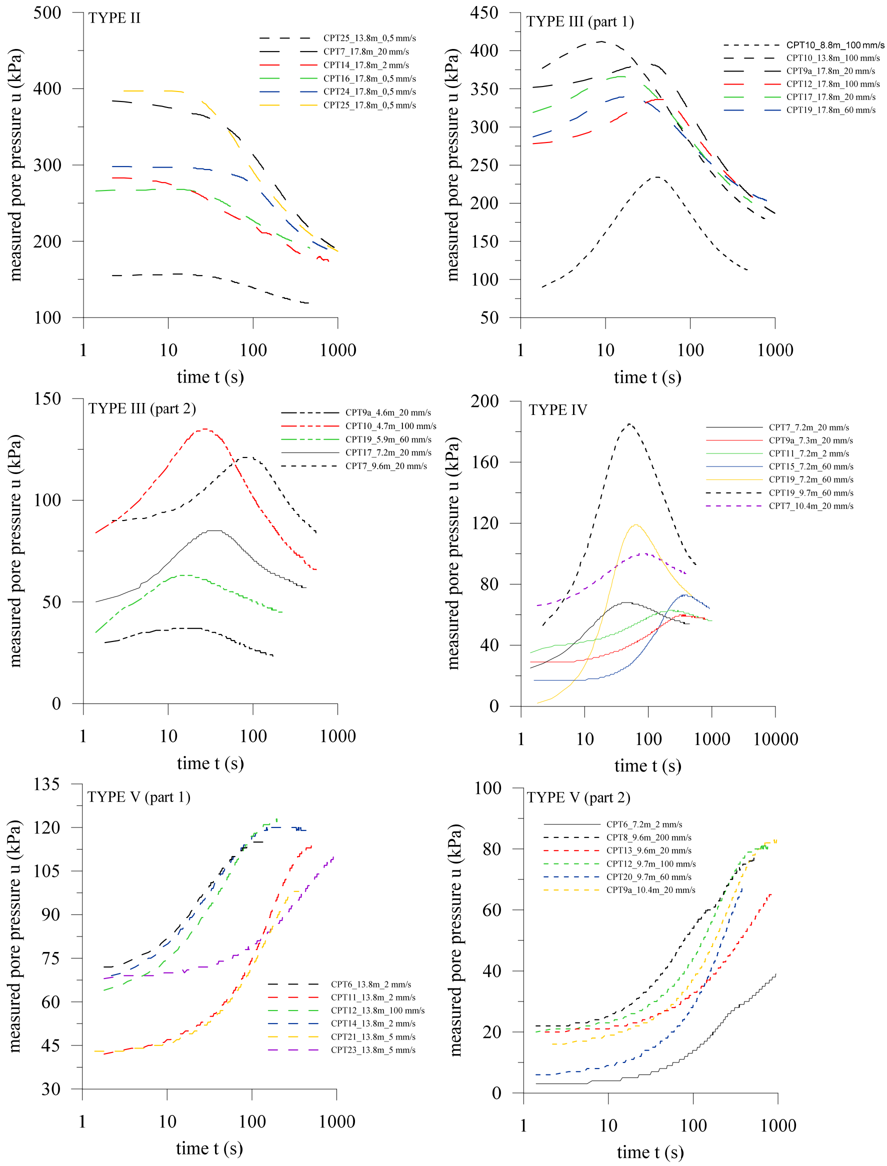

Figure 7-4 Dissipation test results sorted by type of dissipation behaviour: Type II-V.

The results are presented in terms of measured pore pressure versus time (t). In general, the results in the silt layer (4 m - 14 m) are dominated by a non-standard dissipation behaviour (Type III and IV) whereas a standard dissipation behaviour (Type II and V) is observed in the more clayey soil unit below 14 m. Especially the fast tests show a high negative excess pore pressure level upon test start. The results suggest that the results in the silt layer are more influenced by partially drained behaviour than in the layer below, confirming the findings from the rate study at the test site. Especially results for the deepest location at 17.8 m indicate less rate dependency than the dissipation tests carried out in the layer above. Furthermore, hydrostatic pore pressures are measured for the very slow rate tests (Figure 7-3), representing drained penetration conditions which confirm the findings from Chapter 5. Type III and IV dissipation tests represent about half of the analyzed tests and are primarily present in the silt layer. In general, a non-standard dissipation behaviour prevails for the faster rate tests, whereas the monotonic dissipation curves are dominating for the very slow penetration tests.

It is evident, that the negative shear component Δu_{shear} increases with increased penetration rate for the tests carried out at the present test site. Similar results were achieved by Schneider et al. (2007) carrying out centrifuge piezocone studies with different soil mixtures. Results presented by Chow et al. (2020) conducting varied rate penetrometer studies on a reconstituted normally consolidated, natural calcareous silt revealed comparable dissipation curves to this study, supporting the findings from the present study.

7.5 Analysis

7.5.1 Evaluation of t_{50}

In order to establish a t_{50} - profile for the present research site, several methods have been applied to the dissipation data and presented in Figure 7-5.

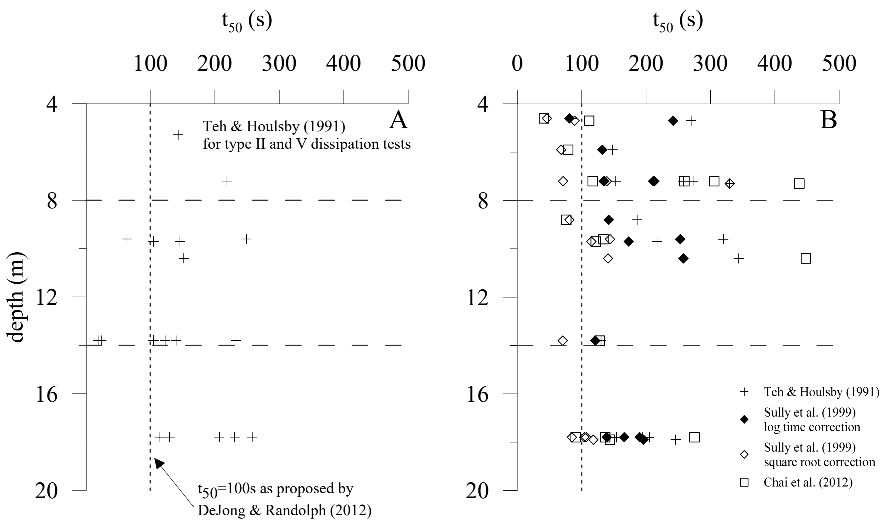

Figure 7-5 t_{50} versus depth profile: A) Teh and Houlsby (1991) for Type II and V dissipation tests & B) Teh and Houlsby (1991), Sully et al. (1999) and Chai et al. (2012) for Type III and IV dissipation tests.

The standard dissipation tests have been analyzed by Teh and Houlsby (1991) whereas the approaches proposed by Burns and Mayne (1998), Sully et al. (1999) and Chai et al. (2012) have been used in addition to evaluated the non-standard dissipation curves. Figure

7-5 (A) shows the results for Type II and V dissipation tests, revealing a scattered t_{50} - profile with depth and representing the rather layered and irregular structure of the soil deposit. The results for the non-standard tests are presented in Figure 7-5 (B). The approach proposed by Burns and Mayne (1998) revealed unrealistic values and has not been considered further in this study. Dissipation studies on natural silt deposits conducted by Paniagua et al. (2016) and Carroll and Paniagua (2018) concluded similar results to the present study. The results show an even more scattered t_{50} profile for the silt layer than the one presented for the standard dissipation tests and no trend with depth can be recognized. The data established by Chai et al. (2012) is very scattered and widespread which makes this method rather inappropriate to use for the current test site. Studies conducted by Mahmoodzadeh and Randolph (2014) support the findings from this study, reporting rather inconsistent results from the method by Chai et al. (2012). Out of the two different methods proposed by Sully et al. (1999), the square root correction method yields the most consistent results for the silt site. Research carried out on different intermediate soil deposits concluded on the successful application of the square root correction method (Mahmoodzadeh and Randolph 2014; Krage et al. 2015; Chow et al. 2020).

Furthermore, Figure 7-5 (B) indicates the risk of overestimating t_{50} when applying the approach by Teh and Houlsby (1991) to non-standard dissipation tests, resulting in unrealistic high values for the present silt site. Figure 7-6 shows an example of a non-standard dissipation test emphasizing that the theoretical pore pressure distribution as proposed by Teh and Houlsby (1991) is not able to reproduce the measured pore pressure results. Even though the square root correction method is not considering the pore pressure development before the maximum value, the methods is more suitable in mapping the measured results.

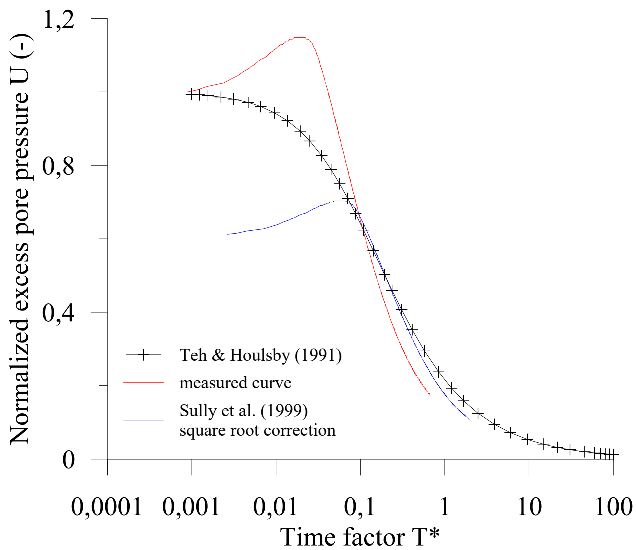

Figure 7-6 Comparison of measured and theoretical curves from Teh and Houlsby (1991) and Sully et al. (1999)

In order to establish a design t_{50} - profile, the results for Type II to V dissipation tests have been summarized in Figure 7-7 (A) by applying the approach by Teh and Houlsby (1991) to the standard tests and the square root correction method by Sully et al. (1999) to the non-standard dissipation tests. Values for t_{50} vary between 20 s and 330 s and are widespread in the silt layer. Below 14 m the results converge more, indicating less rate dependency on the results in the more clayey region of the soil deposit. The limit of 100 s below where to consider partial drainage as proposed by DeJong and Randolph (2012) has been indicated in the plot. The graph reveals that the tests in the silt layer are influenced by partial consolidation, which must be considered when conducting a design profile for c_h, otherwise the risk of underestimating the consolidation value is significant. Figure 7-7 (B) presents all t_{50} results of the site plotted over penetration rate of the respective dissipation test, indicating the standard penetration rate in addition. Even though the results show a rather scattered picture, a trend of decreasing t_{50} values with increasing penetration rate is identified. Carroll and Paniagua (2018) suggested that the high negative excess pore pressures generated during very fast penetration tests lead to a significant increase in dissipation rate compared to the slower tests. Studies carried out on a natural silt deposit and laboratory studies on reconstituted natural silt samples indicate similar trends to the one observed in the present study (Paniagua et al. 2016).

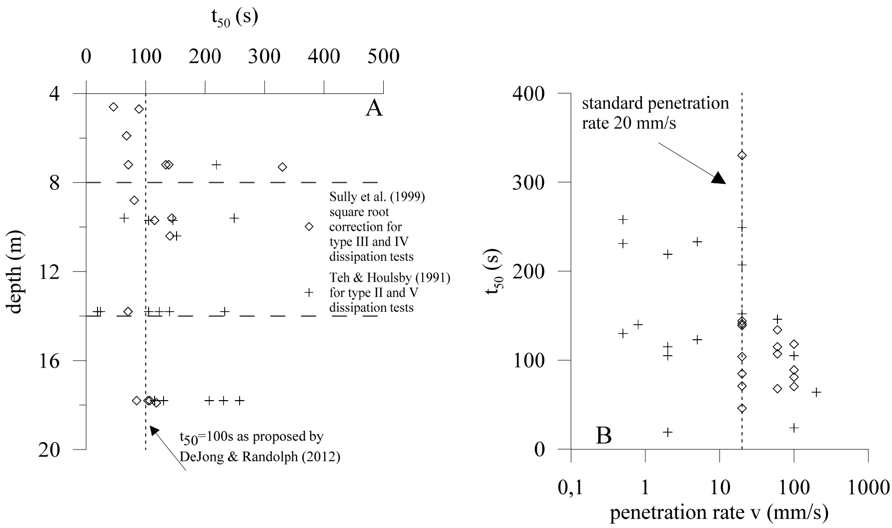

Figure 7-7 A) Design profile for t_{50} with depth & B) penetration rate versus design t_{50}.

7.5.2 Coefficient of consolidation

The interpretation of c_h in soil deposits where partial consolidation is present, is complicated and no framework exist on how to establish design values in these conditions. Nevertheless, an attempt has been made in the present study to identify an estimated profile. In order to study the effect of accounting for partial drainage on the results, the dissipation tests influenced with $t_{50} < 100$ s haven been analyzed by both Teh and Houlsby (1991) and DeJong and Randolph (2012). Figure 7-8 (A) and (B) show results of c_h with depth for the two different methods used, marking the proposed separation line of silt and clay values proposed by DeJong and Randolph (2012) at 95 m²/year. It becomes evident that by accounting for partial consolidation, the c_h values increase by a factor of 1.3, moving the range of data more towards the expected silt region compared to the results from Figure 7-8 (A). The results from the present study indicate that the approach suggested by Teh and Houlsby (1991) is not recommended for silt deposits where partial consolidation takes place during penetration.

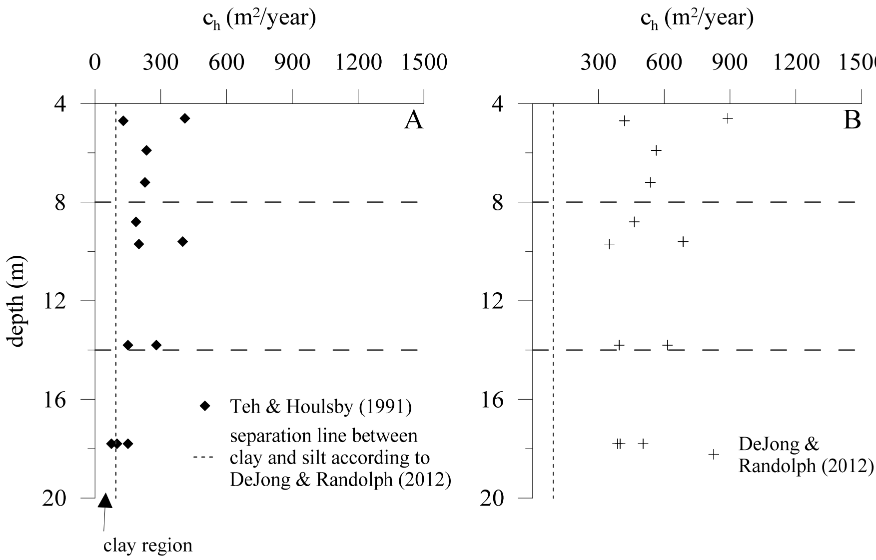

Figure 7-8 Coefficient of consolidation for dissipation tests with t_{50} < 100 s after: A) Teh and Houlsby (1991) & B) DeJong and Randolph (2012).

In order to clarify the consequences of neglecting partial consolidation during the analysis of intermediate soils, DeJong and Randolph (2012) proposed a graphical solution, including the method by Teh and Houlsby (1991) into the logarithmic plot of c_h over t_{50} for a 10 cm² cone and being dependent on I_R (Robertson et al. 1992). For small values of c_h the solution coincides with the new approach, whereas for t_{50} < 100 s partial consolidation increases and the solutions diverge. Figure 7-9 (A) shows the results of dissipation tests influenced by partial consolidation and analyzed by Teh and Houlsby (1991), revealing solutions plotted around the theoretical line of $I_R = 200$ as expected. Furthermore, the dissipation data has been plotted into Figure 7-9 (B) representing the results applying the approach by DeJong and Randolph (2012). As the influence of partial consolidation increases, the results follow the lines suggested by DeJong and Randolph (2012), to prohibit an underestimation of c_h.

Finally, Figure 7-10 (A) shows an estimated c_h - profile for the Halsen-Stjørdal silt test site, including the oedometer test results for c_v. The results reveal a rather scattered profile for the silt layer, confirming the layered structure of the deposit. Ratios of c_h over c_v reveal scattered values of up to 3.1, representing ranges similar to those reported in the literature (Mahmoodzadeh and Randolph 2014; Mahmoodzadeh et al. 2014). Nevertheless, Randolph

(2016) emphasized that a comparison of c_h from the field to c_v established in the laboratory needs to be treated with great care since the link between the elasto-plastic compressional response of the soil and the field values is complicated. Furthermore, the rate dependency of c_h has been investigated in Figure 7-10 (B), indicating an increasing trend of c_h with increasing penetration rate.

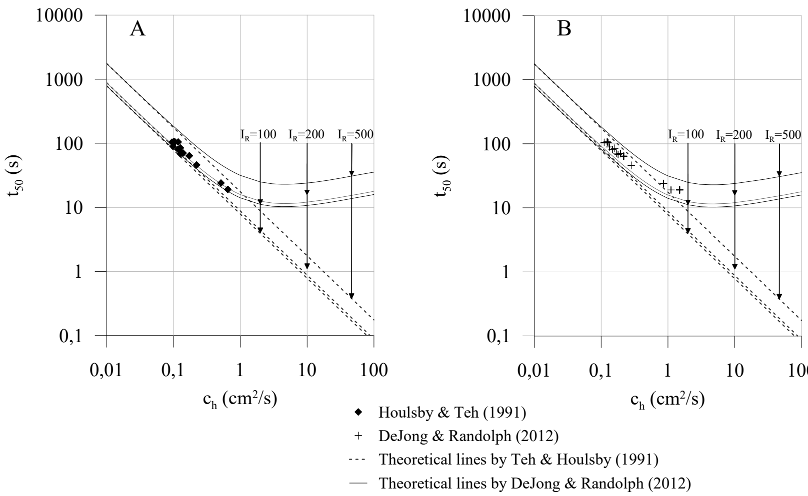

Figure 7-9 Variation of t_{50} < 100 s with c_h: A) Teh and Houlsby (1991) & B) DeJong and Randolph (2012).

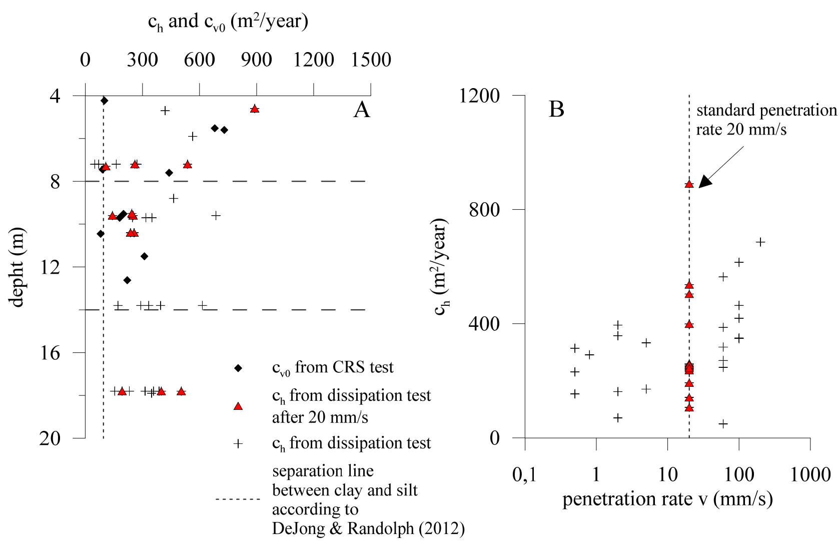

Figure 7-10 A) Estimated c_h - profile including c_v & B) c_h versus penetration rate.

Furthermore, DeJong et al. (2013) established a field decision chart to check whether a certain penetration rate is conducted under partial drained conditions. The results from Figure 7-11 are plotted into the chart, showing c_h over penetration rate. In addition, different lines for normalized penetration rate are indicated as well as regions where to expect an undrained, drained or rather partially drained penetration. As expected and concluded above, the majority of the data falls into the partially drained region and plots into the silt limits as defined by DeJong and Randolph (2012). Due to the absence of data about c_h for the slow penetration tests, the drained results are not visible in this chart. Nevertheless, the undrained region can be clearly defined and coincides with findings from previous rate studies in this silt deposit.

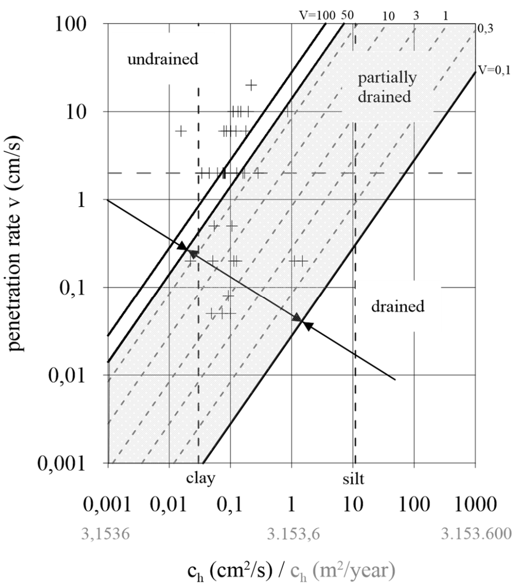

Figure 7-11 Field decision chart after DeJong et al. (2013) including dissipation data from Halsen-Stjørdal.

7.5.3 Examples from a silt site in Norway - Halden

To justify the findings from the Halsen-Stjørdal test site, data from the NGTS site for silts has been used to determine consolidation properties. The research test site is situated south of Oslo and represents one of five NGTS research test sites in Norway. The natural silt deposit is about 10 m thick, consisting of a low plastic, sandy, clayey silt layer. Blaker et al. (2019) and Blaker (2020) present an in-depth description about the ground conditions and various field and laboratory tests conducted at the site.

During the site investigation program, series of varied rate CPTU tests have been carried out, alongside with several dissipation tests at different depths of interest, covering a wide range of penetration rates (Paniagua et al. 2016; Carroll and Paniagua 2018). Three different depths of interest have been selected in the present study to investigate the influence of penetration rate on the dissipation behaviour and consolidation results for the silt. Figure 7-12 shows the dissipation tests in terms of measured pore pressure against time and divided into the three different locations, covering penetration rates between 1 mm/s and up to 300 mm/s. The results indicate non-standard dissipation behaviour (Type III) for most of the tests, except for the very slow one where a monotonic dissipation behaviour is observed. No Type IV or V dissipation tests can be found from the results in Halden, compared to the tests carried out in Halsen-Stjørdal, indicating less shear induced pore pressures being present.

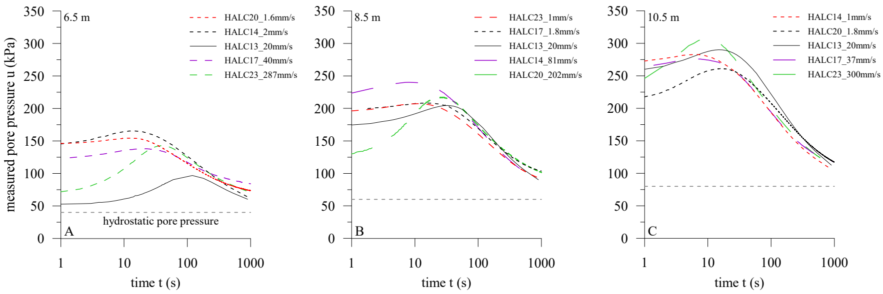

Figure 7-12 CPTU dissipation test results from Halden.

The determination of a t_{50} profile for the Type III dissipation tests revealed less influence of partial consolidation than the results from Halsen-Stjørdal show. Only a few tests yield values of t_{50} less than 100 s, indicating less influence of partial consolidation on the test results. Rate studies indicated a trend of slightly reduced t_{50} values with depth, but not as clear as for the present test site. Furthermore, c_h values have been deduced by applying a rigidity index of 147 for the shallow tests, whereas a value of 126 has been used for the deeper tests as reported by Carroll and Paniagua (2018). For the relevant tests where partial consolidation must be accounted for, the approach by DeJong and Randolph (2012) has been used. The results yield values being well inside the proposed silt range and indicating an increasing trend with depth. Rate studies reveal no clear dependency of c_h with penetration rate. Investigations carried out at the same deposit confirm the results from the present study, emphasizing the use of the proposed methods in natural silt deposits.

Even though the results for the two different natural silt sites revealed promising results, care must be taken when evaluating consolidation parameters in deposits, where partial consolidation may influence the results. The characteristic layering and irregular structure of natural silt deposits complicate the interpretation and challenges may occur in fitting the measured results to the theory used. This might lead to less reliable results which do not represent the real soil behaviour of a deposit altogether (Sandven 2003). Instead of establishing c_h values for a particular silt site, researchers proposed to deduce estimates of the permeability (k) directly by applying empirical correlations between t_{50} and k (Parez and Fauriel 1988; Robertson et al. 1992) or graphical solutions for k for certain values of t_{50} and normalized cone resistance (Robertson 2010).

7.6 Section summary

- The analysis of dissipation tests in intermediate soil deposits where partial consolidation influences the results is complicated and not straightforward. A standardized interpretation framework is up to now not available for these soils.

- Existing approaches to determine consolidation parameters from dissipation tests require a careful estimate of the rigidity index. The methods proposed by Krage et al. (2014) revealed reliable results, corresponding to rigidity index results from the laboratory study carried out at the test site.

- The analysis of the dissipation tests indicated Type III and IV (non-standard) dissipation tests occur mostly in the silt layer whereas the layer below 14 m is characterised by mainly Type II and V dissipation tests (standard).

- The dissipation test results yielded a strong rate dependency in the silt layer. Dissipation tests carried out after a slow penetration resulted in drained behaviour whereas the tests after faster rate tests indicated partial consolidated behaviour. Dissipation tests after very fast rates showed undrained behaviour.

- The square root correction method proposed by Sully et al. (1999) revealed the most consistent t_{50} results for non-standard dissipation tests.

- Rate dependency could be detected, revealing increased c_h and decreasing t_{50} values for increasing penetration rates, indicating an increased dissipation rate with penetration rate.

- For dissipation tests where $t_{50} < 100$ s, partial drainage during penetration must be accounted for. If neglected, c_h values in partially drained materials will be underestimated. The approach suggested by DeJong and Randolph (2012) yielded reasonable results for the present silt after a standard penetration rate. For dissipation tests carried out after penetration rates slower than about 5 mm/s, the tests can only be used to confirm the drained penetration situation.

- Ratios of c_h to c_v revealed factors of up to 3 which is reasonable. Nevertheless, a comparison of c_h from the field to c_v established in the laboratory needs to be treated with great care since the link between the elasto-plastic compressional response of the soil and the field values is complicated (Randolph 2016).

- The conclusions made at the Halsen-Stjørdal test site have been verified by successfully applying the suggested approaches to data from a NGTS research silt site.

- Even though the results for the two different natural silt sites revealed promising results, care must be taken when evaluating consolidation parameters in deposits, where partial consolidation may influence the results. To overcome problems of estimating c_h suggestions have been made to directly deduce estimates of permeability by empirical or graphical solutions.

8 Summary

Due to the presence of an unknown degree of partial drainage during standard cone penetration tests, the interpretation of CPTU results in silt deposits is complicated and not straightforward. Applying undrained or drained interpretation methods in silt may lead to a significant over- or underestimation of the geotechnical design parameters. Even though a few research test silt sites have been established in Norway and internationally over the last two decades, a standardized framework is still missing and conclusions are difficult to make, which emphasizes the further need to broaden the database of silts.

The present PhD study was conducted at the NTNU, to investigate the drainage behaviour of natural silt and the influence of partial drainage on the interpretation of parameters used for geotechnical design purposes. A research test site was established in Halsen-Stjørdal close to Trondheim, consisting of a 10 m thick silt deposit. Several CPTU tests were carried out, with penetration rates varying between 0.5 mm/s and 200 mm/s, to cover all possible drainage conditions. To study the consolidation behaviour of the silt, various dissipation tests were conducted after different penetration rates and at variable depth of interest in the silt layer. Soil samples were taken to study the material behaviour in the laboratory facilities at the NTNU.

The results from the laboratory study revealed a sandy, partly clayey coarse silt deposit with medium sensitivity and low plasticity. Sample disturbance is a considerable challenge. Several criteria for evaluating sample disturbance have been considered. The strain energy compression ratio concept was used and seemed to work well in these soil conditions, revealing modest but reasonable sample quality, given the difficult soil conditions at the site. Challenges were faced when building the soil specimen into the oedometer and triaxial testing apparatus caused by the rather layered, irregular and non-cohesive soil structure of the silt.

Test results showed a typical silt behavior, revealing oedometer plots of rather rounded shape without any indication of p_c' and the absence of a unique NCL line. The absence of a marked p_c' is believed to be a consequence of sample disturbance. The method proposed by Becker et al. (1987) could be successfully applied to the data and revealed reasonable OCR values between 1 and 2, being consistent with what was expected from the geological history of the area and supporting the application of this method in silty deposits. CAUC tests showed a strong dilative behaviour for the samples in the silt layer, developing dilative pore pressures at higher strains. Due to the dilative nature of the material, shear stress increases during shear without a clear maximum value which complicates the interpretation. Selecting design values for s_u at u_{max} or $\varepsilon = 1\%$ revealed the most promising results, ensuring no dilative pore pressures and representing a suitable method for practical application.

The interpretation of the varied rate CPTU tests revealed a strong rate dependency of the measured parameters, indicating decreasing q_t values and increasing negative Δu_2 values for an increased penetration rate. The investigation of the drainage situation at the silt site resulted in drained and undrained limits of $V < 0.2 - 0.3$ and $V > 40 - 50$ respectively, yielding partially drained penetration conditions when pushing the cone with the standard penetration rate. Although the use of undrained shear strength values in silts with $B_q < 0.4$ has been reported questionable by several researchers since silt rarely behaves as an undrained material, the undrained shear strength parameter still should be given some attention (Senneset et al. 1982; Long et al. 2010). To identify undrained strength values for the silt site from CPTU results, the N_{kt} approach revealed the most reliable results, being located within the range of results from the laboratory. Neglecting the effects of partial drainage results in an overestimation of about 20% of the undrained shear strength compared to the fully undrained penetration condition. Furthermore, the NTH method could be successfully applied to the silt data, indicating values of φ' comparable to laboratory results by applying a beta angle of + 20 degrees. The rate dependency study of φ' resulted in a significant underestimation when using a standard penetration test compared to the drained condition, underlining the importance of taking partial drainage into consideration when dealing with the interpretation of strength parameters in silt.

Dissipation tests have been analyzed after varied penetration rates. The non-standard Type III and IV dissipation tests dominate in the silt layer, yielding a strong rate dependency on the results. Dissipation tests carried out after slow penetration rates showed a drained behaviour whereas the faster tests indicate partial consolidation. Very fast tests are effectively undrained. For tests where $t_{50} < 100$ s, partial consolidation must be accounted for. Applying the approach by DeJong and Randolph (2012) yielded reasonable results for the coefficient of consolidation for the present silt, varying in average between 300 m^2/year and 600 m^2/year for a standard penetration rate. Ratios of the horizontal coefficient of consolidation from the field to the vertical coefficient of consolidation from the laboratory indicate factors of up to 3. Nevertheless, a comparison needs to be treated with care since the link between the elasto-plastic compressional response of the soil and the field values is complicated (Randolph 2016). The proposed methods to use in silty materials have been tested on CPTU dissipation data from a NGTS research silt site south of Norway. The results verified the findings from the present study, emphasizing on the importance of including partial consolidation effects when establishing design consolidation parameters in silts.

9 Conclusions

During the present PhD study, a natural silt test site at Halsen-Stjørdal was established. The deposit comprised of a 10 m thick silt deposit which can be characterized as a non-plastic coarse sandy silt. Various CPTU tests with penetration rates varying between 0.5 mm/s and 200 mm/s were conducted, accompanied by several dissipation tests at different depths in the silt layer. Soil samples were taken at the site and investigated in the facilities at the NTNU.

The present study carried out at NTNU contributes to broaden the database of the engineering behaviour of silts for Norway and internationally and increases the understanding of these rather difficult materials. The main conclusions from the present PhD study are as follows:

- The sample quality of the silt deposit was thoroughly considered using several sample disturbance criteria, leading to applying the approach suggested by DeJong et al. (2018) for intermediate soils, revealing modest but reasonable sample quality given the existing challenging soil conditions.

- The interpretation of oedometer tests in silt deposits is challenging and sample disturbance during the test procedure is likely to occur. Nevertheless, the interpretation of these tests from the present site showed promising results. In particular the method proposed by Becker et al. (1987) revealed consistent results for the interpretation of OCR and being in agreement with that expected from the geological history of the area.

- The determination of undrained shear strength in silt deposits is difficult due to the dilative nature of these materials. During shear, values of shear stress increase without a clear maximum value which is characteristic for many silt deposits. Picking design values of s_u at a defined strain level or at maximum pore pressure as proposed by Brandon et al. (2006) revealed promising and conservative results being well below the fully mobilized failure and ensuring no dilative pore pressures.

- Results from standard penetration rate CPTU tests support the laboratory test results, revealing a rather irregular silt deposit. The diagram developed by Schneider et al. (2008) gave the most consistent results, defining the deposit as intermediate and confirming the findings from the laboratory study.

- The rate study in the silt layer yielded a strong rate dependency of the measured parameters. Values of q_t decrease with increasing rate whereas the results for Δu_2 show an opposite trend with increasing high negative measured pore pressures with increasing rate. The study confirms the assumption of a strong rate dependency of the interpreted values for the soil parameters in silt deposits.

- Although the use of undrained shear strength values in silts is questionable since silt rarely behaves as an undrained material, the undrained strength parameter still should be given some attention (Senneset et al. 1982; Long et al. 2010). The N_{kt} approach is the most suitable method for the determination of s_u in silts from CPTU test results. Nevertheless, the rate study revealed an overestimation of the design parameters of 20 % if partial drainage is not considered.

- The NTH method can be recommended for silt deposits to deduce reasonable values of φ'. The investigation of the rate dependency of this parameter revealed a strong rate dependency, resulting in an underestimation of this parameter if partial drainage is neglected.

- For projects in silts where partial drainage is present during a standard rate CPTU test, it is highly recommended to conduct additional slow and fast rate tests representing drained and undrained conditions and to account for this during the determination of design values. The development of site-specific factors will lead to more realistic soil profiles and increase the confidence-interval of soil parameter determination.

- The various rate dissipation tests revealed a strong rate dependency. Non-standard dissipation curves generally prevail in silt deposits and the interpretation of these is not straightforward.

- The square root correction plot developed by Sully et al. (1999) is recommended to be used in silt deposits. This technique revealed the most consistent t_{50} results for non-standard dissipation tests. To account for partial consolidation, dissipation tests with $t_{50} < 100$ s are analysed by the method suggested by DeJong and Randolph (2012), producing reasonable results for the present silt site for the standard penetration rate. This procedure is highly recommended for silt deposits with non-standard dissipation curves and the presence of partial consolidation.

- Conclusions and recommendations made at the Halsen-Stjørdal silt site during the interpretation of the dissipation tests could be successfully applied to data from the NGTS Halden silt site, verifying the results achieved in the present PhD study.

- Consolidation analysis in silt deposits is difficult and not straightforward and care needs to be taken when evaluating design consolidation parameters in deposits where partial consolidation might have influenced the evaluation of soil parameters from CPTU tests. Schnaid et al. (2020) proposed a new approach to assess the degree of drainage by avoiding the use of c_h and introducing a normalized penetration rate $\overline{V}_h$. The application of this method showed promising results in the present silt site and further research is warranted in order to verify the application in these soil conditions.

10 Recommendations for future work

- The development of a consistent and well-founded interpretation framework for silts implies that the experiences and recommendations made in the present silt study need to be tested on various silt sites with different properties and compositions. Thereby more reliable correlations can be established, and confidence can be built on the interpretation of parameters.

- Soil sampling in silt deposits is challenging and further research in this area is needed to increase the sample quality and hence the quality of the developed soil data from the laboratory. Using block sampling techniques at the Halsen silt site are unlikely to be suitable for these soil conditions since the soil will fall apart as experiences have shown.

- Freezing techniques might be considered to improve the handling of the soil in the laboratory and hence the sample quality (Wride et al. 2000).

- For future tests it is recommended to use samples with an increased diameter, e.g. 75 mm instead of 54 mm in order to overcome problems of building and preparing the samples in the laboratory prior to testing. Using samples with a larger cross-sectional area can be especially effective for very scattered silt deposits since the representative cross sectional area is increased and the resulting geotechnical design parameters are more representative.

- Carrying out triaxial tests in loose, non-plastic and low cohesive silts is challenging and the risk of densifying the sample during the preparing procedures is high. By extruding the sample directly into the triaxial membrane, the risk of disturbing the samples might be considerably reduced (Wijewickreme and Sanin 2006).

- The method by Becker et al. (1987) to interpret preconsolidation stress values in silt deposits revealed promising results. Further work on the application of this method on silty materials is warranted for its use in various silt deposits.

- The two criteria (at defined strain level or at maximum pore pressure) developed by Brandon et al. (2006) revealed reasonable results for the interpretation of undrained shear strength from CAUC tests in silt. Nevertheless, these results need to be treated with caution, since possible densification might have influenced the results. More research on the interpretation of s_u in silt deposits it warranted and the influence of the penetration rate on the final design values needs further study.

- The application of the NTH method to determine the friction angle in silts showed promising results. More research is needed on the use of this method for establishing design values in silts and its rate dependency on the final design parameters. Furthermore, choosing a proper plastification angle as input parameter into the equation can be challenging and further research is warranted on its influence on the final results.

- The interpretation of consolidation parameters from CPTU dissipation tests in silt is complicated and further research is warranted on the effect of rate induced pore pressures in silt and on the determination of consolidation design parameters. Sensitivity studies of various parameters and their influence on the results can be a useful addition to improve the interpretation methods.

- Approaches precluding the determination of c_h (i.e. using permeability instead of c_h) to evaluate the drainage condition of a silt deposit seem promising and further research is warranted on different silt deposits with diverse properties to cover different silt behaviours.

- To increase the database and type of measuring parameters, it is recommended to carry out seismic and/or resistivity CPTU tests for future silt deposits. In addition, including dilatometer or full flow penetrometers into the field investigation program could be a valuable addition to the collected data sets (Carroll 2013).

- The pore pressure field developed around a penetrating CPTU cone in silt deposits is still not completely understood. To investigate the progress and distribution of pore pressures during various penetration rate CPTU tests, a pore pressure device has been designed by the PhD student and built at NTNU during the present PhD project, which measures the pore pressure at four different locations. The u_1, u_2 and u_3 position are included as well as one location further up the shaft. Figure 10-1shows the final design. This device can be applied to various silt deposits, carrying out CPTU tests with different rates and include dissipation tests to study the pore pressure fields that are generated during penetration. The application of this device requires careful filter saturation as well as a cable-based pushing system since the signals are transported through cables inside the cone device.

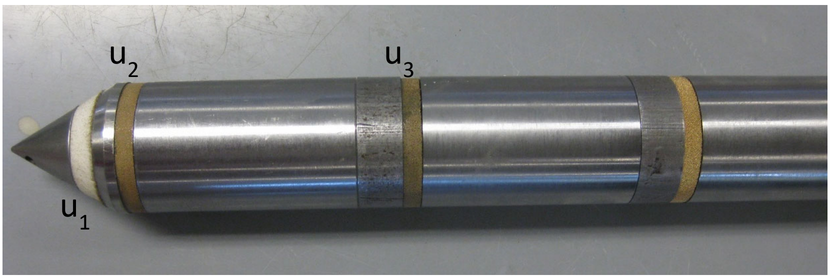

Figure 10-1 CPTU pore pressure device.

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Appendix

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Effect of piezocone penetration rate on the classification of Norwegian silt

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ABSTRACT: Interpretation of cone penetration tests (CPTU) in intermediate soils is complex due to partially drained conditions during penetration. In order to gain more insight into the material behavior of silt during a CPTU, an intensive test program was carried out in the field and several soil samples were taken and analysed. In addition a set of tests were performed in the laboratory under controlled conditions using a mini-piezocone (Fugro miniature CPTU, owned by the University of Colorado). The aim is to develop an improved interpretation basis for the CPTU in intermediate soils. The present paper gives an overview over the results obtained and shows how a change in penetration rate affects the response of the CPTU-readings. The data have been plotted into existing soil classification charts and the results from the field and laboratory investigations are compared. The Schneider et al. (2007) chart seems promising in separating the results from the different rate tests.

1 INTRODUCTION

Cone penetration testing in silty soils is difficult due to partial drainage occurring during a standard penetration rate. The CPTU is generally carried out with a rate of 20 mm/s ± 5 mm/s according to the International Reference Test Procedure (IRTP) of the International Society of Soil Mechanics and Foundation Engineering (ISSMFE). Thereby one assumes to achieve fully undrained conditions for clayey soils and fully drained conditions for sandy soils (Lunne et al. 1997). However, in intermediate soils e.g. silts, partial drainage is likely to occur during a standard penetration rate, which may lead to over- or underestimation of the geotechnical soil parameters either using analysis tools developed for fully undrained or drained behavior.

Researchers have shown that the presence of partial drainage has significant influence on the corrected cone resistance (q_t), the pore pressure (u_2) and to some extent the sleeve friction (f_s). Most of the research has been carried out under controlled conditions in the laboratory using a calibration chamber or a centrifuge and varying the penetration speed of the advancing probe using either a cone or full flow probes (Stewart & Randolph 1991, Randolph & Hope 2004, Silva & Bolton 2005, Schneider et al 2007, Paniagua 2014). Up till now, only a few field studies have been reported in the literature, as for example by Kim et al. (2008), Martinez et al. (2016), Poulsen et al. (2013) and Holmsgaard et al. (2016).

There may be several reasons for the lack of research in the field. Usually silty soils consist of a mixture of both finer and coarser materials. They often appear as small layers or lenses which makes it difficult to obtain undisturbed samples and complicates the handling of the material in the laboratory. Moreover, the interpretation of the CPTU results are complicated due to the rather complex micro fabric of the soil, e.g. Sandven (2002).

In order to study and understand the penetration processes taking place during a CPTU in intermediate soils, an intensive field study has been carried out by the Norwegian University of Science and Technology (NTNU) on a silt site in Stjørdal, close to Trondheim, Norway. In total 25 CPTU's were conducted with penetration speeds varying between 0.5 – 200 mm/s using a standard 35.7 mm cone and measuring the pore pressure at the u_2 position directly behind the cone. Several dissipation tests were done at different depths of interest where the penetration is paused and the pore pressure development is logged over a certain time interval. High-quality 54 mm steel samples were taken and carefully examined in the laboratory.

In addition, a series of CPTU's were performed in the laboratory under controlled conditions using a Colorado minicone (diameter 12 mm) and a fully saturated silt specimen. The soil sample was built inside a Plexiglas cylinder (d = 100 mm) with an internally padded layer of neoprene in order to compensate for boundary effects. By using a slurry deposition method, a silt sample is built into the cylinder and an overburden pressure of 80 kPa was applied. The CPTU tests were performed using three different penetration rates: 0.06 mm/s, 6 mm/s and 50 mm/s, representing a slow, standard and fast penetration rate respectively, Paniagua (2014).

The paper gives an overview over the basic laboratory results obtained and shows CPTU curves for the different penetration rates. The results are plotted in selected existing soil classification charts and compared to the minicone results from the laboratory. The aim of the present paper is to show the influence of varying the rate of penetration in intermediate soils and how they affect the interpretation of the CPTU results. The analysis of the dissipation tests is not presented in this paper.

2 SITE DESCRIPTION

2.1 *Halsen, Stjørdal*

The research test site consists of a thick silt deposit which is situated in the Stjørdal valley about 35 km east of Trondheim in Norway. During the Quaternary period, the Scandinavian Peninsula was fully covered by a massive icecap. However several periods of warmer climate caused the icecap to retreat temporarily. During the last de-glaciation, rivers of melt-water transported clays and silts into the sea. These soils are representative for the Halsen test site where the thickness of the sediments may reach 200 – 300 m over bedrock. Clayey materials govern these deposits but in some parts due to increased or irregular water velocities, silts and fine sands are more dominant. This is typical for the Halsen test site where the fine sediments are dominated by silt, with layers and pockets of clay and coarse sand (Sandven 2002).

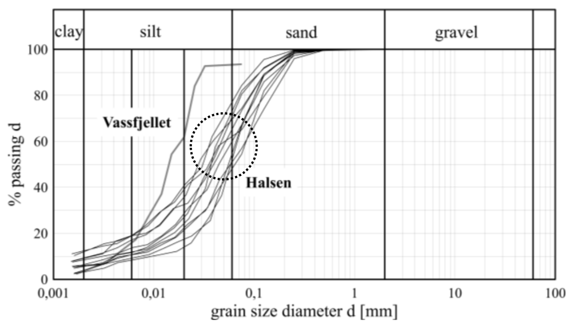

Figure 1. Grain size distribution: Halsen-Stjørdal and Vassfjellet

Grain size distributions have been determined and Figure 1 shows a summation plot of some of the test results including one test from the Vassfjellet silt. It is common Norwegian practice to use the recommendations made by the Norwegian Geotechnical Society (NGF) in order to classify the soil type NGF (2011). Herein the soil is defined as SILT if more than 45% of the grains are between 0.002 mm and 0.06 mm and less than 15% is clay (< 0.002 mm).

As can be seen from Figure 1, most of the soil from the Halsen test site consists of either sandy or clayey SILT with an average silt content of 55% and the majority of the particles falling into the coarse silt spectrum (e.g. 0.02 – 0.06 mm). The average soil grain density is about 2.66 g/cm^3. The soil gradation is middle to poorly graded with an average coefficient of uniformity of C_u= 17 which is defined as the ratio of d_{60} over d_{10}.

Figure 2 shows basic laboratory results for the Halsen test site. The water content varies between 20% and 35% with an average value of about 25%. Measurements of the bulk density are around 2.1 g/cm^3 on average. The distribution of the falling cone (fc) shear strength over depth shows a rather varied behavior with minimum values of 20 kPa and maximum values of 95 kPa. The distribution of the clay content over the depth shows an increase of clay content with depth.

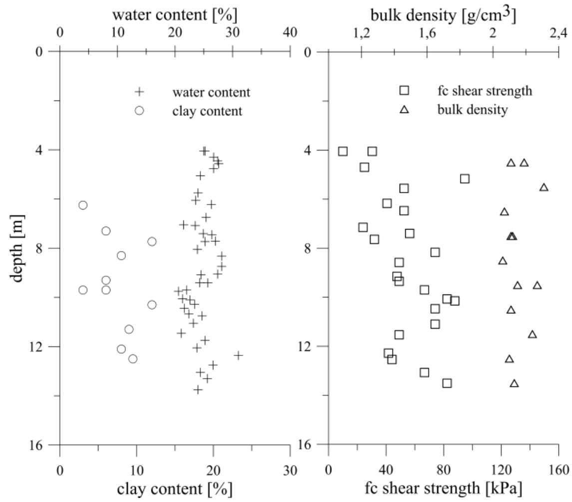

Figure 2. Overview over basic soil parameters

In general, there is little evidence of soil properties varying with depth. Findings from the laboratory reflect the natural variation of the silty soil and underline the challenges in handling and characterizing these soils in the laboratory and in the field. Figure 3 shows a cross section of a sample from 7.5 m depth opened in the laboratory. The sand and clay lenses, which in this case seem to form vertical columns rather than horizontal layers, are clearly visible as well as cracks and holes inside and along the sample.

The laboratory results underline the geological history of the test area. To ensure the highest possible sample quality, all samples taken in the field have been analyzed within 24 hours of sampling time. The focus of the present project was on the silt layer between 4 m and down to more than 13 m. The ground water table is located at about 2.8 m which fits well with the findings from Sandven (2002).

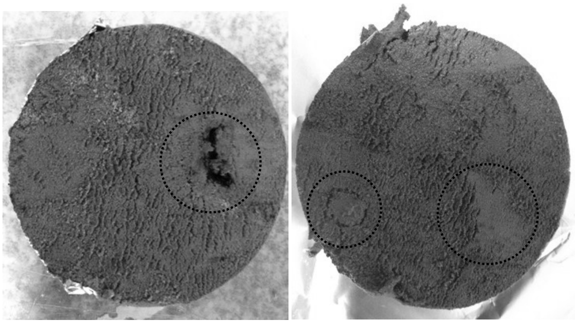

Figure 3. Cross section of soil specimen from 7.5 m, Halsen

2.2 *Vassfjellet*

The silt, which has been used in the laboratory study by Paniagua (2014), is from Vassfjellet, Klæbu, Norway, south of the Trondheim. The soil consists of a combination of glacier river deposits and peat. Due to its low cohesion, it was impossible to obtain undisturbed samples. Therefore disturbed samples were taken and rebuilt in the laboratory by a slurry deposition method.

The soil consist of a non-plastic, medium to coarse silt with a silt content of 92% and a clay content of 2.5%, see Figure 1. The silt from Vassfjellet is less sandy and clayey than the silt from the Halsen site. The soil grain density is 2.46 g/cm^3 and the organic content is lower than 2%. A maximum dry density of 1.57 g/cm^3 is obtained at 22% optimum water content and 95% saturation.

3 OVERVIEW OVER EXISTING SOIL CLASSIFICATION CHARTS

Soil classification charts, or more correctly soil behavior charts have been established since approximately 1965. It has to be stated that the application of these charts does not give accurate predictions of the soil type in terms of grain size distribution, but it is an indicator for the behavior of the present soil. Robertson (1990) stated that the classification charts are global and should only be used as a guide to define soil behavior. The general idea is that sandy soils usually generate high cone resistance and low friction ratios, whereas clayey soils have low cone resistance and high friction ratios (Lunne et al. 1997 and Robertson et al. 1986).

Ideally, all three measured parameters during a CPTU should be combined in order to achieve more reliable soil classification. Nevertheless, researchers are aware of the fact that the measured sleeve friction is often the less accurate and less reliable parameter of the three (for measurements below the groundwater table). In order to overcome the problems with the sleeve friction, Wroth (1984) and Senneset and Janbu (1985) were one of the first researchers to introduce the pore pressure parameter ratio B_q:

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \qquad (1)$$

where u_2 = measured pore pressure, u_0 = in-situ pore pressure, q_t = corrected cone resistance and σ_{v0} = total overburden stress. The original classification chart by Senneset & Janbu (1985) included the q_t against B_q whereas Senneset et al. (1989) revised the chart and used the normalized cone resistance Q_t according to Equation 2 instead of q_t.

The most widely used charts have been suggested by Robertson et al. (1986), which are based on either B_q or friction ratio versus q_t. In total 12 different soil behavior types were categorized to describe the different behavior and give a first indication of the drainage condition during cone penetration. Robertson (1990) later modified the existing charts to overcome issues related to CPTU soundings in greater depths (> 30 m) by normalizing the q_t and the friction ratio F_r in the following way (see Equation 2 and 3)

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \qquad (2)$$

$$F_r = \frac{f_t}{q_t - \sigma_{v0}} \qquad (3)$$

where $\sigma`_{v0}$ = effective vertical stress and f_t = corrected sleeve friction. Eslami & Fellenius (1997) established a classification chart used for pile design by applying data from more than 100 case histories in various soil conditions. The chart uses non-normalized parameters such as the effective cone resistance q_e ($q_e = q_t - u_2$) versus the measured friction f_t and divides the soil types into five different classes.

More recent developments are described by Schneider et al. (2008) where the classification charts are based on Q_t versus $\Delta u/\sigma`_{v0}$ and Q_t versus B_q to take into account the effects of partial consolidation and yield stress ratio. Robertson (2013) developed a revised version of the Robertson (1990) chart where it is possible to distinguish between drained and undrained penetration as well as contractive and dilative penetration behavior.

4 CPTU RESULTS

4.1 *Halsen, Stjørdal*

Several CPTU soundings are carried out at the Halsen test site between 4 to 14 m. Due to a very stiff and coarse top layer, it was decided to predrill the first 4 m. Different rates of penetration have been used, but only one group of tests has been selected for the present paper: the slow test with an average penetration rate of about 0.5 mm/s representing drained conditions, the standard test at 20 mm/s and the fast test with a penetration rate of 200 mm/s corresponding to an undrained penetration. In order to facilitate the interpretation of the CPTU results, the recordings have been smoothened by applying a moving average over a measuring interval of 100 mm, similar to Holmsgaard et al. (2016).

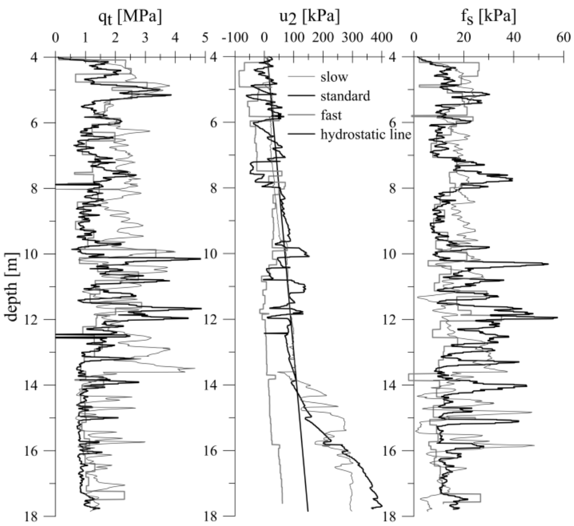

Figure 4. CPTU results Halsen (not smoothed)

Figure 4 shows the measured q_t, u_2 and f_s results for some of the tests carried out at Halsen. Between 4 to 14 m the soil consist of a rather coarse sandy silt layer with an average cone resistance at standard penetration rate of about 1,5 MPa and pore pressure values varying around the hydrostatic level at standard penetration rate. Below 12 m, the clay content increases resulting in a lower cone resistance and higher positive excess pore pressures exceeding the hydrostatic level when penetrating the cone at 20 mm/s (see also Figure 2).

The CPTU results pick up the different layering of the soil deposit rather precisely. The measured sleeve friction values are somewhat more scattered than the cone resistance and the pore pressure. Nevertheless, slightly higher friction values are observed during a slower penetration process. As stated by Lunne et al. (1997), the friction measurements have to be used with care due to less reliability.

Several researchers have observed similar trends for the cone resistance, i.e. increasing cone resistance values with decreasing penetration speed. This can be explained by partial consolidation effects occurring in front of the advancing cone during a slower penetration rate and allowing the pore pressure to dissipate and hence the cone resistance to increase (Kim et al. 2008 and Schneider et al. 2007). The drainage conditions during penetration change from drained (slow penetration) to partially drained (medium penetration rate) to undrained (fast penetration).

However, at Halsen the measured pore pressure profiles show opposite trends. During an increased penetration rate the pore pressure decreases and develops high negative values. During the slow rate a drained penetration is assumed by measuring almost hydrostatic pore pressures and a high cone resistance. Increasing the penetration rate leads to an increase in negative excess pore pressure. The measured pore pressure u_2 during a CPTU consists of the in situ pore pressure u_0 and the excess pore pressure Δu_2. The measured Δu_2 can further be separated into the following quantities (see Equation 4, Burns & Mayne 1998)

$$u_2 = u_0 + \Delta u_{2,oct} + \Delta u_{2,shear} \tag{4}$$

where $\Delta u_{2,oct}$ = mean octahedral normal stress component and $\Delta u_{2,shear}$ = shear component of the measured pore pressure (Wroth 1984, Baligh & Levadoux 1986 and Schneider et al. 2007). Shear induced pore pressures can have a significant effect in OC (over consolidated) clays and sandy silts, due to high negative shear occurring in the zone of intense shearing next to the penetrating cone. Further away from this zone the mean octahedral normal stress component dominates, resulting in larger positive excess pore pressures (Schneider et al. 2007).

Furthermore, the data has been applied to some of the soil classification charts discussed in this paper. The focus is on the 4 m thick sandy silt layer that is present between 4 to 8 m. Friction based classification diagrams have not been considered due to the scattered and rather less reliable results as described above.

The top line of Figure 5 shows the Halsen CPTU data (from left to right) plotted into the Senneset et al. (1989), Robertson (1990) and the Schneider et al. (2008) chart. The Senneset et al. (1989) chart defines the soil as a stiff clay-silt or a loose sand for all penetration rates. Robertson (1990) plots the data mostly into zone 4 and 5, which are defined as silt and sand mixtures. The results for the slow tests are plotted completely in the sand mixture zone, consistent with a drained penetration. The Schneider et al. (2008) diagram shows the most distinct classification where the data is plotted mostly into zone 3 corresponding

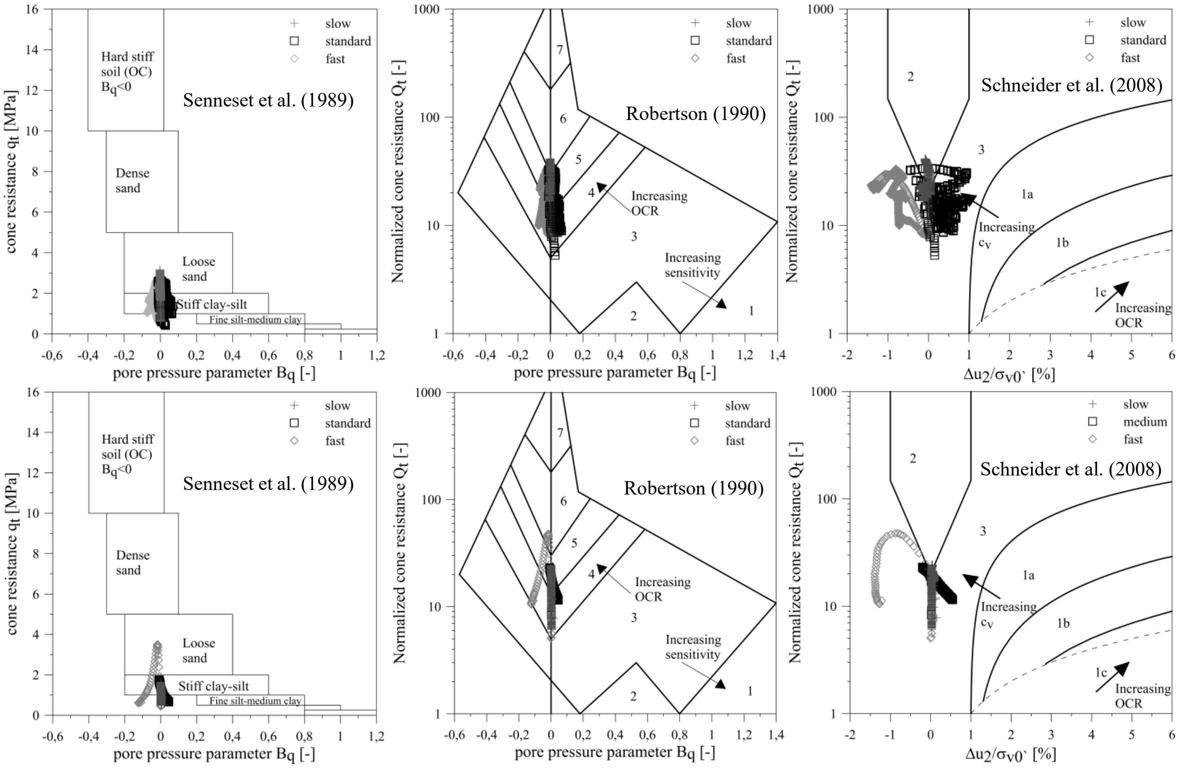

Figure 5. Soil classification charts: Halsen (top) and Vassfjellet (bottom)

to transitional soil behavior. Nevertheless results for the slow penetration test are plotted mostly in zone 2 which is defined as a drained sand.

4.2 *Vassfjellet*

Figure 6 shows the CPTU results for the tests carried out in the laboratory for the Vassfjellet silt including three different penetration rates (Paniagua 2014).

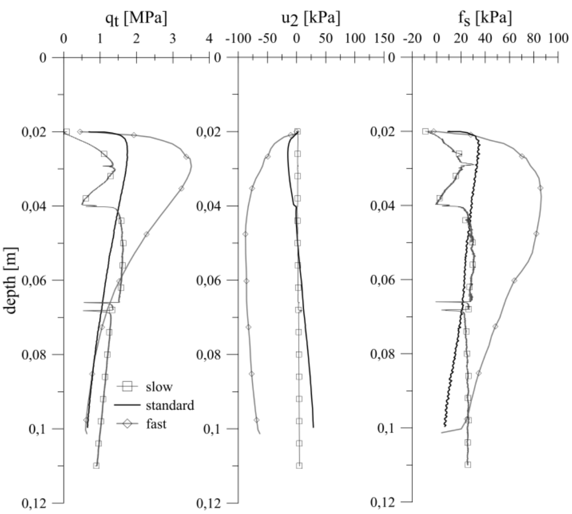

Figure 6. CPTU results Vassfjellet

The cone resistance shows the highest values during the fast penetration and the lowest results for the slow penetration, i.e. opposite to the Halsen test results. In contrast to the cone resistance, pore pressure values increase with increasing penetration rate, e.g. developing high negative excess pore pressure values at the fastest penetration rate, i.e. similar to the results from the Halsen test site. The sleeve friction values show a similar trend as the cone resistance, e.g. high values for fast penetration and lower values for the slow penetration rate.

The bottom row of Figure 5 shows the results for the Vassfjellet silt plotted into three different soil classification charts. The Senneset et al. (1989) chart classifies the data as silt or stiff clay for the standard and fast rate whereas the data for the slow test are plotted more towards the loose sand - soil behavior. Robertson (1990) defines the soil for all three penetration rates as silt and sand mixture. Schneider et al. (2008) plots the data data into the transitional zone showing an increasing coefficient of consolidation for the increasing penetration rate, Paniagua & Nordal (2015).

5 CONCLUSIONS

The present paper gives an overview over the results obtained during variable rate CPTU tests in silty soils both in the laboratory and in the field. The results show how varying the rate of penetration effects the CPTU readings and hence the degree of drainage and the classification of the soil behavior.

Overall, during a slow penetration, the results for the Halsen test site show a contractive penetration behavior, e.g. developing high cone resistance values and close to hydrostatic pore pressures. In contrast the fast penetration tests show lower cone resistance values and high negative excess pore pressures which corresponds to a more dilative soil behavior. This can be explained by the influence of the dilative behavior giving a shear induced pore pressure component during fast penetration which results in a zone of intense shearing next to the CPTU device.

The results from the laboratory study also show a dilative penetration behavior. During a slow penetration the cone resistance is low and the pore pressures are close to the hydrostatic level. Increasing the penetration rate leads to an increase in penetration resistance and an increase in negative excess pore pressure.

Plotting the CPTU results from the Halsen test site and Vassfjellet silt onto the different soil classification charts shows a general agreement of classifying the soil as transitional and intermediate. In particular the Schneider et al. (2007) chart seems to be promising and support the use of different penetration rates allowing one to make conclusions about the degree of drainage during penetration for identifying silts.

The results show a clear rate dependency of the measured parameters and hence emphasize the need of extended research in the interpretation field of intermediate soils. Further field tests are necessary to gain more insight into the understanding of intermediate soil behavior during a CPTU and an improvement of the interpretation methods is necessary in order to increase the accuracy of developed engineering soil parameters.

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Research article

Geotechnical characterization of Halsen-Stjørdal silt, Norway

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Abstract: The evaluation of geotechnical parameters for design problems in silty soils is complicated due to partially drained conditions and irregular soil structure, including small layers and pockets of both coarser and finer material. Many established methods to define soil parameters in clay and sand exist but little guidance is given to practicing engineers on how to interpret soil parameters for silty materials. This paper presents the results of an extensive laboratory and field test program which was carried out at a silt testing site Halsen-Stjørdal in Norway. The main objective is to broaden the database of the engineering behaviour of silts and to gain a better understanding of the behaviour of these soils. Cone penetration tests (CPTU) were performed and shear wave velocity measurements close to the site were used to supplement the CPTU results, confirming the coarse, silty nature of the deposit. In addition, several samples were taken using thin walled 54 mm steel sample tubes and examined in the laboratory by means of index, oedometer and triaxial tests. Recently developed methods to determine sample quality in intermediate low plastic soils were adopted and showed promising results. The interpretation of the oedometer tests was challenging due to the shapes of the curves. The results did not identify the yield or preconsolidation stress clearly partly due to the nature of the silt and partly due to sample disturbance. Triaxial test results on the silt showed a strong dilative behaviour developing negative pore pressures with increasing axial strain. The shape of the stress paths revealed no unique undrained shear strength of the silt. Although many researchers doubt the use of undrained shear strength (s_u) for partially drained materials, this parameter is still frequently used. Several methods were applied to determine values of an apparent s_u in the silt in order to provide an overview over the range of strength values. The results from this study contribute to the existing database and increase the understanding of silty soils.

Keywords: silt; CPTU; laboratory test; drainage; sample quality

1. Introduction

Interpretation of piezocone penetration test (CPTU) results in silty soils is complex due to the partially drained conditions during penetration and the natural variability of many deposits. Silty soils often consist of a mixture of both finer and coarser materials with little cohesion. The nature of these soils complicates the sampling and handling in the laboratory as well as the interpretation of the laboratory and CPTU results. Furthermore, little guidance exists for practicing engineers on how to determine the appropriate soil parameters for a specific geotechnical design problem. Up to now only a few field research projects on undisturbed silty soil have been published in the literature analyzing the behaviour of these soils even though they are present in many parts of the world. In Norway, several research sites exist which have been studied in detail over the last years. Sandven [1] studied a test site, underlain by a thick silt deposit, close to the present research site near Trondheim in Norway. Various field and laboratory tests were carried out and furthermore a case record of building settlement was reported and analyzed. Another glaciomarine silt test site in the western part of Norway close to Bergen (Os) was described by Long et al. [2]. The Os site was thoroughly investigated by using CPTU, in situ vane tests, total soundings and soil sampling using different soil samplers. A more recent comprehensive study was reported by Blaker et al. [3] which comprised work at the Norwegian Geotechnical Test Site (NGTS) silt test site at Halden approximately 120 km south of Oslo. The low plasticity clayey silt deposit was characterized in detail over several years including geophysical and geotechnical in situ investigation methods as well as intensive soil sampling using different sizes and samplers. All of these investigations highlight the lack of a practicable framework for establishing sample quality and determining design parameters for silty soils. Up to now engineers apply clay or sand based methods which often reveal questionable results for silty materials [4,5].

The present study carried out by the Norwegian University of Science and Technology (NTNU) incorporates a detailed field and laboratory study of a coarse sandy silt at a site in Stjørdal some 35 km north-east of Trondheim. One of the main objectives of the work was to gain a better understanding and insight into the behaviour of silty soils and to give guidance to practicing engineers on how to determine geotechnical parameters for design purposes. In order to obtain more knowledge, series of CPTUs were performed to study the behaviour of the silt deposit in situ. Shear wave velocity measurements close to the test area were used to support the CPTU measurements. A series of samples using thin walled 54 mm steel sample tubes have been taken in the field and analysed in the laboratory. Oedometer and triaxial tests were used to study the compressional behaviour of the silt and to obtain effective stress soil parameters and the undrained shear strength of the material.

The present paper initially gives some details of the background geology and then focuses on the outcome of the laboratory investigation program. Sample quality has been examined using the well known clay based volume change criteria [4] and more recently developed models employing strain energy and compression ratios [6]. The oedometer tests have been interpreted using the methods proposed by Janbu [7] and Becker et al. [8]. Undrained shear strength values have been determined by applying different criteria as suggested by Brandon et al. [9]. A series of standard speed CPTUs are presented and applied to several existing soil behaviour charts [1,10,11].

The present study provides information to geotechnical engineers working with silts as well as for researchers so as to broaden the database for establishing new correlations for these materials.

2. Halsen-Stjørdal site

The research site Halsen-Stjørdal is situated in the Stjørdal valley about 35 km north-east of Trondheim in Norway. The silt deposit is about 9 m thick (4–13 m). It is overlain by a very coarse and stiff 4 m sand layer. In order to carry out CPTU tests the top layer needed predrilling. A more clayey silt—sand layer is present underneath the silt deposit. No evidence of the depth to bedrock was found. The ground water table was at about 2.8 m depth.

2.1. Geology background

The geological history of the location has been studied in detail by Sveian [12]. The youngest glacial period, also named the Quaternary period, lead to an enormous icecap that covered the entire Scandinavia. Numerous phases of warmer and colder climate caused the icecap to retreat or advance respectively. This period of time had the most significant influence on the Norwegian geology especially with respect to the type and structure of the deposited materials. During the last de-glaciation, the Stjørdal valley was transformed into a deep and long fjord. Figure 1 shows a Quaternary map of the Stjørdal region indicating that the test area consists mainly of river deposits (the yellow parts on the map).

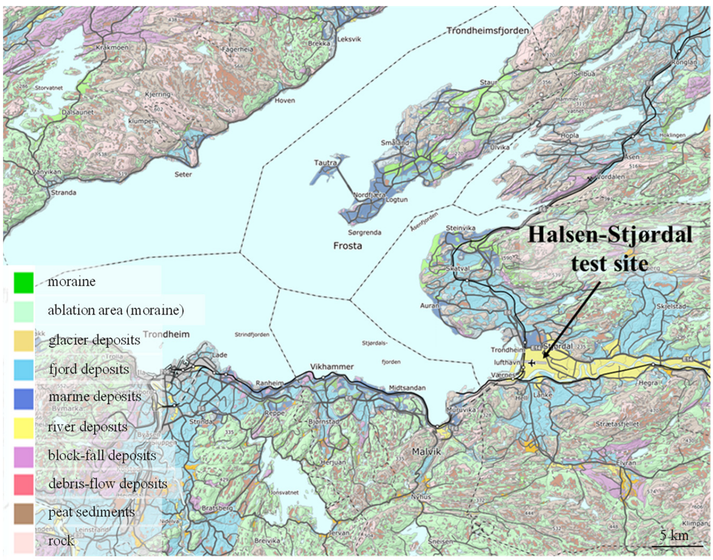

Figure 1. Quaternary map of Stjørdal from NGU [13] but modified by the authors.

Fine-grained particles as silt and clay were transported by rivers or melt-water into the sea forming thick deposits of these sediments reaching 200–300 m of thickness over bedrock. Clayey materials govern these deposits but in some parts due to increased or irregular water velocities, silts and fine sands are more dominant. This is typical for the Halsen-Stjørdal test site where the fine sediments are dominated by silt, with layers and pockets of clay and coarse sand forming a very irregular picture. However, there is no evidence that the Halsen-Stjørdal silt deposit has been overridden by any massive ice glacier since indications of glacial advances have only been found further up the Stjørdal valley. Furthermore, researchers found out that the location of the Halsen-Stjørdal site is not directly influenced by previous river erosion. Hence one can expect no distinct pre-consolidation due to erosion of the soils in the area [14].

2.2. Testing program

The field-testing program at the Halsen-Stjørdal site included 25 CPTUs with various penetration rates down to a depth of 18 m. Only the standard rate penetration tests are used for the present study. The test site covers an area of about 11 m × 8 m and the different CPTU tests have been distributed evenly in order to characterize the rather irregular soil deposit. A standard 35.7 mm friction cone was used and the pore pressures were measured at the u_2 position directly behind the cone head [15]. Furthermore, various samples were taken at 1 m intervals down to a depth of 13 m using thin walled 54 mm steel sample tubes in three different boreholes [5].

The boreholes surround the CPTU tests and are spread over the test area to detect spacial variations of the soil deposit. All samples used for this project have been handled carefully during the sampling process and when taken back to the laboratory. To ensure the highest possible sample quality, all samples taken in the field have been extruded in the laboratory within 24 hours of sampling time [16]. The laboratory investigation program included basic index testing as well as oedometer and triaxial testing. Figure 2 shows an overview map of the test site.

Figure 2. Overview map of the test area from NGU [17] but modified by the authors.

3. Field test results

3.1. Cone penetration tests

Several CPTUs have been carried out at the test field. Generally, the test was carried out with a standard rate of 20 mm/s ± 5 mm/s according to the ISO 22476-1 [18]. This approach assumes undrained penetration conditions for clays and fully drained conditions for sands. However, this is often not the case when dealing with silty soils. Partial drained conditions are most likely to occur during a penetration test in a silty material leading to over- or underestimation of the geotechnical soil parameters.

Figure 3 shows some of the results from the standard speed CPTUs from the site in terms of cone resistance (q_t) corrected for out of balance pore water pressure effects, pore pressure u_2 and sleeve friction (f_s). The top layer of the deposit consists of a very coarse and stiff sand which was difficult to penetrate without losing saturation of the pore pressure filters. Therefore, it was decided to predrill down to about 4 m. The hydrostatic pore pressure is drawn into the pore pressure plot with a ground water table at about 2.8 m depth. Measurements of the cone resistance are rather irregular detecting thin layers of various materials over short lengths. The average cone resistance is about 1.5 MPa with some peaks between 10 m and 12 m, which represents a typical value for a loose to medium stiff silt. The pore pressure measurements are somewhat more scattered reflecting the natural variability of the soil deposit. Changes in thin layers can easily be identified confirming the good quality of the tests. Especially above 12 m the values dilate rather frequently showing even negative pore pressures for some readings. The four CPTUs show very scattered sleeve friction results which increase with depth in the silt layer. Lunne et al. [15] stated that measurements of the sleeve friction are less reliable and have to be used with care. Nevertheless, the results confirm the coarse silty nature of the deposit. Below 11 m, the soil becomes more clayey which results in a reduced cone resistance to about 1 MPa and an increase in pore pressure to values which exceed hydrostatic levels. Even though measurements of the sleeve friction are scattered, one can notice a reduction in friction in the more clayey part of the deposit. Overall, the CPTU readings show a very layered and rather turbulent silt deposit which confirm the previous laboratory results from this study.

From the measured parameters during a CPTU test, several derived values can be used to deduce soil parameters and describe the soil behaviour using various published soil behaviour charts. The application of these charts is used as a guide to define soil behaviour rather than a definition of soil type or grain size distribution. It is recommended to combine all three measured parameters to achieve a more reliable soil classification. Senneset and Janbu [19] introduced the pore pressure parameter B_q ($= (u_2 - u_0)/(q_t - \sigma_{v0})$). Using B_q together with the cone resistance, Senneset et al. [1] established a soil behaviour chart without using sleeve friction measurements. One of the most widely used charts in the world has been suggested by Robertson et al. [20] and later revised [11]. These are based on a combination of either normalized cone resistance Q_t ($= (q_t - \sigma_{v0})/\sigma'_{v0}$) and B_q or friction ratio F_r ($= f_t/(q_t - \sigma_{v0})$). Recent developments in this field focus more in detail on detecting the behaviour of intermediate soils as for example the diagram by Schneider et al. [10] which is based on Q_t and $\Delta u/\sigma'_{v0}$ and accounts for partial consolidation stress effects. Figure 4 shows results of the application of the test data to the charts described above. Friction based diagrams have been neglected due to the rather less reliable results. The data has been divided into the sandy silt layer (4–8 m) and the clayey silt layer (8–14 m). Senneset et al. [1] defines the soil mainly as stiff clay to silt without clearly differing between the two layers although showing some tendency towards the loose silt. The Robertson's approach [11] plots the data mostly in zone 4 and 5 for the upper layer (silt and sand mixtures) and in zone 3 for the lower layer (clay to silty clay). The Schneider et al. [10] diagram plots the data for both layers into zone 3 corresponding to a transitional soil behaviour. All three soil behaviour charts work well in identifying the soil as a silty material, nevertheless the Schneider et al. [10] diagram shows the most consistent results classifying the soil as intermediate soil behaviour throughout the whole layer.

3.2. Shear wave velocity

Profiles of shear wave velocity (V_s) which were generated at the Halsen-Stjørdal site used both the Spectral Analysis of Surface Waves (SASW) method and the Multichannel Analysis of Surface Waves Method (MASW) [21]. The SASW method uses a single pair of receivers that are placed collinear with an impulsive source (e.g. a sledgehammer). The test is repeated a number of times for different geometrical configurations. The MASW technique was introduced in the late 1990's by the Kansas Geological Survey [22] in order to address some problems associated with SASW. The MASW method exploits multichannel recording and processing techniques that are similar to those used in conventional seismic reflection surveys. At Halsen 24, 10 Hz geophones spaced at 1 m intervals were used for MASW and a 5 kg sledgehammer was used as the impulsive source. Source receiver offsets were 0 m and 2 m. The MASW data was inverted using the software Surfseis, assuming a 10 layer soil model. A simple hand calculation approach was used to invert the SASW data assuming that the depth of penetration, z, of a particular wave is one third its wavelength (λ), i.e. $z = \lambda/3$ [23].

The SASW and MASW profiles obtained at Halsen are shown on Figure 5. The irregularity of the SASW profile is due to the simple method of inversion used. Nevertheless, the SASW values are of similar magnitude as the MASW data and serve to give confidence in the MASW results. The values of V_s from the MASW profile in the silt layer are on average about 125 m/s which is a quite low value and corresponds to a rather loose material (class E) according to NIBS [24]. Furthermore, V_s values are similar to the well-known loose Holmen sand deposit from Drammen, where average values of about 140 m/s were reported [25].

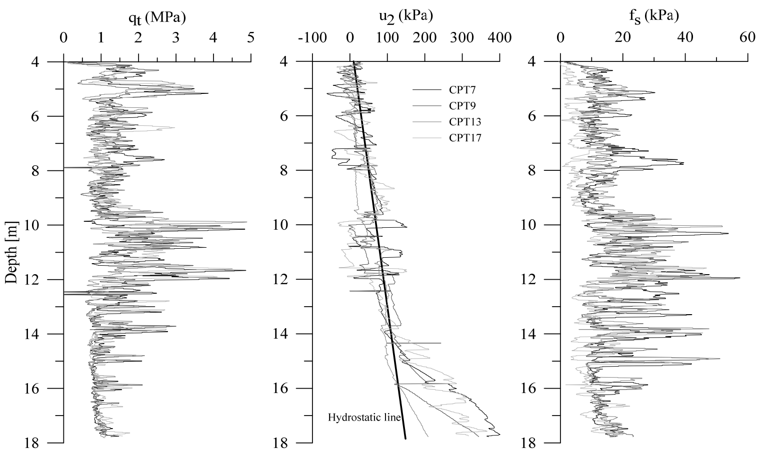

Figure 3. CPTU results from Halsen-Stjørdal.

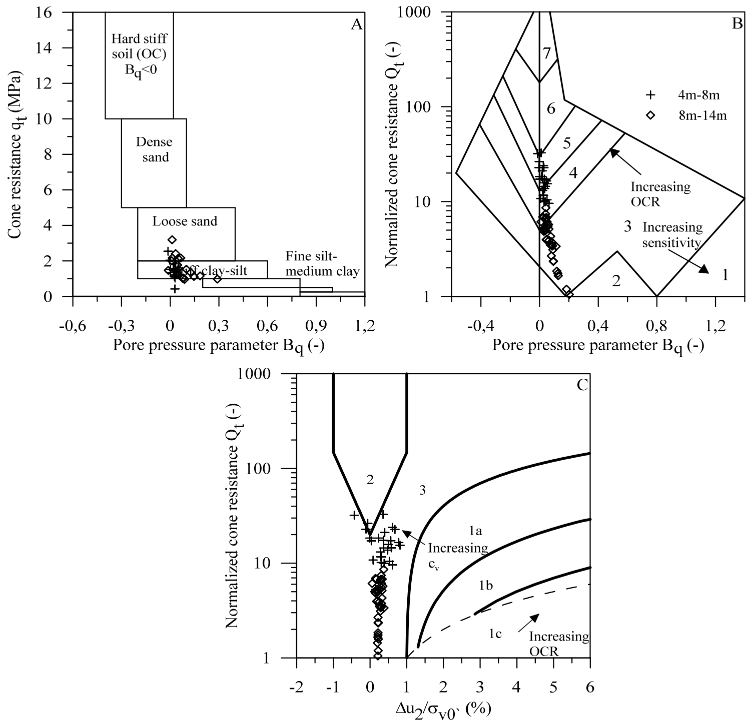

Figure 4. Halsen-Stjørdal data plotted into soil behaviour charts according to original data from: A) Senneset et al. [1], B) Robertson [11] and C) Schneider et al. [10] but modified by the authors.

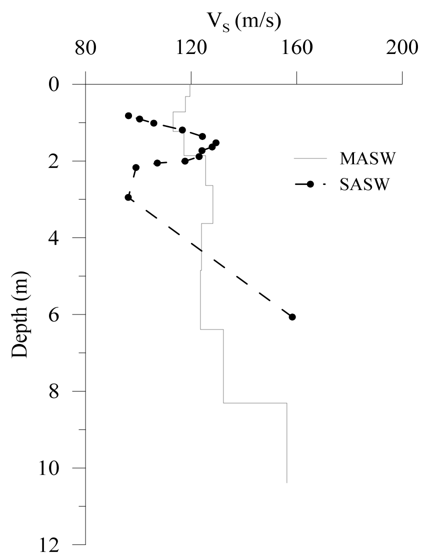

Figure 5. SASW and MASW profile Halsen-Stjørdal.

4. Laboratory results

4.1. Sample quality

The evaluation of sample quality is of high importance for practicing engineers and in research since the results of the laboratory test will be used directly for design and to establish new approaches and correlations. Using low quality soil data will eventually lead to an over- or underestimation of the geotechnical problem which may have crucial outcome. The two concepts that are mostly used worldwide are either based on evaluating the strain upon reconsolidation to in situ stress, named volumetric strain ε_{vol} [5] or on the change of the void ratio (e) normalized by the in situ void ratio $\Delta e/e_0$ [4]. The latter method has been widely used in Norway and is recommended for use in the assessment of sample quality [26]. The background for all these methods is that high quality samples experience less strain induced by sampling than low quality samples. Therefore, comparing the recompression strain that evolves during reconsolidation of the sample to the in situ stress state has become a useful tool. It should be noted that the criteria for the evaluation of sample disturbances originates from studies on marine clays (sampling depth 0–25 m) with an overconsolidation ratio (OCR) varying between 1–4 and plasticity index (IP) between 6% and 43%. Table 1 summarizes the criteria used to assess sample quality for clay materials depending on the OCR.

Table 1. Criteria to evaluate sample disturbance with original data [4,27] and modified by the authors.

Sample quality designation		$\Delta e/e_0$ criteria		
ε_{vol} (%)	SQD	OCR 1–2	OCR 2–4	Rating
<1	A	<0.04	<0.03	Very good to excellent
1–2	B	0.04–0.07	0.03–0.05	Good to fair
2–4	C	0.07–0.14	0.05–0.10	Poor
4–8	D	>0.14	>0.10	Very poor
>8	E			

Care has to be taken when applying these methods to soils whose parameters fall outside the above mentioned range [28]. In particular, silts may suffer from densification during shearing and sampling, showing an unrealistic low void ratio or change of volumetric strain upon recompression to in situ stresses. Therefore, the samples appear to be of high quality even though they are highly disturbed [3,29]. Applying these methods to silty materials is challenging and will often result in misleading conclusions and may not reflect the true quality of the specimen [30]. Other methods exist to identify sample quality which make use of measuring shear wave velocity or suction but are not part of the present study [31,32]. No well-founded framework currently exists to assess the sample quality in silts. Recent studies have shown that strain energy and compression ratios can be useful tools to assess sample quality in intermediate, low plastic soils [6]. This method is based on the concept developed by Becker et al. [8] and defines the work as the energy necessary to compress the soil to a given stress state. Equation 1 defines the work per unit volume for a given load increment, where σ_i' and σ_{i+1}' are effective stresses and ε_i and ε_{i+1} strains at the beginning (i) and at the end (i+1) of the current load increment:

$$\Delta W_{oed} = \left[\frac{\sigma`_i + \sigma`_{i+1}}{2}\right](\epsilon_{i+1} - \varepsilon_i) \quad (1)$$

The cumulative work increments (ΔW_{oed}) for loading to a certain stress level (σ_v`) can be plotted against $\sigma_v$` and be used to interpret the preconsolidation stress (p_c`) from an oedometer test. The in situ vertical effective stresses ranged from 70 to 140 kPa for the present study. Based on this concept, strain energy compression indices were established for both recompression (C_{rw}) and virgin compression (C_{cw}). C_{rw} represents the initial recompression strain energy index, which is defined from seating stress to in situ condition. Since the oedometer tests in the present study have no unloading reloading loop, C_{cw} has been evaluated from the stress interval between 2.5–5 p_c`[6].

The ratio of strain energy compression indices (C_{rw}/C_{cw}) is a useful indication of sample disturbance for a wide range of soil types and in situ conditions since it normalizes the influence of plasticity and is independent of the in situ stress and OCR. The database used to establish the quality criteria consists of soil mixtures with IP varying between 0 to 31% and maximum effective in situ stress of 1000 kPa. Table 2 shows the recommended ranges to assess the sample quality in terms of void ratio or strain energy [6]. Since C_{rw}/C_{cw} is the same as C_r/C_c for a linear stress-strain behaviour, the ranges in the table have identical numbers.

Table 2. Sample quality criteria using compression indices with original data [6] and modified by the authors.

$C_{r,w}/C_{c,w}$	C_r/C_c	Sample quality rating
<0.15	<0.15	High
0.15–0.40	0.15–0.40	Moderate
>0.4	>0.4	Low

In the present study, the data has been applied to the clay based volume change criteria as well as to the strain energy and compression ratio criteria using results from the oedometer tests. Figure 6 shows the sample quality assessment for the clay based volume change criteria. The volumetric strain defines all samples as poor quality whereas the void ratio defines half of the samples as good to fair and the other half as poor. None of them are characterized as very poor. It seems that the sample quality is decreasing with increasing depth which supports findings from other researchers who report that the approach using void ratio is effective stress dependent. This means that deeper samples are expected to have lager Δe independent of sample quality [33].

The results for strain energy and compression ratios can be seen on the right hand side of Figure 6 and on Figure 7. None of the samples have been classified as high quality whereas four samples are of moderate and the remaining ones are of low quality. Since the method is independent of the present effective stress level it also reveals reasonable results for deeper samples (see Figure 6). The sample quality could be increased by extruding the samples directly in the field. Thereby any influence on the sample quality due to transport, temperature changes, vibrations or shock loads can be eliminated [34]. Figure 7 shows the combination of the void ratio and the compression ratio approach. Even though most of the samples are defined as poor quality for the present material, none of them is defined as very poor. Given the fact that high quality samples in silt are difficult to obtain, the achieved sample quality is acceptable for this type of material, hence using the results from the oedometer and triaxial tests is reasonable.

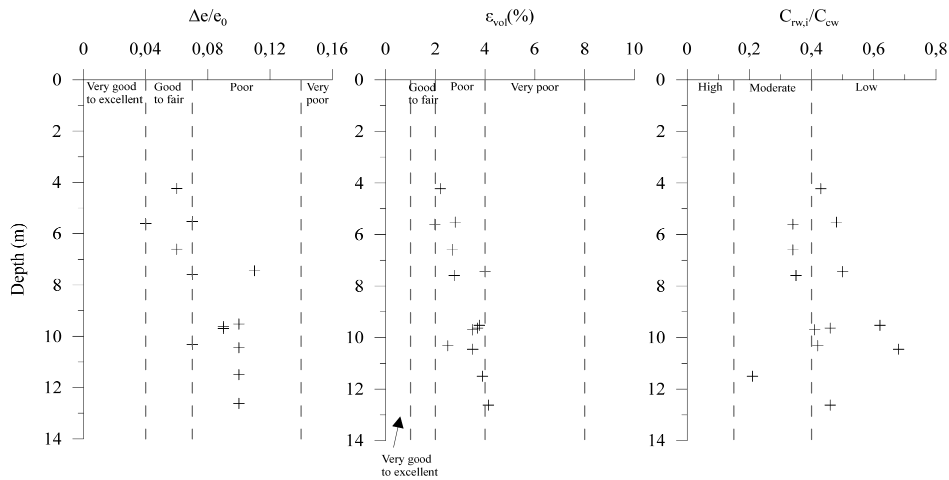

Figure 6. Sample quality assessment.

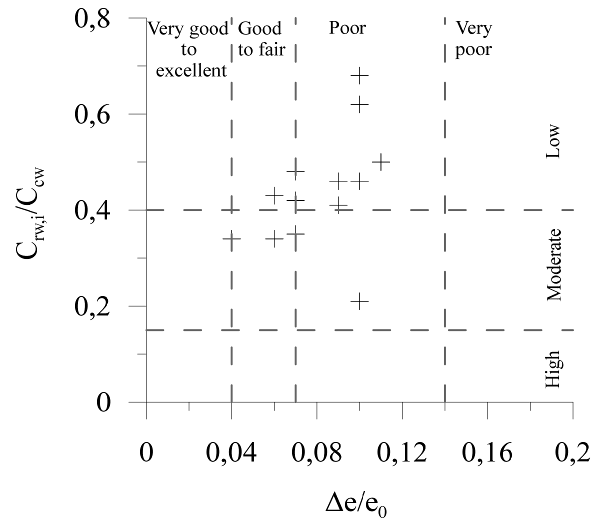

Figure 7. Sample quality assessment compression ratio criteria.

4.2. Index test results

Grain size distributions have been determined and Figure 8 shows a summary plot of some of the tests. In Norway, the recommendations made by the Norwegian Geotechnical Society (NGF) are used in practice to classify soil types [35]. A soil is hereby defined as a SILT if more than 45% of the grains are between 0.002 mm and 0.06 mm and if less than 15% of the particles are clay (<0.002 mm).

Figure 8 shows that the soil deposit consists of either sandy or clayey SILT with an average silt content of 55% and the majority of the particles falling into the coarse silt spectrum (0.02–0.06 mm).The average coefficient of uniformity (C_u) is 17 which reflects a well graded silt deposit. While the clay content varies with depth, reflecting the irregularity of the soil deposit, it generally increases with depth. Measurements of the soil grain density yielded an average value of 2.66 g/cm^3. Sandven [14] carried out a microscope study nearby the present research field which showed mainly angular grain shapes with some slightly rounded grains, which is typical for short-transported glaciofluvial sediments.

Figure 9 gives an overview of the index test results that were obtained from the laboratory tests. Due to the very low plasticity and very coarse structure of the sediment, it was not possible to achieve results for the Atterberg limits. Sensitivity (S_t) values from the falling cone test increase with depth being about 10 at the top of the silt layer and 28 at 14 m, which is characteristic for a medium sensitive silt. In Norway a material is defined as medium sensitive if the sensitivity is between 8 and 30 [35]. The measured natural water content of the deposit varies between 20% and 35% with an average content of 25%. Measurements of the bulk density are more scattered with an average of about 2.1 g/cm^3. The in situ void ratio, which was calculated using the water content and grain density measurements from the index test results, decreases with depth from about 0.7 to a minimum of 0.5.

Figure 10 shows a cross section of a soil specimen at 7.5 m depth from the test site showing small lenses of sand and clay as well as cracks and holes. Due to the non-uniform soil deposit, it was moredifficult to obtain representative soil parameters for the site. The findings from the laboratory investigations reflect the natural variation of the silty material of this deposit and emphasize the challenges in handling the soil in the laboratory and in the field. Overall, the results from the basic laboratory tests are in accordance to findings from previous work on a research site close to Halsen-Stjørdal [14]. There exist up to now two different research silt sites in Norway [2,3] in addition to the present test site, which report a different material behavior and composition, making this test site a useful extension to the existing database.

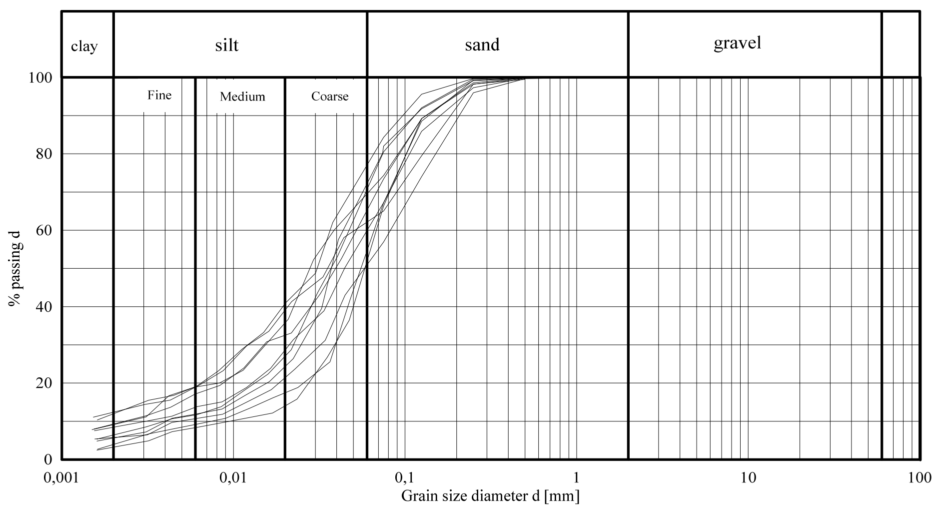

Figure 8. Grain size distribution Halsen-Stjørdal.

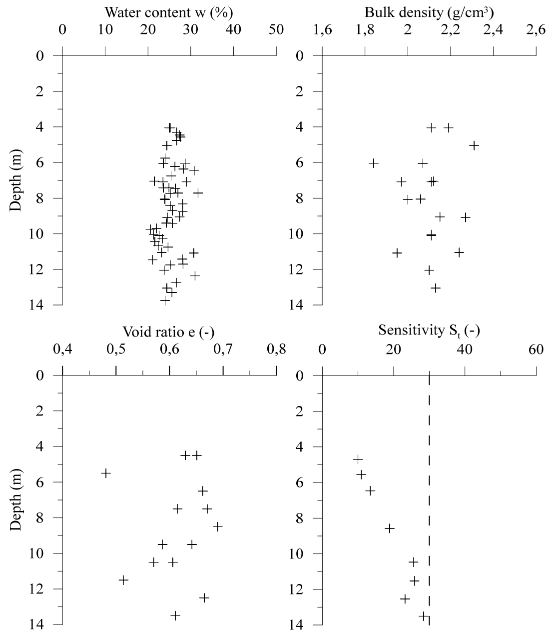

Figure 9. Basic soil parameters.

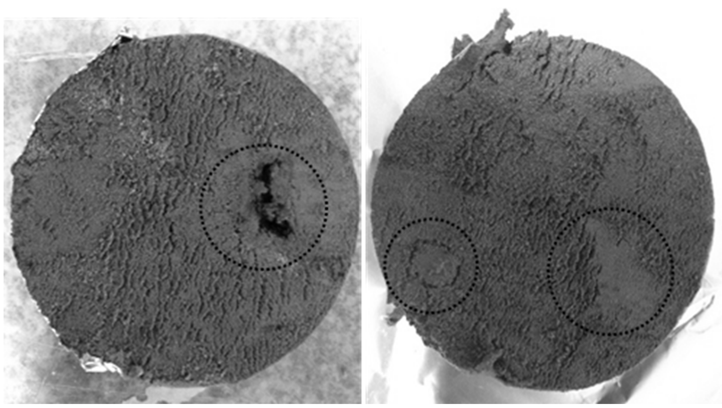

Figure 10. Cross section of a 54mm soil specimen at 7.5 m depth [36].

4.3. Oedometer tests

Interpretation of 1D compression curves for silty soils is difficult and, currently no international accredited framework exists to deduce deformation parameters. A series of oedometer tests were conducted on 2 cm thick specimens to study the vertical one dimensional compression behaviour of the silty material using the standard procedure proposed by Sandbækken et al. [37]. Due to the more confined procedure using a steel ring to support the soil and smaller size of specimen, handling the material during preparation of the sample prior to testing was not as challenging as for the triaxial tests. Constant rate of strain (CRS) tests with axial strain rates varying between 2%/h and 5%/h were carried out on samples taken from depths between 4–13 m. The pore pressure was measured at the bottom of the specimen allowing for a single drainage. The measured pore pressure decreased for all tests continuously and stabilized towards the end of the test procedure. Figure 11 shows the results in terms of σ_v` versus void ratio (e), constrained modulus (M) and strain (ε) respectively. The log $\sigma_v$` versus e plots are of rounded nature and no distinct preconsolidation stress can be identified. This might be partly due to the silty nature of the material itself and partly due to the influence of sample disturbance [2,38]. Overall, it can be assumed that especially the upper silty layer has experienced some densification as shown by the relative flat σ_v` versus e plots indicating little change in void ratio during further compression. Furthermore, the results do not lead to a unique Normal Compression Line (NCL), which is typical for many silts, but instead continue parallel to one another with increasing stress level. Several researchers studying the behaviour of silt, have reported this phenomenon and have suggested that complex factors govern the non-convergent behaviour of silty soils [31,39,40].

All test results show a steady increase of the constrained modulus with increasing σ_v` and no constant behaviour in the lower stress ranges or reduction around the preconsolidation stress as would be seen for clay soils. The behaviour observed is characteristic for silty soils [41]. In order to deduce modulus numbers (m) and M values for the silt, the tangent modulus concept has been applied to the data which defines M as the ratio of $\Delta\sigma_v$` over $\Delta\varepsilon$. M varies for different types of soils and hence can be expressed with the following general equation by means of m, the reference stress (σ_a) and an exponent number (a) which varies with soil type [7]:

$$M = m\sigma_a \left[\frac{\sigma`}{\sigma_a}\right]^{1-a} \tag{2}$$

Using an exponent number a of 0.25, the measured data can be reproduced fairly well. Figure 11 B and E show the results for two different depths applying the method described above by Janbu [7]. The results obtained coincide well with findings from other silt sites in Norway using the proposed method and confirm the gradual increase of M with increasing σ_v` of silty soils under compression [2,3,14].

Figure 12 shows how the modulus numbers deduced by the above mentioned method fit into the ranges suggested by [7]. This defines the lower and upper bounds for silt and sand respectively and suggests a rapid drop of m for increasing porosity (n). Modulus numbers for the present silt vary in the range of 45 to 110 with relatively low porosities between 36% and 43% which is characteristic for Norwegian silts and represent the lower bound of the silt range suggested by [7].

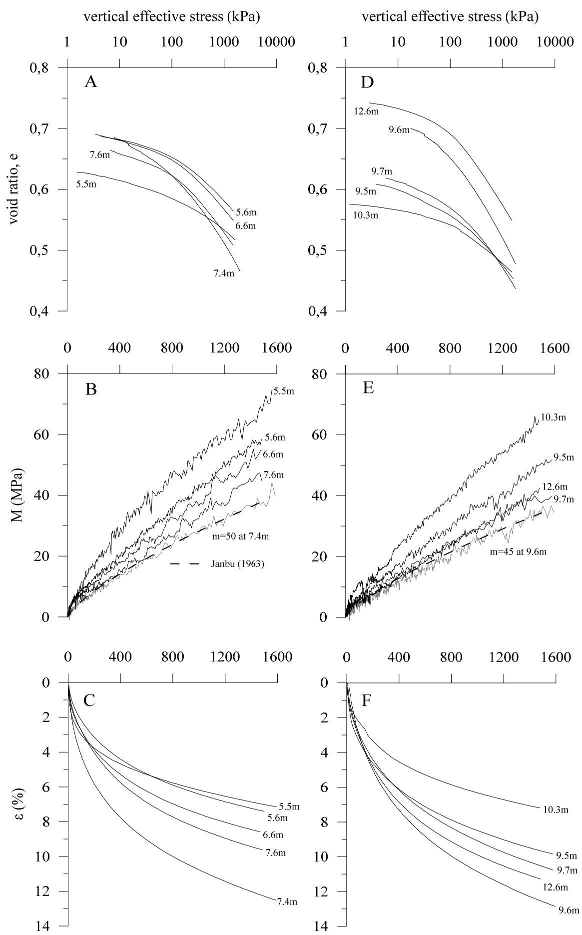

Figure 11. Oedometer test result: σv` versus e, M and ε: 4–8m (A–C) and 9–13m (D–F).

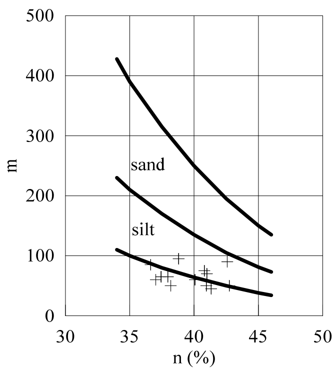

Figure 12. Modulus number (m) ranges for sand and silt over porosity taken from Janbu [7] and modified by the authors.

Furthermore the deformation Modulus (M), which corresponds to the constrained deformation modulus from an oedometer test, can be predicted by CPTU test data. Senneset et al. [42] stated that a linear correlation between the cone resistance and M_0 (Modulus at σ_{v0}') gave the best fit for the data by using $M_0 = 2q_t$ (for $q_t < 2.5$ MPa). Sandven [43] proposed later the following relationship for loose to medium dense silts using a dimensionless modulus number $\alpha_i = 6 \pm 2$:

$$M_0 = \alpha_i q_n = \alpha_i (q_t - \sigma_{v0}) \tag{3}$$

Figure 13 shows the M_0 values from the oedometer tests together with the predictions given by the CPTU results. The results from the oedometer test vary between 5 MPa and 11 MPa. Using $\alpha_i = 6$ reveal constrained modulus numbers from the CPTU measurements which are very similar to the oedometer test results.

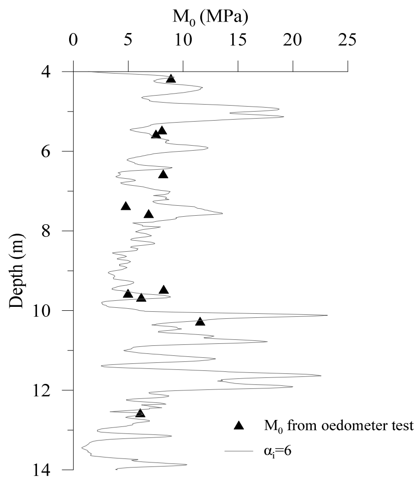

Figure 13. Oedometer and CPTU M_0 values.

Due to the particular compression behaviour of the silty material, existing methods to deduce p_c` often do not work. Geometric approaches based on axial strain plots are not useful due to the characteristic shapes of the curves [44]. The constrained modulus curves cannot be used due to the lack of a significant change of behaviour in the preconsolidation stress region as for clayey materials [7]. Several studies have been carried out to review existing methods used for clay and to investigate their application to silty soils [38,45]. In the absence of a reliable method for determining the preconsolidation stress in silts, the technique outlined by Becker et al. [8] based on the work criteria has been used in the present study. It is important to note that this method was developed for clay soils which exhibited classical "clay" like behaviour when plotted in log σ'_v versus e format, often showing a change of behavior around the preconsolidation stress level. As can be seen from Figures 11A,B this type of behaviour may not necessarily be observed in the case of silts. Although the technique shows promising results, further work on its use in silts is warranted. Figure 14 shows results for an oedometer test from 7.4 m depth, yielding a preconsolidation stress of about 130 kPa. Furthermore, the values for OCR over depth can be seen for the whole range of samples between 4 m and 13 m. Overall, it can be noted that the silt is normally to lightly overconsolidated with values between 1–2 and decreasing with depth. These findings support the geological history of the location and reflect previous results carried out in the area [14] Furthermore it supports the successful application of the technique by Becker et al. [8] for the material in the present study.

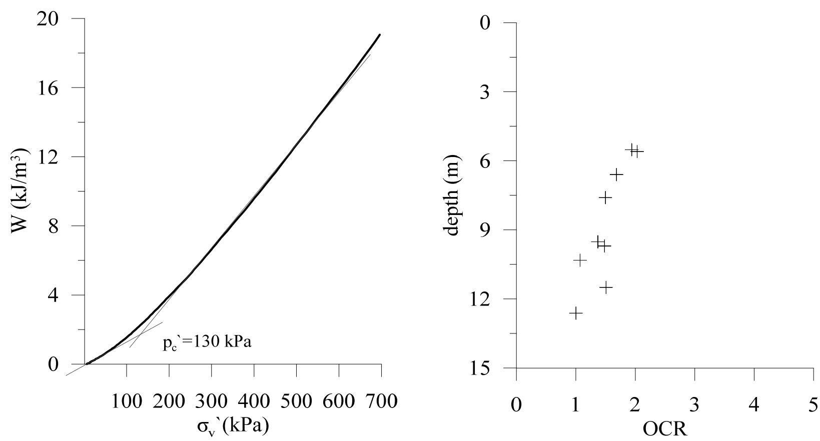

Figure 14. Work based concept by Becker et al. [8] for 7.4 m sample (left) and OCR for Halsen-Stjørdal (right).

4.4. Triaxial tests

Silty materials can show different behaviour patterns during undrained triaxial compression. Usually, positive pore pressures are generated at relatively low strains for both dense and loose silts. For loose silts, the peak pore pressure occurs at low strain levels and remains constant. In contrast a dense silt develops negative pore pressures with increasing strains. Examining the stress path, both dense and loose silts reach failure at relatively low strain levels. As shearing proceeds, the stress paths move parallel to the failure line. Due to the dilative nature of these materials, the undrained strength of silts increases during shear without a clear maximum value. This means there is no unique shear strength value. This might be one of the significant differences between clay and silt when it comes to triaxial compression [46].

Anisotropically consolidated undrained compression tests (CAUC) have been executed on all of the samples, applying the standard test procedure as described by Berre [47]. Challenges experienced during handling the low plastic material in the laboratory included the process of preparing the sample prior to be building it into the triaxial apparatus. It was difficult to keep the sample in place and lateral displacements occurred at times due to self weight of the specimen. For future tests it is recommended to try taking 75 mm samples instead of 54 mm and use an approach where the sample is extruded directly into the triaxial membrane [48]. By using 75 mm samples, a larger cross sectional area can be tested which is especially welcome for non-homogenous soil deposits. The larger the sample the more representative is the test result. Furthermore, it was difficult to keep the sample vertical during trimming due to the rather coarse structure. Due to the above discussed challenges, it cannot be precluded that densification of the samples might have occured during preparation. This issue will be addressed later when discussing shear strength of the material. All samples have been consolidated to in situ effective stress levels using an assumed coefficient of earth pressure at rest (K_0) of 0.5 for the site which is in accordance with work carried out by other researchers [14]. B-values were checked prior to consolidation and reached the appropriate value of >0.95. For most of the tests, a backpressure of 200 kPa is used and the specimen are sheared at a strain rate of 4%/hour. During consolidation back to in situ stress level, volumetric strains were recorded varying between 1.1% and 4.5%. Figure 15 shows the results of the tests in terms of shear stress (τ) or pore pressure (Δu) vs. axial strain (ε) or effective radial stress (σ_3\`). Part A–C represents results from the upper silt layer (4–8 m) whereas part D–F of Figure 15 shows the results from 9 m to 13 m.

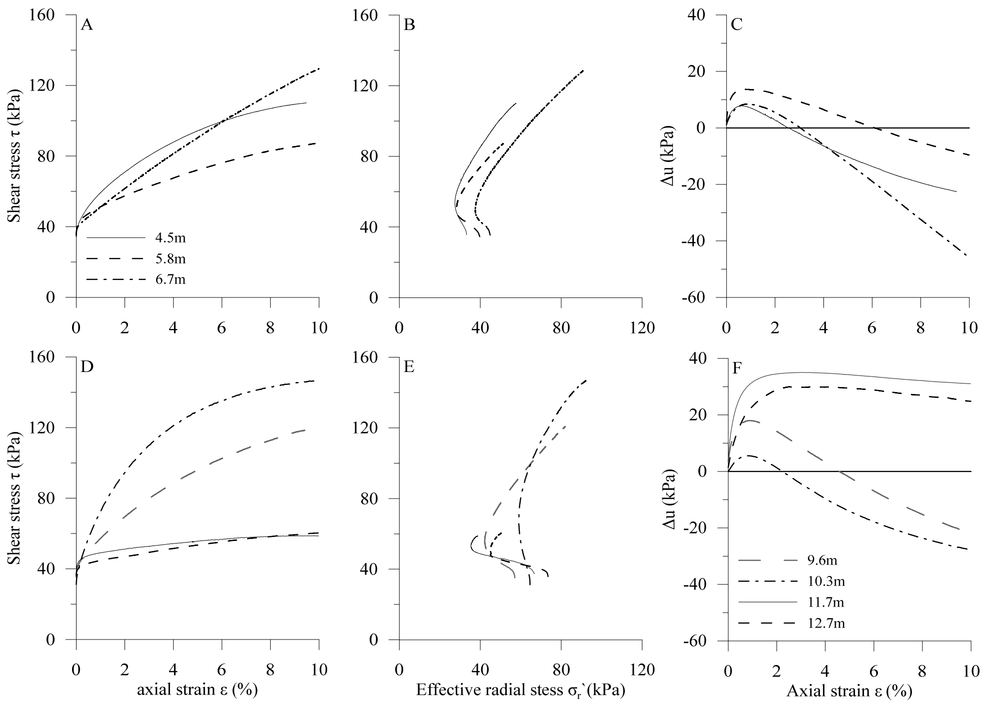

Figure 15. Triaxial test results: 4–8m (A–C) and 9–13m (D–F).

The results for the samples taken from 4 m to 10 m show a strong dilative behaviour with increasing τ and increasing ε without reaching a defined maximum shear. The stress paths reveal some contraction upon shearing followed by dilation with increased effective stress level continuing along a clear failure line. Up to a strain level of about 1%, the pore pressure increases for all samples. Subsequently it decreases and dilates into negative pore pressures at higher strains. The triaxial tests taken at deeper depths from 11 m and 12 m behave differently and show a contractive behaviour with a more distinct maximum shear strength and positive pore pressures at lager strains due to less coarser material and higher clay content at deeper depths.

Effective stress strength parameters in terms of friction angle (φ\`) and effective cohesion (c’) have been determined from the triaxial tests and are especially required for long-term stability analysis. Due to the varied nature of the silt at Halsen-Stjørdal, a range of friction angles have been found, varying between 34.2° and 38.7° with an average of 36.9° and cohesion value of 7 kPa for the silt layer. These results are consistent with findings from other silt sites in Norway. Blaker et al. [8] found a friction angle of 36° for the Halden silt in the Oslo region whereas Long et al. [2] reported a friction angle of 35° for the Os silt south of Bergen.

Despite the fact that many researchers doubt the use of undrained shear strength (s_u) for partially drained materials, this parameter is still frequently applied by many engineers [2]. Figure 16 shows shear strength plots for the test site. It is common practice in Norway to apply the SHANSEP method to establish a site specific s_u-profile by using s_u/σ_v`= S(OCR)$^m$. As recommended by Ladd et al. [49] a m-value of 0.8 and $S_{silt}$ = 0.2–0.3 were used. By including upper and lower bound values for OCR as indicated by the oedometer tests this resulted in $s_u/\sigma_v$` limits of 0.3–0.52 for this present silt. The results from index testing using the Swedish fall cone are varied as expected with values between 20 kPa and 80 kPa for the deeper samples, but most of the results plot close to the s_u/σ_v` limits. The results from the CAUC tests have been analysed using different strength criteria presented by Brandon et al. [9] for the purposes of deriving s_u:

(1) at peak deviatoric stress
(2) at ε = 1%
(3) at reaching the Mohr-Coulomb line
(4) at Δu = 0
(5) at maximum pore pressure (u_{max})

The results for the different strength criteria can be found in Figure 16. There is currently little guidance in literature which criteria to use for silty materials. Also, the dilative tendency of the low plastic silt makes it difficult to define general failure criteria. For the upper silt layer (4–10 m) the criteria (1) and (3) did not work since the shear strength values are unrealistically high due to the dilatoric behaviour of the material. The pore pressure criterion equal zero (4) could not be applied either since it occurs at high strains, which leads to unrealisticly high values of shear strength. Criterion 4 does however show less scattered results than criteria (1) and (3). Criteria (2) and (5) revealed the most consistent and realistic values for this site, since the limit strain has been chosen at 1% where no dilation occurs. This coincides for almost all tests with the strain at maximum pore pressure. Shear strength values for these two criteria follow the 0.3–0.52 s_u/σ_v` limits and are less scattered. Several researchers found the u_{max} criterion as the most promising one since it is on the conservative side being well below the fully mobilized failure and ensuring no dilative pore pressures [2,9]. It should be noted that the shear strength criteria work in a more consistent way for the lower layer, which is more clayey. Other than criterion (4), which cannot be applied due to contractive pore pressure development, all shear strength values plot well between the su/σ_v` limits. This demonstrates the application for these models in more contractive soils.

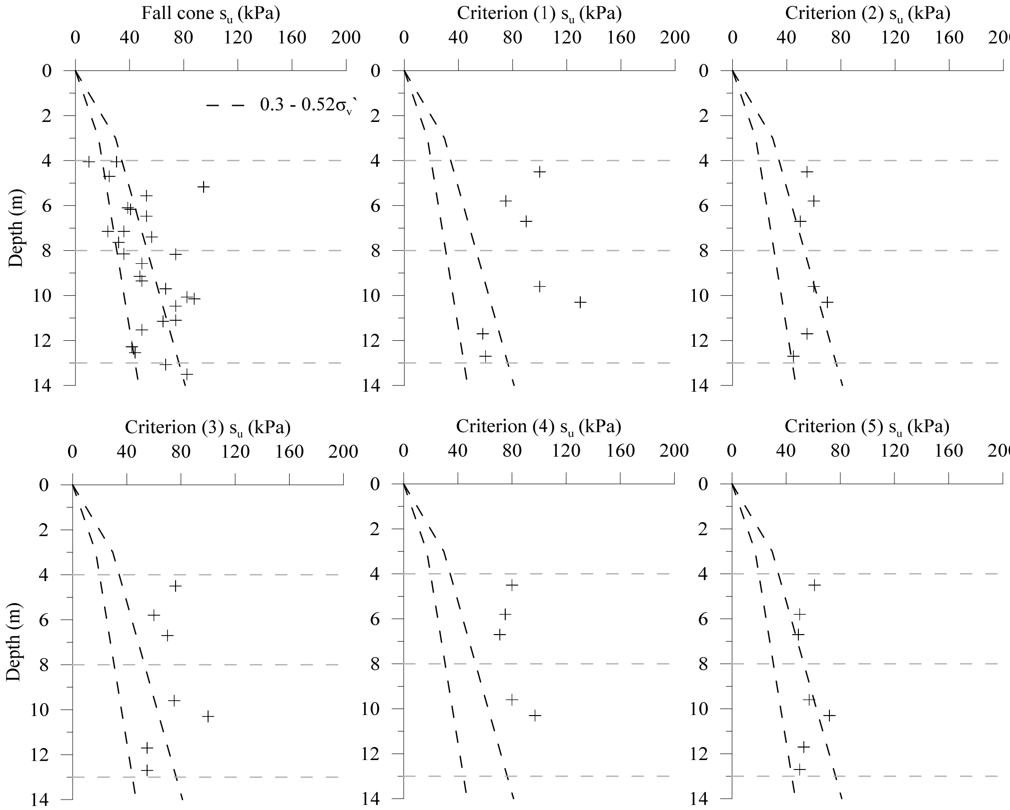

Figure 16. Undrained shear strength derived from falling cone and CAUC tests.

5. Summary and conclusions

Dealing with low plastic silts and silty material is difficult and challenging both in the field and in the laboratory. It is often assumed that silts behave either as clay or as sand but in reality neither of these reflect the real behaviour of these soils. The ongoing research project at NTNU incorporates an extensive field and laboratory study at a very low plasticity silt test site Halsen-Stjørdal in Norway. The present paper discusses results from various laboratory tests and results from CPTU tests. Sample quality predictions in silt have been discussed.No well-founded framework currently exist to assess the sample quality in these soils. A recent developed method using strain energy and compression ratios seems promising and shows quality results in consistence with the void ratio criteria, revealing a reasonable sample quality without being dependent on plasticity, effective stress level or OCR [6].

The CRS curves show mostly rounded shape curves with no distinct preconsolidation stress which is mostly due to the silty nature of the material itself and partly due to the influence of sample disturbance [2]. The Janbu 1D model [7] for compression has been successfully applied and revealed realistic modulus values for the site. Using the linear approach proposed by Senneset et al. [42] gave constrained modulus numbers from the CPTU which are very similar to the oedometer rest results and confirm the usefulness of this tool for silty soils. The technique developed by Becker et al. [8] to deduce preconsolidation stress revealed reliable results and showed a lightly overconsolidated material for the silt deposit which is in agreement with the geological history of the area and earlier reported values by Sandven [14]. For future work, it is worth looking deeper into the method proposed by Becker et al. [8] and apply strain calculations as suggested by Janbu [7].

Triaxial test results revealed a strong dilative behaviour and significant negative pore pressures were developed with increasing axial strain. The lower more clayey samples showed a more contractive behaviour in triaxial compression. Due to the dilative nature of the materials, the undrained shear strength of silts increases during shear without reaching a clear maximum value, e.g. no unique shear strength value exist. This might be one of the most significant differences between clay and silt when it comes to triaxial compression and identifies the dilemma many practicing engineers face when dealing with silty soils. The present study shows that picking s_u at a defined maximum strain (here 1%) or at maximum pore pressure ensuring no dilation gives values close to the 0.3 to 0.52 σ_v\` lines as proposed by Ladd [49]. Given the fact that possible densification took place during sampling, one has to be careful when choosing parameters for geotechnical design purposes and especially the CAUC tests need to be treated with caution. Therefore, choosing strength values on the conservative side is appropriate.

For future tests it is recommended to use bigger samples, e.g. 75 mm instead of 54 mm in order to be able to overcome problems related to building and preparing the samples for testing. Furthermore, a larger cross sectional area can be tested which is especially welcome for non-homogenous soil deposits. Using an approach where the sample is extruded directly into the triaxial membrane is highly recommended [48]. Due to the complex soil structure of the silt deposit, taking block samples would be impossible. The soil would fall apart and it would be difficult to get the block out of the borehole. Furthermore, trimming the sample in the laboratory would be difficult due to the coarse and partly loose structure.

The piezocone penetration test is a reliable tool in silt where shear induced pore pressures due to dilation have significant effects. The Schneider et al. [10] chart suggests the soils investigated are intermediate soils which is consistent with the general behaviour of the deposit. Shear wave velocity measurements have been used to confirm the loose structure of the soil deposit. Due to the issues mentioned above concerning densification and disturbance during sampling, more emphasis should be placed on in situ testing methods in future investigations of deposits such as these.

The present study carried out at NTNU contributes to broaden the database of the engineering behaviour of silts for Norway and internationally and increases the understanding of these rather difficult materials. More research is needed on natural silty soils to be able to establish a reliable framework and to form recommendations for practicing engineers.

Conflict of interest

All authors declare no conflict of interest.

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