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Suction Anchor Penetration

Estimating Penetration Resistance Based on CPT Sleeve Friction

André Mikkelsen Nårstad

Geotechnics and Geohazards
Submission date: June 2017
Supervisor: Gudmund Reidar Eiksund, IBM
Co-supervisor: Gisle Håland, Statoil

Norwegian University of Science and Technology
Department of Civil and Environmental Engineering

NORWEGIAN UNIVERSITY OF SCIENCE AND TECHNOLOGY
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

Report title: **Suction Anchor Penetration; Estimating Penetration Resistance based on CPT sleeve friction**	Date: **10.06.2017** Number of pages (including appendices): **90** Master thesis x
Name: André Mikkelsen Nårstad	
Professor in charge/supervisor: Proffesor Gudmund Reidar Eiksund, NTNU	
Other external professional contacts/ co-supervisor: Gisle Håland, Statoil	

Abstract:

Oil and gas is number one commodity when it comes to transportation and energy, and with increasing worldwide demand for oil the need to utilise deeper offshore oilfield are of the essence. Great depths present challenges not only technologically but also economically. With the stagnating oil prices from the last quarter of 2014 forces the oil industry to find the best economical solutions. One economical solution is the use of suction anchors to hold floating structures in place, this method also solves the problem with producing oil in ultra-deep waters.

This thesis has been looking at recommended methods to estimate suction needed to install suction anchors at depth which gives suitable bearing capacity. Necessary suction has also been estimated with a new proposed method, a method utilising measured sleeve friction from cone penetration test. For a method to be useful it should estimate suction as close to the real suction measured, and yet be simple enough to keep investigation cost low.

All the methods utilised shows that remoulded shear strength plays a crucial role in estimating suction. The methods are based on two soil investigations techniques; values from laboratory test and cone penetration test measurement.

The new proposed method indicates that using sleeve friction directly (not corrected for pore pressure) for Gjøa field gives estimations twice as high as what measured. When sleeve friction is reduced to fit the measured suction, it is found that a fixed value with layer and depth shows promising results. The reduction of sleeve friction shows potential correlation with liquidity index.

The main conclusion from this thesis is that estimating suction for suction anchors can't be conducted solely based on cone penetration test measurement, and laboratory test must be conducted. With more research, this method could possible reduce the amount of costly laboratory test due to the use of index test.

Keyword:

1. Geotechnics
2. Suction anchors
3. CPT sleeve friction
4. Remoulded shear strength

André Nårstad
(signature.)

MSc Thesis

TBA4900 – Geotechnical Engineering

June 2017

By

André Mikkelsen Nårstad

Title: Suction Anchor Penetration; Estimating Penetration Resistance Based on CPT Sleeve Friction

BACKGROUND

Extracting oil and gas from deeper and smaller oil field has challenged the oil and gas industry to be innovative. One solution to solve this problem is the use of floating platforms and subsea installation, which all uses suction anchor technology. Suction anchors is forced down by under pressure contributed by pumps, which suction needed to penetrate the soil varies from location to location. Suction anchor has been used for a few decades, however how to estimate necessary suction is mainly done by a few approaches. One large factor to estimating suction is the friction contributed by the anchor skirts. This thesis considers estimating suction with the use of sleeve friction obtained by cone penetration tests.

PROBLEM FORMULATION

When estimating necessary suction to reach a given depth which gives enough bearing capacity is today estimated by the recommendations given by Det Norske Veritas and American Petroleum Institute. The methods are reliable, however they need thorough knowledge about the ground conditions throughout the depth. Remoulded undrained shear strength becomes one of the larger uncertainties when estimating resistance. The goal with this thesis is to check whether the use of measured CPT sleeve friction is a viable option when estimating penetration resistance. Gisle Håland at Statoil proposed the problem formulation for this thesis.

CONSIDERATION

The problem formulation changed in the middle of March. The original proposed problem formulation was to consider possible causes to deviation between measured suction and estimated suction, where the remoulded shear strength is assumed to be equal to $s_u^C \cdot 1/S_t$. It was changed to look whether the sleeve friction from CPT could be used as the frictional force contribution alongside the skirts. This change lead to less time investigating other installation fields, due to the long process of digitising CPT data from reports.

Department of Civil and Environmental Engineering, NTNU

Date: 6/6-2017

Gudmund Eiksund

Gudmund Reidar Eiksund
Professor
Geotechnical division NTNU

Preface

The master thesis is written during the 4^{th} semester at Norwegian University of Science and Technology in Trondheim and counts for 30 points of study. Duration of the work is set to 20 weeks, and last from 15^{th} January to 10^{th} of June. The thesis marks the last semester of the master program *Geotechnics and Geohazards*.

The idea for this thesis was made by Gisle Håland from Statoil, which also have been the co-supervisor during this thesis.

The use of suction anchors has increased the last decades, and have become a widely used method mooring floating offshore platforms to the seabed. When installing suction anchors the need to know necessary suction to reach wanted depths is of the essence. At the time, this is estimated based on thorough soil investigation which increase installation costs. The thesis goal is to check if there is a possibility to estimate suction based on measured sleeve friction from CPT.

André Nårstad

André Mikkelsen Nårstad

Trondheim, 10^{th} June 2017

Acknowledgment

I would like to send my gratitude's to Gisle Håland at Statoil for suggesting this interesting topic. Without his support and guidance this thesis would have been impossible to complete this thesis.

I would also thank Professor Gudmund Reidar Eiksund at NTNU, for the guidance I have received and for holding his door open at any time when I needed answer to questions.

I would also send my thanks to my fellow students, for the good times during lunch and contribution to a good social environment during the writing of this thesis.

A.N.

Abstract

Oil and gas are number one commodity when it comes to transportation and energy, and with increasing worldwide demand for oil the need to utilise deeper offshore oilfield are of the essence. Great depths present challenges not only technologically but also economically. With the stagnating oil prices from the last quarter of 2014 forces the oil industry to find the best economical solutions. One economical solution is the use of suction anchors to hold floating structures in place, this method also solves the problem with producing oil in ultra-deep waters.

This thesis has been looking at recommended methods to estimate suction needed to install suction anchors at the depth which gives suitable bearing capacity. Necessary suction has also been estimated with a new proposed method, a method utilising measured sleeve friction from cone penetration test. For a method to be useful it should estimate suction as close to the real suction measured, and yet be simple enough to keep investigation cost low.

All the methods utilised shows that remoulded shear strength plays a crucial role in estimating suction. The methods are based on two soil investigations techniques; values from laboratory test and cone penetration test measurement.

The new proposed method indicates that using sleeve friction directly (not corrected for pore pressure) for Gjøa field gives estimations twice as high as what measured. When sleeve friction is reduced to fit the measured suction, it is found that a fixed value with layer and depth shows promising results. The reduction of sleeve friction shows potential correlation with liquidity index.

The main conclusion from this thesis is that estimating suction for suction anchors can't be conducted solely based on cone penetration test measurement, and laboratory test must be conducted. With more research, this method could possible reduce the amount of costly laboratory test due to the use of index test.

Contents

List of Figures

List of Tables

Chapter 1

Introduction

1.1 Background

Oil and gas is number one commodity when it comes to transportation and energy, and with increasing worldwide demand for oil the need to utilise deeper offshore oilfield are of the essence. Great depths present challenges not only technologically but also economically. With the stagnating oil prices from the last quarter of 2014 forces the oil industry to find the best economical solutions. One economical solution is the use of suction anchors to hold floating structures in place, this method also solves the problem with producing oil in ultra-deep waters.

Suction anchors is essentially a big metal cup which is installed with pinpoint accuracy on the sea bottom. It's cheaply made, nevertheless it's a considerable risk factor when looking at holding the structure afloat. Knowing the bearing capacity is crucial. The measured installation data should confirm the estimated bearing capacity, however, finding a solution to estimate the installation data before installing the anchor will benefit the project economically.

Problem Formulation

When estimating necessary suction to reach a given depth which gives enough bearing capacity is today estimated by the recommendations given by Det Norske Veritas (DNV) and American Petroleum Institute (API). The methods are reliable, however they need thorough knowledge about the ground conditions throughout the depth. Remoulded undrained shear strength becomes one of the larger uncertainties when estimating resistance. The goal with this thesis is to check whether the use of measured CPT sleeve friction is a viable option when estimating penetration resistance.

1.2 Objectives

The main objectives of this thesis are

1. Estimate suction after DNV CPT method from 1992.
2. Estimate suction after DNV RP E303 from 2005.
3. Try to find a possible option for estimating suction with the use of sleeve friction from CPT measurement.

1.3 Limitations

The problem formulation for this thesis was changed in the middle of march. The task went from looking at factors affecting the difference in estimating skirt resistance with $s_u^C \cdot 1/S_t$ and measured suction, to find a usable method which utilise sleeve friction from CPT. This affected the time left to investigate soil conditions and gather data from other fields.

The other limitation was accesses to digital data from measured CPT. All CPT data use in this thesis is manually digitised from soil condition reports. This process is time consuming since all point between needs to be interpolated.

1.4 Structure of the Report

- **Chapter 2** - Foundation of Floating Offshore Structures
  - This chapter gives a brief introduction to foundation of floating offshore structures and how suction anchors are use when mooring floating platforms. A general overview of how suction anchors and caissons is installed is also presented.
- **Chapter 3** - Suction Anchor Penetration Analysis
  - This chapter presents the methods used to estimate the results presented in chapter 6.

- **Chapter 4** - Cone Penetration Test
  - This chapter presents CPT in general and factors affecting the CPT measurement.
- **Chapter 5** - Gjøa Field Data
  - This chapter presents the soil conditions at Gjøa and anchor properties. This information is used in combination with the methods described in chapter 3.
- **Chapter 6** - Results
  - This chapter presents the results and findings with a brief discussion.
- **Chapter 7** - Summary and Conclusions
  - This chapter is a summary of the thesis, discussions are made and recommendations for further work.

Chapter 2

Foundation of Floating Offshore Structures

During the 70's the use of floating production structures was introduces by the industry. The need for a cheaper solution to produce oil and gas from smaller oil- and gas field at deep water were central and it was assumed that floating production structures would be a viable solution (Larsen et al., 2011).

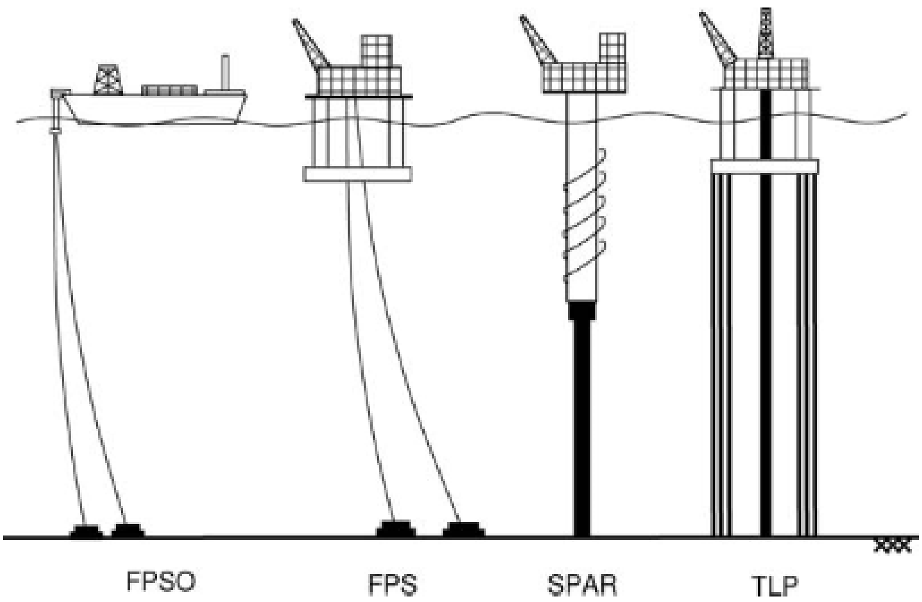


Figure 2.1: Different floating structures in use. Floating Production, Storage and Offshore platform (FPSO), Floating Production System (FPS), SPAR, Tension Leg Platform (TLP). (Source: Randolph and Gourvenec, 2011).

The different floating structures depicted in figure 2.1 have different advantages. Brief description is listed below.

Floating Production Storage and Offshore platform (FPSO)

Floating production storage and offshore platform is shaped like a ship and is widely used floating facility (Randolph and Gourvenec, 2011). FPSO has been showed viable at smaller oil fields with no infrastructure. The oil and gas are stored on ship and is transferred directly to petroleum tankers. FPSO is vulnerable to wind and waves from the side, to handle this problem the foundation is connected to a turret so that is possible for the FPSO to move keeping the bow against wind and waves (Larsen et al., 2011).

Floating Production System (FPS)

Floating production system or semi-submersible platform was first used on the British shelf in 1975, and was originally used as a drilling platform. The hull consist of four hollow legs connected to a ring pontoon. This type of platform is usually used as a drilling platform, production platform or a combination of both, and it's believed viable at depths reaching 3000m. One disadvantage with FPSs is that it is vulnerable to accidents. Several accidents have happened over the years, to mention some few: Alexander L. Kjelland (North sea, deaths count 123 (Smith-Solbakken)),Petrobras 36 (Brazilian coast) and Deepwater horizon (Gulf of Mexico, deaths count 11 (Vinnem)) (Larsen et al., 2011).

SPAR

Spar platforms have large draft (150 to 200 meters from waterline to bottom of the platform), a direct effect of this is there is small vertical movements, which also makes it a competitor to the TLP (Larsen et al., 2011). The spar platform is shaped like a cylinder and is usually ballasted with iron ore to keep it in a vertical position. It is believed that spar platform could be used at depth reaching 3000m. The spar is hold in place with either chain, wire, fibre ropes or a combination of all (Randolph and Gourvenec, 2011).

Tension Leg Platform (TLP)

The first tension leg platform was installed on British shelf in 1984, in Norway it's only two TLPs installed (Snorre and Heidrun). Tension is kept vertical by steel tubes and keeps the platform from moving in vertical direction, the only vertical movement is the elastic strain in the steel. One advantage when utilising TLP in contrary to platforms with long legs is that this method is possible in depths above <300m. It is believed to be possible to use TLP at water depth of 2000m (Larsen et al., 2011).

2.1 Mooring Systems for Floating Structures

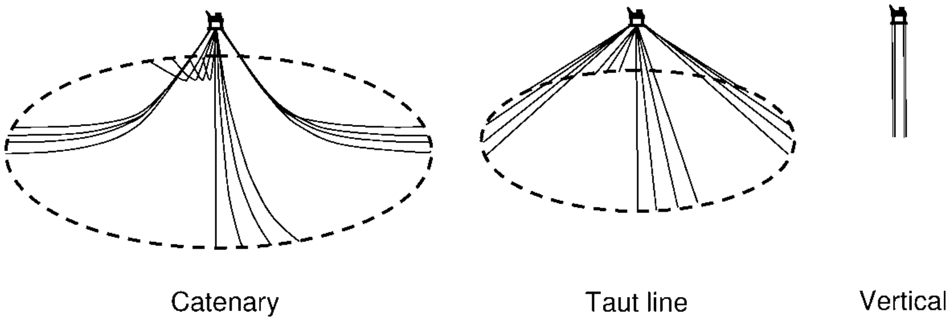


Figure 2.2: Three different mooring systems for buoyant platform. The first being a catenary mooring system where chain is used, the second being taut line where polyester ropes is used and the last being a vertical mooring system (Randolph and Gourvenec, 2011, p.310).

Figure 2.2 visualise how floating platforms is anchored to the seafloor. The catenary name derives from the geometrical shape the chain makes when is it subjected to its own self weight, only being hold in place at its ends. The advantage of the catenary mooring system is that the chain is laying on the seafloor in advance of the anchor. Benefit of this method is that the angle of the chain is close to zero, so the load direction on the anchor is purely horizontal (Randolph and Gourvenec, 2011, p.309).

The catenary system has its limits, the weight of the line becomes to heavy at great depths, so the taut line mooring where developed to handle this problem. Instead of using wire of chain the taut system uses synthetic ropes, which are of lighter fabric than the steel used in the catenary system. The other difference between the two systems is the angle in the connection to the anchor. The taut line have an angle of 30-45° in comparison to the catenary which mention above is close to horizontal. This angle makes the taut line linear and lowers the length needed for the line, which makes the total weight of the line lighter than the catenary line, illustrated in figure 2.3. Other benefits of using the taut line rather than the catenary line is that the footprint caused by the length of the line is lower. There is also less movement on the platform due to constant tension in the taut line (Randolph and Gourvenec, 2011).

The third system is the vertical mooring. Steel cables goes vertical down to the anchoring system and applies vertical loads on the anchors (TLP platforms).

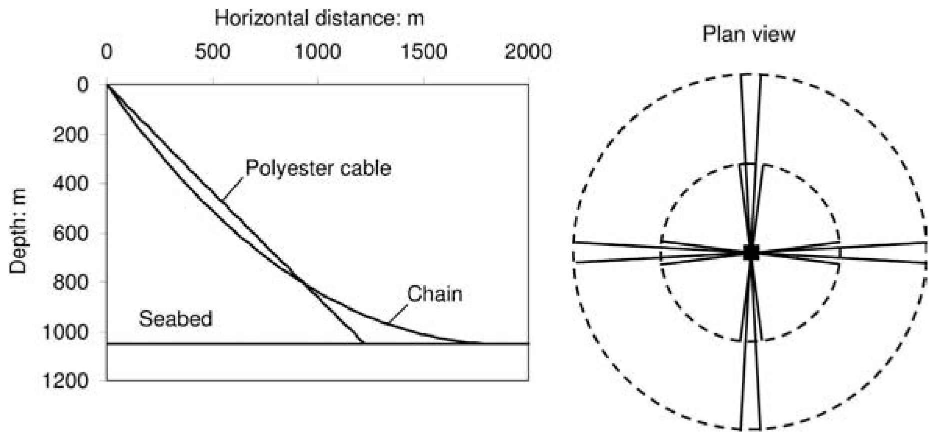


Figure 2.3: The shape difference between catenary line and taut line (Source: Randolph and Gourvenec, 2011, p.311).

2.2 Suction Anchors

Suction anchors technology have been over the last decades widely used for offshore structure, and have been applied by the biggest offshore oil producing countries (Tjelta, 2001). Table 2.1 and 2.2 is a list of installed suction anchors from 1987 to 2004, and it shows the variety in size, depths and location.
Suction anchor is of cylindrical shape, and are enclosed at one end, either with a flat top (see figure 2.4) or dome shaped (like the one used at Gjøa). The size of the suction anchor varies, usually the height to diameter ration is less than 6 (Andersen et al., 2005).
Due to the "simple" design, suction anchors is economical and easy to install at great water depths (Huang et al., 2003). The installation of suction anchors is further described in section 2.4.

When loaded vertically the self weight, friction alongside the skirt and reversed bearing capacity of the anchor is the main contributor to the bearing capacity of the anchor. Horizontal loaded suction anchors are loaded in order z/L equal 0,65-0,7 where z is the action of load. This is done to maximise bearing capacity of the anchor by preventing rotation (Randolph and Gourvenec, 2011). DNV (2005) gives recommendations on calculating bearing capacity of suction anchors in clay. How to estimate bearing capacity of suction anchors will not be presented in this thesis.

Figure 2.4: Illustration of suction anchor with chain attached to a pad-eye (Source: Technology).

Year	Location	Field	Floater	Depth (m)	D x L (m x m)	No.
1987	North Sea	Gorm	FSO	40	3,5 x 8,5	12
1991	North Sea	Snorre	TLP	335	30 x 13	4
1993	North Sea	Tordis	Well head	200	9 x 6,5	1
1994	North Sea	Heidrun	TLP	375	44 x 4,5	4
1995	West Africa	Nkossa	Barge	170	4,5-5 x 12	12
1995	North Sea	Yme	Loading buoy	100	5 x 7	8
1995	North Sea	Hrding	Loading buoy	110	5 x 8-10	8
1996	North Sea	Norne	FPSO	375	5 x 10	12
1997	North Sea	Njord	Semi FPU/FSU	330	5 x 8-10	12
					5 x 7-10	8
1997	North Sea	Curlew	FPSO	90	5-7 x 9-12	9
1997	Offshore Brazil	Marlim P19-P26	Semi FPU	770-1000	4,7 x 13	32
1997	West of Shetland	Schiehallion	FPSO	400	6,5 x 12	14
1997	North Sea	Visund	Semi FPU	345	5 x 11	16
1997	South China Sea	Lufeng	FPSO	30	5 x 10	8
1997	Adriatic Sea	Aquila	FPSO	850	4,5-5 x 16	8
1998	Timor Sea	Laminaria	FPSO	400	5,5 x 13	12
1998	Offshore Brazil	Marlim P33	FPSO	740-840	4,7 x 20	6
1998	Offshore Brazil	Marlim P18	Riser support	900	18 x 16,2	2
1998	North Sea	Siri	Loading buoy	60	4,25 x 4,6	1
-	North Sea	Snorre B		335		
1998	North Sea	Bandd	4 cells	90	4,3 x 11	1
1998	North Sea	Aasgard	Mid water arch	303	9,5 x 7,5-7,7	3
1998	North Sea	Aasgard A	FPSO	350	5 x 11	12
1999	North Sea	Aasgard A	Tieback	350		
1999	North Sea	Aasgard B & C	Semi-FPU & FSO	350	5 x 10	16
					5 x 12	9
1999	Offshore Brazil	Marlim P35	FPSO	810-910	4,8 x 17	6
1999	North Sea	Troll C	Semi-FPU	350	5 x 15	12
1999	West Africa	Kuito	FPSO	400	3,5 x 14	12
					3,5 x 11	
1999	Gulf of Mexico	Diana	SPAR	1500	6,5 x 30	12
1999	Gulf of Mexico	Preset anchors		-	2,8 x 16	1
					2,8 x 13	1
-	Gulf of Mexico	Green canyon 854	Semi-Rig	1650	3,7 x 18,5	8
1999	Gulf of Mexico	MODU	-	-		
2000	North Sea	Hanze		40	6,5 x 66,1	1
2001	West Africa	Girassol	Riser tower	1350	8 x 20	3

Table 2.1: List of installed suction anchors from 1981 to 2004 after Andersen et al. (2005).

2001	West Africa	Girassol	FPSO	1350	4, x 17	16
2001	West Africa	Girassol	Loading buoy	1350	5 x 18	6
					5 x 16,1	3
2002	Gulf of Mexico	NaKika	FDS	1920	4,3 x 23,8	16
2002	Gulf of Mexico	Horn Mountain	SPAR	1650	5,5 x 27,4	6
					5,5 x 29	3
2002	South China Sea	Wenchang	FPSO	120	5,5 x 12,1	9
					5,5 x 12,8	
2003	Offshore Brazil	Barracuda	FPSO	825	5 x 16,5	18
2003	Offshore Brazil	Caratinga	FPSO	1030	5 x 16,5	18
2003	Offshore Nigeria	Bonga	FPSO	980	5 x 17,5	12
					5 x 16	
2003	Offshore Nigeria	Bonga	SPM	943	3,5 x 18	9
					3,5 x 16	
2003	Gulf of Mexico	Red Hawk	-	1600	5,5 x 22,9	8
2003	Gulf of Mexico	Devils Tower	-	1700	5,8 x 34,8	9
2003	North Sea	Ardmore	4 cells	78	6,5 x 3	1
					6,5 x 5,5	1
2003	South China Sea	Panyu		105	5 x 11,7	9
					6 x 12,7	
2003	Gulf of Mexico	Holstein	-	1280	5,5 x 36,3/38,4	16
2004	Gulf of Mexico	Thunder Horse	Semi FPU	1830	5,5 x 27,5	16
2004	Gulf of Mexico	Thunder Horse	Manifold	1830	6,4 x 23,8	4
2004	Gulf of Mexico	Thunder Horse	PLET	1830	5,5 x 26	4
2004	Gulf of Mexico	Thunder Horse	Water injection	1830	3,4 x 19	3
					3,4 x 20	2
2004	Gulf of Mexico	Mad Dog	-	1600	5,5 x -	11
					7,6 x -	

Table 2.2: continuation of table 2.1

2.3 Subsea Installations

For several practical reasons, different structures is installed on the seafloor where suction anchor technology is utilised. A common subsea structure is the integrated template structure (ITS) showed in figure 2.5. The depicted template has four suction anchors, one in each corner.

Figure 2.5: ITS being lowered of a barge. (Picture: Hydro)

Subsea systems are used in shallow and deep waters, and in combination with fixed platforms. Usually the ITS is located at some distance away from the platform and is connected with pipelines to the platform or directly to process plants onshore (Larsen et al., 2011). An example of combination between platform and ITS is Goliat, operated by ENI Norway. Goliat FPSO is located in the Barents Sea with eight ITS which delivers oil and gas to the FPSO with pipelines. The weight of the ITS is approximately 300 metric tonnes with 8-9 meter skirts (Eni).

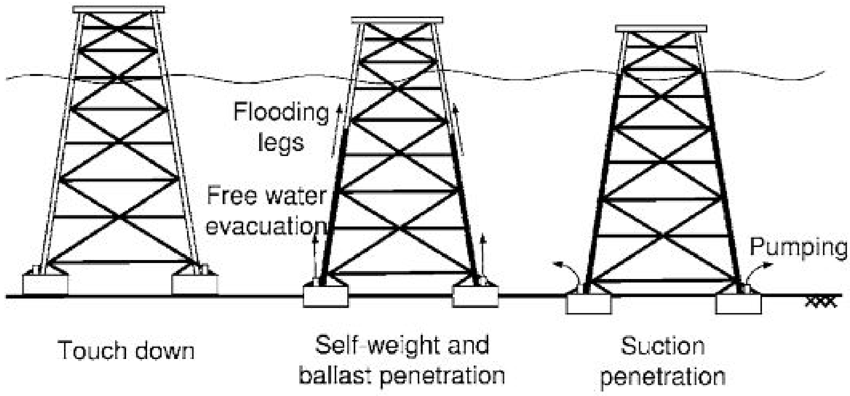


Figure 2.6: The stages of installing suction foundation (Randolph and Gourvenec, 2011).

2.4 Installation of Suction Foundation

Figure 2.6 illustrates how suction foundation is installed. The installation itself is divided into two stages. Stage one being self-weight penetration, where the weight of the structure will force the foundation down into the soil. The self-weight penetration stops when the weight of the structure is in equilibrium with soil strength.

The second stage is the suction penetration. Suction is applied by a remotely operated underwater vehicle (ROV) at the top off the anchor. Water is pumped out from the inside of the anchor, a pressure difference occurs. The water pressure surrounding the anchor will be higher than the water pressure inside the anchor, forcing the anchor deeper into the soil. It's important that the self-weight penetration is deep enough, to prevent the development of water channels going from inside the anchor to the top of the seabed outside the anchor. If water channels occurs it will be impossible to produce under pressure inside the anchor, and the anchor will not penetrate the soil (Randolph and Gourvenec, 2011, p. 249).

2.5 Advantages and Disadvantages

There are several advantages and disadvantages according to Ehlers et al. (2004) with the usage of suction anchors. Brief summary is given in table 2.3.

Advantages	Disadvantages
-Simple to install accurately with respect to location, orientation, and penetration	- Heavy - derrick barge may be required
-Leverage design experience with driven piles	-Large - more trips to shore to deploy full anchor spread
-Well-developed design and installation procedures	-Requires ROV for installation
-Anchor with the most experience in deepwater for mooring MODUs and permanent facilities	Requires soil data from advanced laboratory testing for design
	-Concern with holding capacity in layered soils
	-Lack of formal design guidelines
	-Limited data on setup time for uplift

Table 2.3: A brief summary of the advantages and disadvantages of using suction anchors according to Ehlers et al. (2004).

Chapter 3

Suction Anchor Penetration Analysis

There are primarily two approaches to estimating penetration resistance. Estimating resistance directly solely based CPT measurement with tentative factors or use undrained shear strength obtained by laboratory test and CPT. Both the methods uses CPT measurement either directly or indirectly.

3.1 Equilibrium of Forces

To keep the foundation in equilibrium the driving forces must be equal to the resisting forces. For the foundation to penetrate the soil the driving forces must be bigger than the resisting forces. The driving forces when installing an suction foundation is the self weight (Q'_{weight}, the submerged weight of the foundation) of the foundation and the suction forces (Q_{suction}). The resisting forces is the friction alongside the skirt (Q_{friction}), the tip resistance (Q_{tip}), the tension of the crane (Q_{tension}) and additional forces like pad-eye, stiffeners etc (Q_{add}). The forces is depicted in figure 3.1.

$$Q'_{\text{weight}} + Q_{\text{suction}} = Q_{\text{friction}} + Q_{\text{tip}} + Q_{\text{tension}} + Q_{\text{add}} \tag{3.1}$$

The self weight of the foundation and the tension force from the crane will not change during installation (The crane tension is just for keeping the foundation in a vertical position), and will then be permanent load in equilibrium equation. Frictional forces and tip forces will increase with depth, so for penetration to occur the suction force also has to increase with depth.

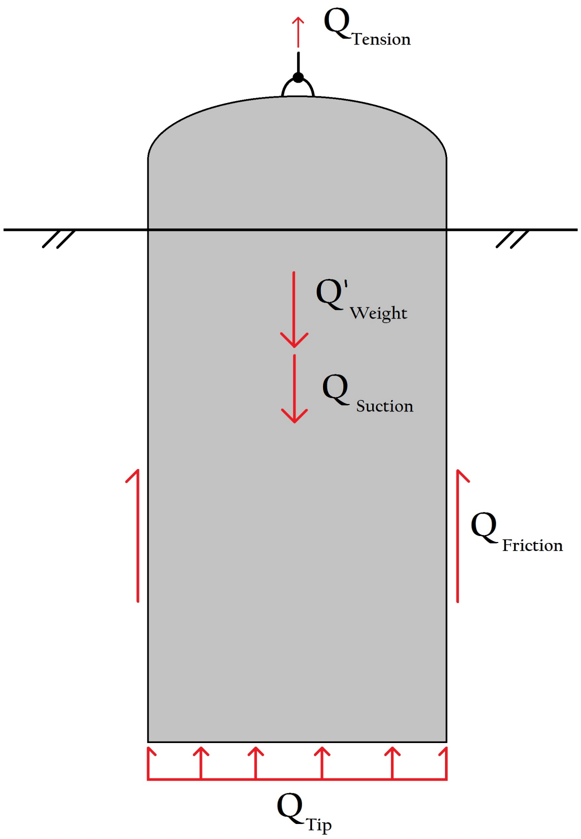


Figure 3.1: Equilibrium of forces during installation of suction anchors.

3.2 Recommended Estimation Approaches

There is no defined standard when it comes to estimating penetration resistance of suction anchors, but several guidelines have been produced by the industry. Det Norske Veritas released a classification note in 1992 (see section 3.2.1) regarding offshore structure foundations and a recommended practice[1] in 2005 regarding suction anchors (see section 3.2.2). American Petroleum Institute (API) have also released a similar guideline as the RP by DNV, which is discussed in section 3.2.3.

3.2.1 Det Norske Veritas Classification Notes - No. 30.4

DNVs classification note from 1992 (DNV, 1992) suggest direct use of CPT data in combination with tentative factors to estimate penetration resistance of steel skirts, dowels and steel ribs. The

[1] After the merge between DNV and Germanischer Lloyd (GL) they have revised RP-E303 (April 2017) to match their layout, with no changes to the content.

tentative values is given in table 3.1. The coefficient values are given for North Sea conditions, which should be suitable for Gjøa.

The factor for tip resistance k_p is the bearing capacity difference between deep small strip foundation and deep circular foundation. In section 6.9 the impact of k_p is studied and it shows low impact level at greater depths. The skin friction factor k_f is related to the value N_{kt} and sensitivity $1/S_t$. The N_{kt} varies with plasticity index I_p, over consolidation ration OCR and undrained shear strength according to Lunne et al. (1997). Based on this the use of the values presented in table 3.1 should be considered with great care.

$$R = k_p(z) A_p \overline{q}_c(d) + A_s \int_0^d k_f(z) \overline{q}_c(z) dz \tag{3.2}$$

where

d = depth of tip penetrating member, m.
$k_p(z)$ = empirical coefficient relating q_c to end resistance.
$k_f(z)$ = empirical coefficient relating q_c to skin friction.
$\overline{q}_c(z)$ = average cone resistance at depth z, MPa.
A_p = tip area of penetrating member, m^2.
A_s = side area of penetrating member, per unit penetration depth, m^2/m.

Type of soil	Most probable (R_{prob})		Highest expected (R_{max})	
	k_p	k_f	k_p	k_f
Clay	0,4	0,03	0,6	0,05
Sand	0,3	0,001	0,6	0,003

Table 3.1: Tentative values for coefficient k_p and k_F. The values are suitable for North Sea conditions according to DNV (1992).

The document just covers foundation methods like piles gravity foundations and jack-up platform, whereas suction anchors and shallow bucket foundations isn't mentioned. The lack of information and guidelines regarding bucket foundation and suction anchors might be due to time and age of when this document was made.

3.2.2 Det Norske Veritas Recommended Practice (RP-E303)

In 2005 Det Norske Veritas (DNV) release a recommended practice (DNV, 2005) for design and installation of suction anchors. The document was intended to give specific requirements for the design and not as an guideline to how it should be designed. The report goes through partial factors and how bearing capacity for suction anchors could be calculated.

The calculation of penetration resistance is put in appendix A of their document with a recommendation on how it could be estimated. The recommended method for calculating penetration resistance is based on Knut H. Andersen and Hans Petter Jostads work (article Andersen and Jostad (1999) and Andersen and Jostad (2004))

The report suggest that the tip resistance could be reduced if the anchor is installed with under pressure or set to zero in clay conditions. Section 6.9 illustrates the magnitude of tip resistance, whether or not to exclude is debatable.

Basic equation estimating penetration resistance

The basic equation is a two-part equation, the tip resistance and side friction. Other influencing geometry (e.g. stiffeners, pad-eye, chain etc.) needs to be added. Excluding auxiliary geometry and -equipment will either give lower or larger penetration resistance.

$$\mathrm{Q_{tot}} = \mathrm{Q_{side}} + \mathrm{Q_{tip}} = \mathrm{A_{wall}} \cdot \alpha \cdot s_{u,D}^{\mathrm{av}} + \left(N_c \cdot s_{\mathrm{u,tip}}^{\mathrm{av}} + \gamma' \cdot z\right) \cdot \mathrm{A_{tip}} \tag{3.3}$$

where

$\mathrm{A_{wall}}$	=	skirt wall area (sum of inside and outside)
$\mathrm{A_{tip}}$	=	skirt tip area
α	=	shear strength factor (normally assumed equal to the inverse of the sensitivity; if the skirt wall is painted or treated in other ways, this must be taken into account in the α-factor)
$\mathrm{s_{u,D}^{av}}$	=	average DSS shear strength over penetration depth
$\mathrm{s_{u,tip}^{av}}$	=	average undrained shear strength at skirt tip level (average of triaxial compression, triaxial extension and DSS shear strengths).
γ'	=	effective unit weight of soil
N_c	=	bearing capacity factor, plane strain conditions (7,5)
z	=	skirt penetration depth

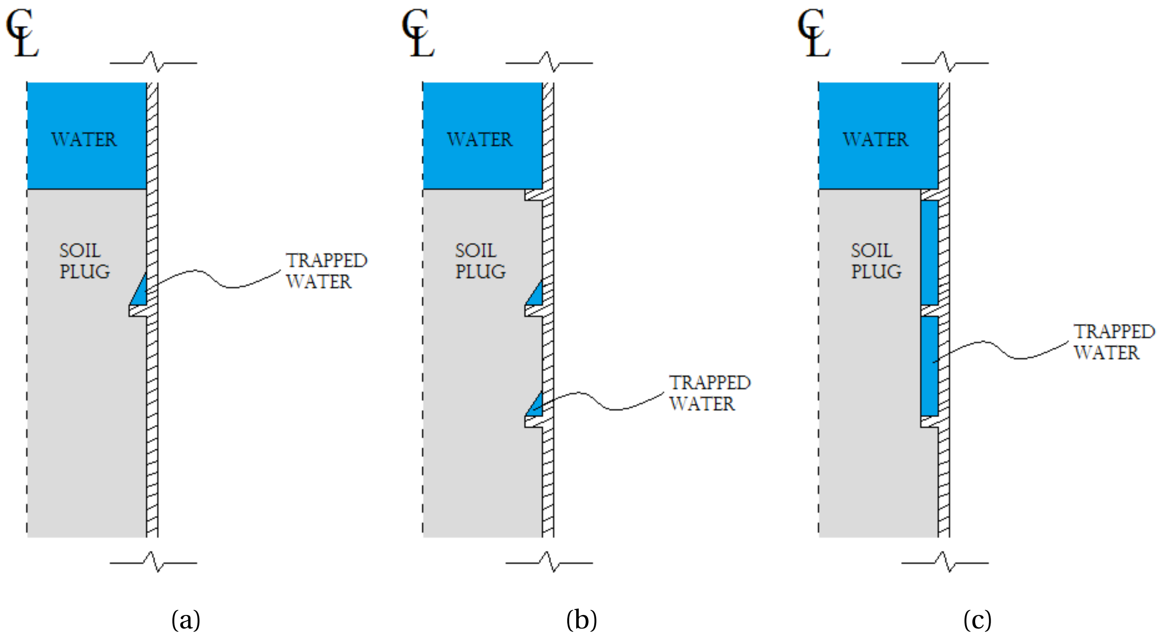


Figure 3.2: Illustration of how water gets trapped behind inside stiffeners for suction anchors. Figure 3.2a shows how water gets trapped behind one stiffener and the clay gets back to the skirt. Figure 3.2b Shows the same as the first figure but for several stiffeners. Figure 3.2c represent the case where the soil doesn't goes back before it hits a new stiffener and the water gets trapped in between the stiffeners.

Inside stiffeners

In cases where there are no rings stiffeners or below the first stiffener they suggest using the same calculation procedures as for the outside of the anchor. In cases where there are ring stiffeners they suggest that the clay should be checked if it goes back against the wall after passing the first stiffener (based on triaxial extension test).
Where there is just one ring stiffener the shear strength is set to zero at some distance behind the ring, assuming water will be trapped in this region.

For several stiffeners, the same assumption as mention above is valid, but where the soil plug is too strong the soil plug doesn't deform back in between the stiffeners and it's assumed that water will be trapped between the stiffeners. Figure 3.2 illustrates how the water is trapped for the different situations.

Outside stiffeners

For the outside stiffeners, the report suggest to use a lower shear strength due to remoulding to calculate the bearing resistance. It's uncertainties to the shear strength of the soil after passing the stiffener and there could become a gap between the wall and the soil. Since there isn't any general accepted method for calculating this sort of problem the report suggests not to use outside stiffeners. Where the skirt wall is of different thickness, it's recommended to keep the outside diameter the same and let the thickness affect the inside.

Necessary under pressure

The necessary under pressure Δu_n needed to reach wanted depth is given by equation 3.4.

$$\Delta u_n = \frac{\left(Q_{\text{tot}} - W'\right)}{A_{\text{in}}} \tag{3.4}$$

where

W' = submerged weight during installation
A_{in} = plan view inside area where under pressure is applied

3.2.3 American Petroleum Institute

The American petroleum institute is an industry trade group, where oil companies cooperate to influence public policy. Besides the lobbying they also develop standards and recommended practises.

The recommended practice report 2SK[2] is a comprehensive report about stationkeeping of floating structures. The penetration analysis of suction anchors is presented in the appendix E of their report, (API, 2005, p.93-118). Their recommendation for calculating the penetration resistance is identical to the one presented by Det Norske Veritas in their RP-report. The reason for the similarity is due to a industry sponsored research between NGI, *Offshore Technology Research Center* (OTRC) and *Centre for Offshore Foundation System* (COFS), to provide API and Deepstar Joint Industry Project VI with data and background information about design and installation of suction anchors at deepwater (Andersen et al., 2005).

3.3 Roughness Alongside Skirt Wall

The roughness of the skirt walls greatly impacts the shear strength affecting the anchor. Usually the steel skirt is rusty, providing enough roughness for the soil to fail inside the soil and not

[2] The one used in this report is the 2005 version, and not the newest release from 2015. Reason being NTNU doesn't subscribe to API standards.

alongside the wall (Andersen et al., 2005). According to experiences told by the supervisors almost every case where anchors for some reasons when further penetration has been aborted a thick layer of clay has been sticking on the skirts. Figure 3.3 shows an suction anchor with a thick layer of clay still holding onto the sides of the skirt after being lifted up onto the installation vessel.

Where the anchors isn't rusty or for some reason have been painted, it is then recommended to reduced $s_{\mathrm{u,rem}}$ with a factor α lower than $1/S_t$. The reduction factor could be obtained by ring shear test (Andersen et al., 2005). The anchors installed at the Girassol FPSO were painted on 20% of the outside surface, this had a great impact on measured suction and reduced the outside friction. From ring shear test, they found out that the adhesion factor α was 1/3 of what were used at the unpainted areas (Colliat and Dendani, 2004).

Figure 3.3: Thick layer of clay sticking to the sides of an suction anchor after being pulled out at the Girasoll oil field. The picture also illustrates how soil fail in the interface between the anchor and clay for where the anchor was painted. Source: Gisle Håland.

3.4 Behaviour of Soil Inside the Anchor

During penetration of suction anchors it seems reasonable to believe that the soil acting on the outside will have an reduction in strength equal to the inverse soil sensitivity or be equal to the remoulded shear strength $s_{u,rem}$. This assumption could be drawn from the fact that the soil experience large strain and that the outer diameter (shouldn't) change over the penetrating depth. However, the behaviour of the soil inside the suction anchor is dependent on inside stiffeners, either with the use of ring stiffeners or increase in skirt thickness. In Andersen and Jostad (2004) they conclude that the effect of anchor penetration to be similar to what a soil tube sample is exposed to, large strain in the outer periphery. And they referring to this large strain as remoulded clay. Thus the use of remoulded shear strength in the basic equation 3.3. This remoulding of the soil happens at the tip of the anchor due to bearing capacity failure (see figure 3.4). The remoulded zone is assumed to be of 1 to 1,5 times the thickness of the wall thickness.

3.5 Soil Plug Inside Anchor

Wall thickness, internal stiffeners and suction will contribute to a soil heave inside the anchor. Total plug heave could be estimated as the total soil displacement going inside the anchor.

During self weight penetration, it is assumed that half wall thickness is displaced inside the anchor, and the other half is displaced outside (Randolph and Gourvenec, 2011). During suction is assumed that 50% to 100% of the soil displaced inside the anchor (Andersen et al., 2005). Due to internal stiffeners, the height of the soil heave could vary based on soil properties, figure 3.2 illustrates how soil could behave inside the anchor with respect to stiffeners.

3.5.1 Plug failure due to suction

To estimate safety against plug failure Andersen et al. (2005) suggest that the internal friction is estimated equal to the frictional force during penetration, and that the tip is estimated as inverse bearing capacity. Equation 3.5 is the allowable under pressure with respect to plug failure after Andersen and Jostad (1999).

$$\Delta u_a = N_c \cdot s_{u,tip}^{av} + A_{inside} \cdot \alpha \cdot \overline{s_u^{DSS}} / A_{in} \tag{3.5}$$

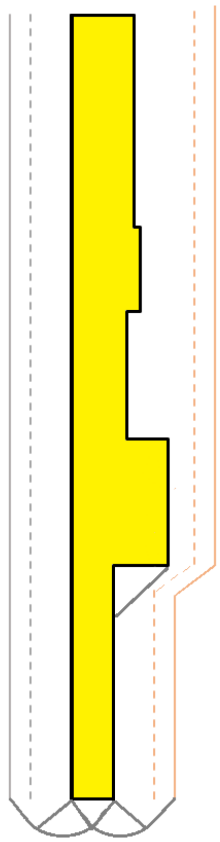


Figure 3.4: Illustrating bearing capacity failure during large strains. The dotted lines indicates 1,0 times the anchor thickness and the solid line being 1,5 times the tip of the anchor. The anchor thickness and geometry is equal to the 18,5 m long suction anchor used at Gjøa.

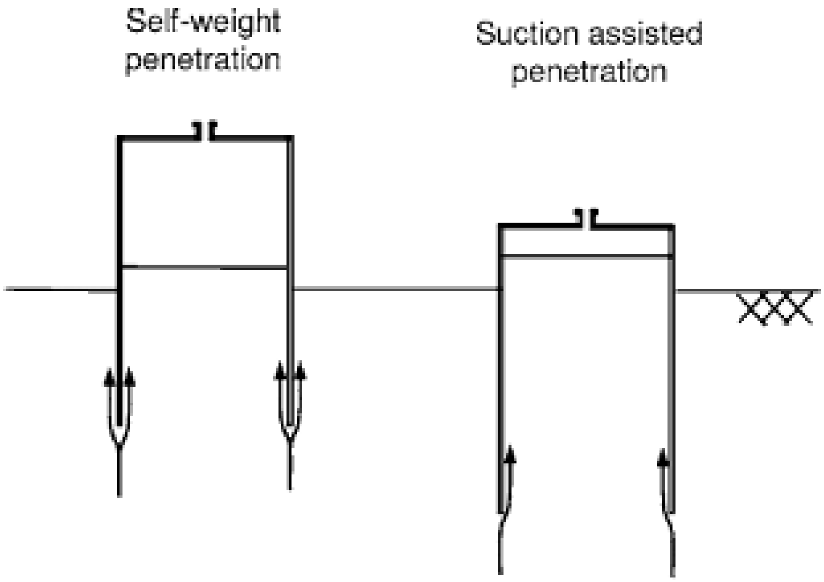


Figure 3.5: Soil flow during self weight penetration and suction. (Source: Randolph and Gourvenec, 2011, p.343)

It's also necessary to address the structural strength of the anchor. With high under pressure the cylindrical shape could implode or buckle inwards. Also in shallower water the suction shouldn't exceed cavitation pressure (DNV, 2005).

3.6 Skirt Friction Based on Sleeve Friction Measurement f_s

Andersen et al. (2005) discuss the importance of remoulded shear strength and its position in estimating penetration resistance. They suggest looking closer on the usage of sleeve friction from CPT directly, to estimate remoulded shear strength. Equation 3.6 is a proposed method to estimate penetration resistance. It's based on the method given by DNV (1992) but the contribution provided by skirt friction is replaced with sleeve friction.

$$Q_{\text{tot}} = k_p(z) A_p q_c(d) + A_s \int_0^d \epsilon \cdot f_s(z)\, \mathrm{dz} \tag{3.6}$$

The resistance contributed by the tip is kept the same as the one proposed in equation 3.2, due to several reasons. The tip resistance alone only contributes marginally to the total resistance (see section 6.9), and the k_p factor is based on solid theory. A factor ϵ adjusts the sleeve friction. The adjustment factor is back calculated in section 6.5 based on measured suction.

Chapter 4

Cone Penetration Test

Cone penetration test (CPT) is a commonly used geotechnical device used to classify soil and interpreter soil characteristics by the industry. It has been around for almost 70 years in Norway. The CPT measures cone resistance (q_c), sleeve friction (f_s) and in a lot of cases pore pressure (u). With the measuring of pore pressure the method is referred to as cone penetration test undrained (CPTU) (Sandven et al., 2014). The procedures when conducting a CPT/CTPU is by penetrating the soil at a rate of 20 mm/s, sending real time data from the piezocone.

4.1 Factor Affecting Measurement

Due to how the probe is made some ambient pore pressure is affecting the cone resistance and sleeve friction. For the cone resistance pore pressure (u_2) is resting on top of the cone. For the sleeve friction pore pressure (u_2) is pushing upwards and (u_3) is pushing downwards on the sleeve. These two effects could be corrected for with the use of equation 4.1 and 4.2. Figure 4.1 illustrates these effects (Lunne et al., 1997).

$$q_t = q_c + u_2(1-a) \tag{4.1}$$

where

a = Cone area ratio

$$f_t = f_s - \frac{u_2 \cdot A_{sb} - u_3 \cdot A_{st}}{A_s} \tag{4.2}$$

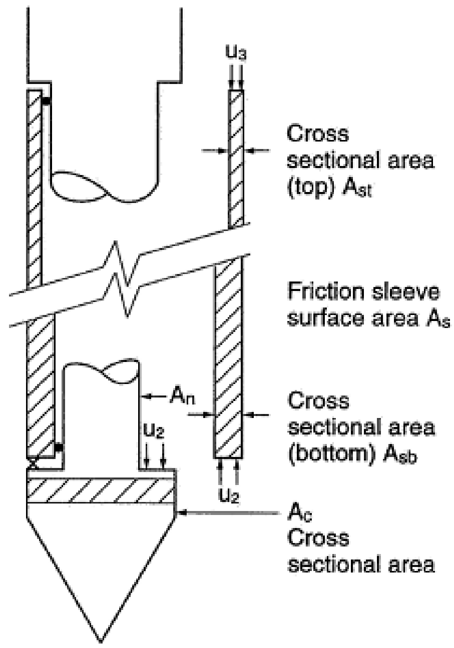


Figure 4.1: Cross section illustrating how pore pressure is affecting cone resistance and sleeve friction measurements. (Soucre: Lunne et al., 1997, p. 26)

For some cases where the end areas of the sleeve friction are different from each other or are quite thick, the pore pressure could affect the sleeve friction greatly by pushing on the sleeve ends. Usually u_3 isn't measured, making this correction is unavailable. Lunne et al. (1997) gives an example where both u_2 and u_3 where measured. Figure 4.2 shows the ratio between the measured sleeve friction and the corrected sleeve friction plotted versus depth, the difference being ±20%. This illustrates the great effect pore pressure have on sleeve friction measurement.

Inclination during penetration

Deflection during penetration should be avoided, not just to protect the probe and rod but for the quality of the reading. If the reading isn't done as vertical as possible the measurement will be falsely with depth, this is illustrated by figure 4.3. Lunne et al. (1997) recommends inclinometer sensors being installed when penetrating deeper than 15 meters to avoid uncertainties related to inclination.

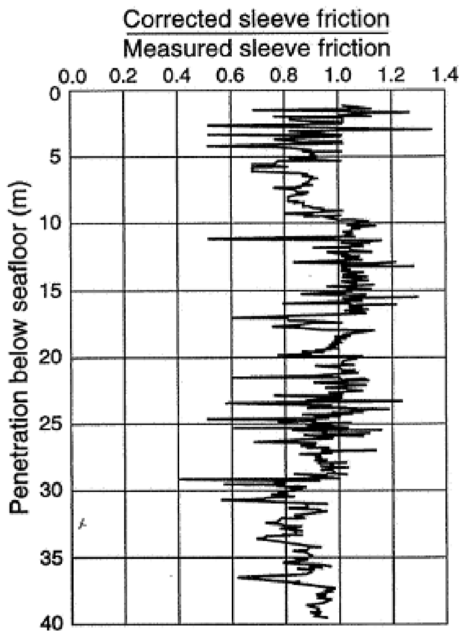


Figure 4.2: The graph illustrating the ratio between the measured and corrected sleeve friction versus depth at Gullfaks C oil field. (Soucre: Lunne et al., 1997, p. 28)

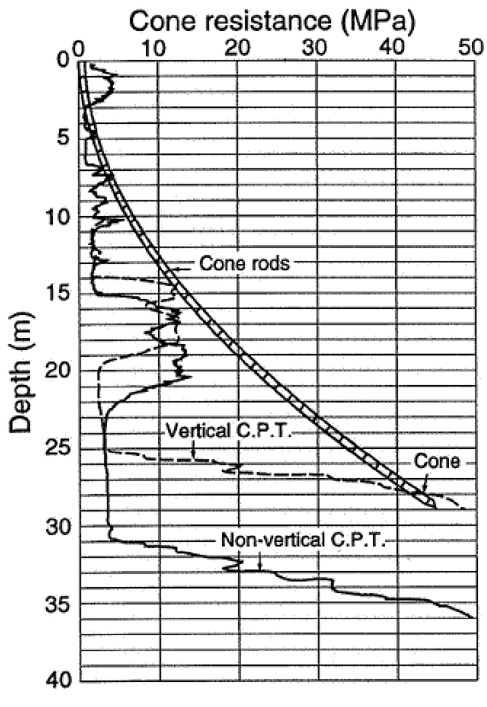


Figure 4.3: Measurement could easily be falsely interpreted due to the inclination of the penetration. (Soucre: Lunne et al., 1997, p. 17)

4.2 Interpretation of CPT Results

Interpretation of soil properties from CPT could be difficult and give differences in accordance to laboratory tests. Usually the CPT is used in conjunction with laboratory test to verify the values and calibrate the CPT values.
The methods used to interpret CPTs is empirical, semi-empirical and theoretical (Sandven et al., 2014). According to Lunne et al. (1997) methods to estimated undrained shear strength is highly to moderately reliable, and sensitivity is moderately reliable.

Lunne et al. (1997) points out that remoulded shear strength measurement done by remoulded unconsolidated undrained (UU) triaxial test correlates well versus sleeve friction. The correlation is seen in figure 4.4.

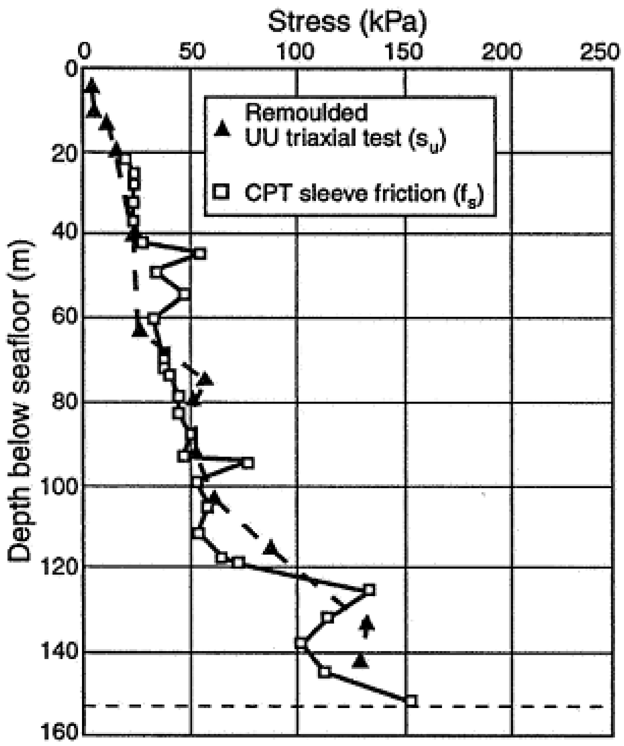


Figure 4.4: Remoulded UU triaxial test correlates well with measured sleeve friction. (Source: Lunne et al., 1997, p.68)

Chapter 5

Gjøa Field Data

This chapter presents the Gjøa field data from the investigation conducted by NGI and Fugro. Values used in the estimations is presented and commented below.

5.1 Gjøa Semi Sub

The Gjøa semi submersible platform is located within block 35/9 and 36/7 in the norther part of the North Sea (Helgesen, 2010). Gjøa platform is hold in place by 16 suction anchors in four clusters, which will be referred to as cluster south-west (SW), north-west (NW), north-east (NE) and south-east (SE). Figure 5.1 shows the anchor clusters location relative to the platform.
The oil reserves is estimated to contain 82 million barrels of oil and condensate, and 40 billion cubic meters of gas (Helgesen, 2010).

5.2 Soil Condition

The soil conditions are taken from NGI soil investigation report conducted in 2006 for Statoil (NGI, 2007). Four CPT/CPTu were conducted at each anchor cluster, one for each anchor. And several soil samples were taken in conjunction with the CPT/CPTu.

The soil profile at Gjøa field is divided into three units. Unit I, Unit IIA and Unit IIB. Unit I being the first 6,5 to 8 meters (see table 5.1 for more detail regarding depth of unit I for each anchor cluster) and unit IIA reaching beyond 21 meters. Due to the maximum height of anchor cluster

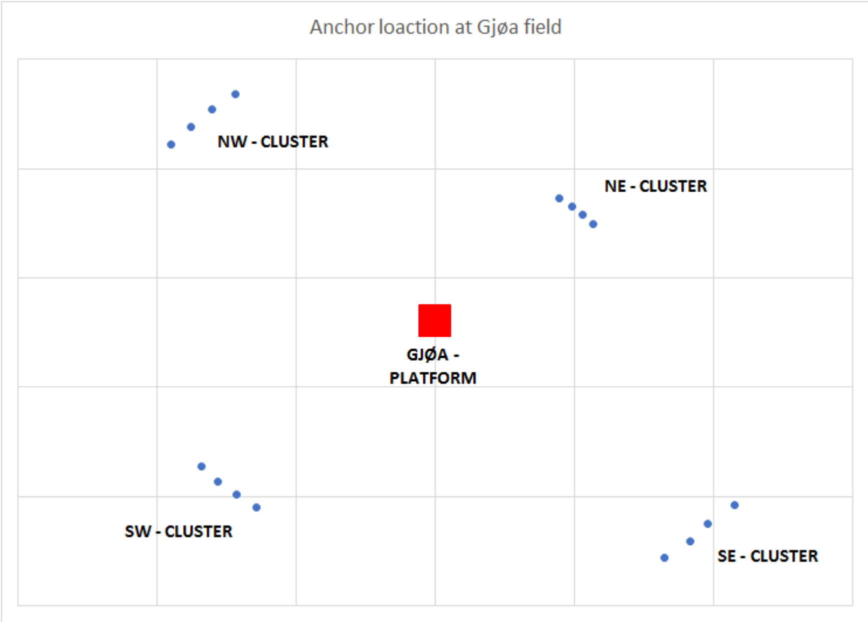


Figure 5.1: Location of anchor cluster holding Gjøa platform in place.

SE unit IIB will not be discussed since it would not be affecting the anchors during penetration.

Unit I is a highly plastic clay with high water content. The clay is ranges from very soft to medium stiffness. Unit IIA is described as a medium to stiff clay, and is medium plastic.

Cluster	Unit I Top - Bottom (m)	Unit IIA Top - Bottom (m)
SW	0 - 6,5	6,5 - 21
NW	0 - 6,4	6,4 - 21
NE	0 - 6,6	6,6 - 21
SE	0 - 8,0	8,0 - 21

Table 5.1: Depth for layer unit I and IIA. Where 21 meters is end-of-borehole (EOB) for all clusters (NGI, 2007).

Unit	w (%)	γ (kN/m^3)	OCR (-)	k_0 (-)	I_p (%)
I	62,0	16,0	2,5 - 1,7	0,75	34,4
IIA	26,0	19,7	1,7 - 1,5	0,70	20,6

Table 5.2: Basic soil parameters recommended by NGI (2007).

5.2.1 CPT data

As a part of the investigation of the soil at Gjøa field, one CPT per suction anchor has been conducted. The CPT data is presented as min, max and average values in appendix A. One of the largest uncertainties with the CPT measurement at Gjøa is the distances from where the CPT were taken and where the suction anchor is installed (The distances is presented in table 5.4). North-East cluster is on average the closest cluster considering where CPT and soil samples were taken. Table 5.3 shows average percent difference between the lowest measured q_c and the highest measured.

Anchor cluster	Difference lowest-highest q_c
SW	60%
NW	45%
NE	30%
SE	35%

Table 5.3: Differences between the lowest measured q_c and the highest measured q_c for all anchor clusters.

Average, minimum and maximum measured values

The CPT data used is a digitisation from CPT plot from NGI (2007). Four CPTs have been conducted at each anchor cluster and average-, minimum and maximum values have been established. It is assumed that average values from the CPT is a good approach as basis for the estimations. The minimum and maximum measurements could be seen as the deviation from the average value.

South-West CPT

The South-West cluster CPT shows great differences between the lowest and highest measured q_c, and the largest difference is from depth 12,5 meters and below (see figure A.4). The average cone resistance indicates a higher resistance from 13,5 meters below seabed. The maximum measured sleeve friction increases linear from 9,5 meters below seabed in contrary to the minimum measured, which keeps its linearity throughout the depth.

North-West CPT

CPT measurements from North-West indicates a linear increment for cone resistance q_c with depth according to the average measurement and has almost the same shape as the sleeve friction.

North-East CPT

The North-East cluster indicates that the soil profile is quite homogeneous over the area where the CPT has been taken (see figure A.1). The cone resistance q_c shows a linear increment with depth, however the sleeve friction has a "bulge" in the measurement from depth 7,0 and down to 12,0 meters, before keeping the same linearity as from seabed down to 7,0 meters.

South-East CPT

South-East area has a clearly linear increment for q_c and f_s with small changes between minimum and maximum measurement. The sleeve friction has an increment in resistance from 8,0 to 10,0 meters before continuing linearly downwards.

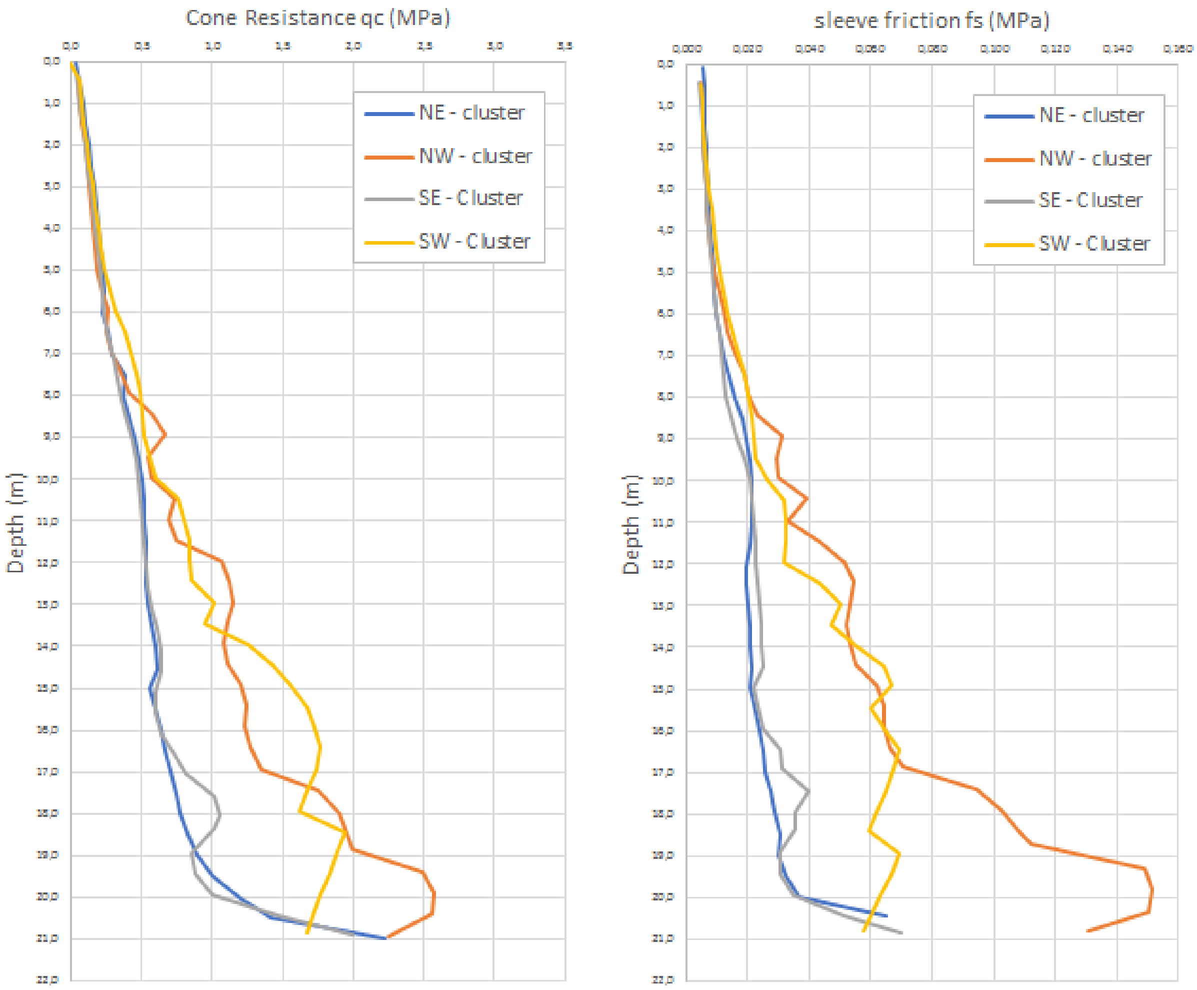
Distances from CPT to actual installation

As mention above one of the biggest uncertainties lies in the distance from where the CPT and samples were collected and where the anchors were installed. From table 5.4 shows the different distances.

Cluster	Avg. distance (m)
SW	125
NW	185
NE	30
SE	139

Table 5.4: Average distance from where CPT and samples were collected and where the anchors were installed.

However when cone resistance and sleeve friction is plotted figure 5.2a and 5.2b respectfully a correlation between the values can be seen. The average values for NE and SE cluster is roughly identical, while going west to cluster NW and SW they show an identical increment. From this it seems that the soil properties is quite homogeneous in the longitudinal direction, and increasing in the latitudinal direction (east to west). From the detailed description above the CPT data from NE and SE has a lower variance than NW and SW. Meaning that the values used to estimate penetration resistance for the eastern values should give a good estimate, and the estimate for the western clusters could higher or lower than what's measured.



(a) (b)

Figure 5.2: Average cone resistance and sleeve friction for all clusters.

5.2.2 Soil undrained shear strength

The recommended undrained shear strength profile for all the anchor clusters is based of laboratory test in conjunction with CPT measurements. The undrained shear strength s_u^C profiles for each cluster is presented in appendix B. To estimate the direct shear strength and extension the relationship presented below has been used as recommended by NGI (2007).

$$s_u^{DSS}/s_u^C = 0,85$$

$$s_u^E/s_u^C = 0,65$$

The recommended remoulded shear strength given in NGI (2007) is based on several different laboratory tests and sleeve friction f_s.

The sleeve friction is multiplied with the average value of remoulded shear strength from fall-cone test over sleeve friction. When multiplying this factor with sleeve friction, one will obtain the expected $s_{\mathrm{u,rem}}$ with depth based on sleeve friction. For simplicity and engineering purpose this factor is a fixed number for a depth interval. The factor used and measured factors with depth is presented in figure 5.3.

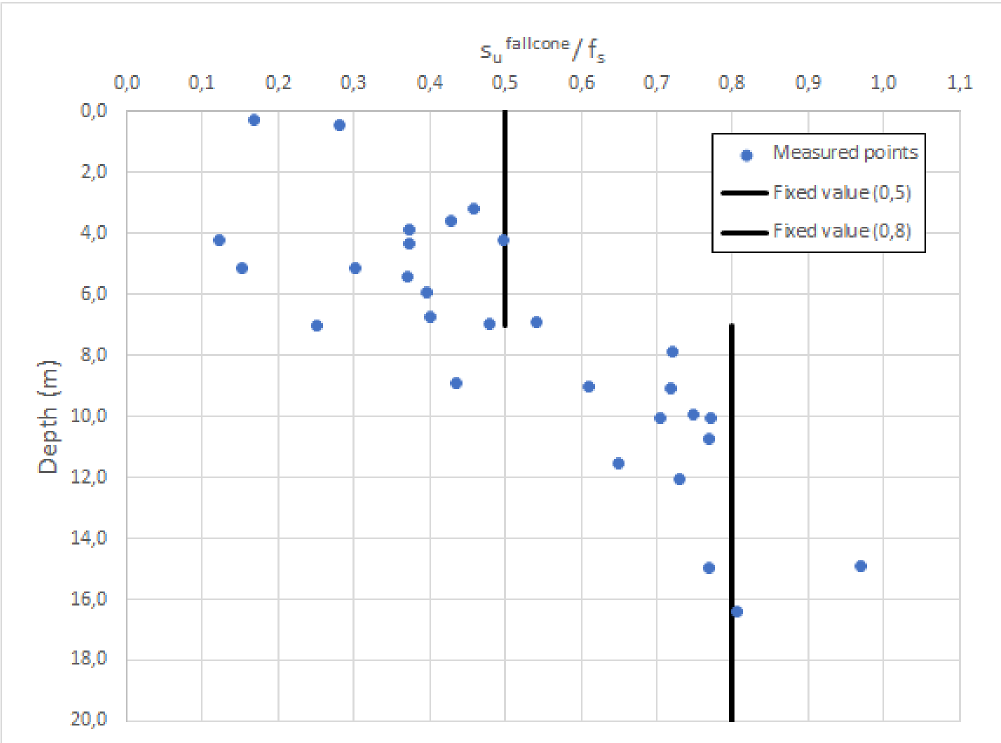


Figure 5.3: Sleeve friction factors versus depth. Both measured and used values are presented (Source: NGI, 2007).

The remoulded shear strength recommended $\mathrm{s_{u,rem}}$ is presented in appendix C. The basic method in section 3.3 suggest the use of an adhesion factor α, being the inverse of sensitivity S_t.

Due to the large differences from the lower and upper values for measured the difference between the $\mathrm{s_{rem}}$ based on $(1/S_t) \cdot \mathrm{s_u^{DSS}}$ where the use of either expected undrained shear strength or either lower and upper bound undrained shear strength will give large variance. When using the inverse sensitivity to reduce the undrained shear strength to achieve good estimates the local sensitivity should be used in conjunction with the local undrained shear strength.

In 2008 Endre Magnus Høva back calculated 46 suction anchors with geometry with close re-

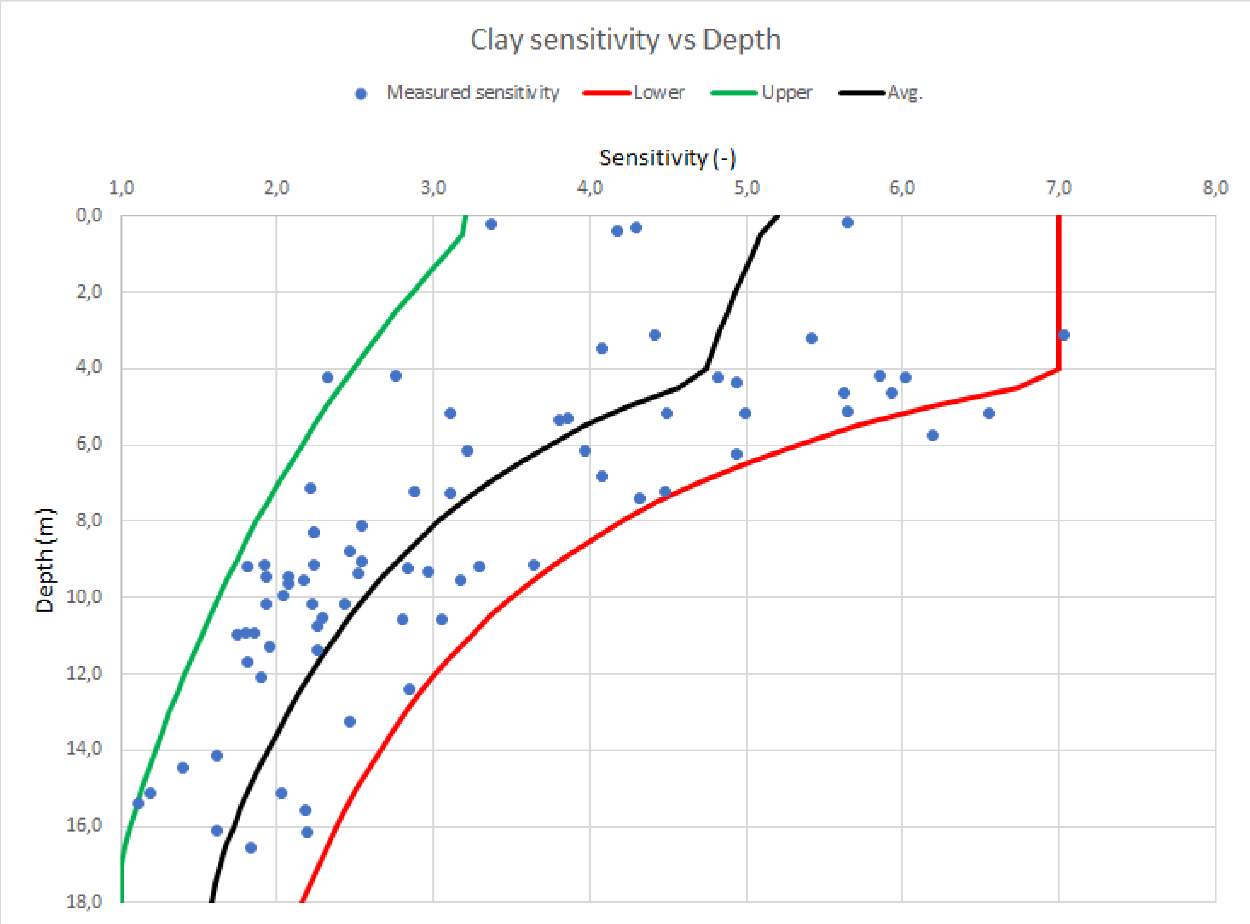


Figure 5.4: Scatter plot of measured sensitivity for suction anchors at Gjøa field (Source: NGI, 2007).

semblance to what used at Gjøa (Høva, 2008). The back calculation included local sensitivity and s_u^c. The back calculation where based on equation 5.1 and 5.2.

$$k = \frac{(\text{Self weight} + \text{Suction} - \text{crane tension}) - \text{tip resistance}}{\text{Skirt resistance}} \approx \frac{1}{S_t} \tag{5.1}$$

$$\alpha = k \cdot S_t = \frac{1}{S_t} \cdot S_t = 1 \tag{5.2}$$

These two equations show that if $\alpha-$value is equal to 1 the used sensitivity and undrained shear strength used is equal to the actual remoulded shear strength. For Høva's research he uses active compression test as undrained shear strength. The results from his research shows that $\alpha-$values is ranging between 0,5 and 1,5 for the last two meters of penetration, with an average of 1,0. Looking at the last meters of penetration will rule out noise in the first meters of suction.

What this research indicates is that the use of sensitivity to derive to remoulded shear strength is for all practical purpose a good estimation given that the local values are used. Høva also argues that the deviation in $\alpha-$value will be ruled out by factor of safety.

Sleeve friction at Gjøa

Figure 5.5 shows how sleeve friction is different from laboratory measured values. The graphs show that sleeve friction f_s is quite much higher than the measured values conducted in the laboratory. Reason for this could lay in the fact that sleeve friction is measured close to the cone, to which degree is the soil remoulded so close to the tip? Looking at the shape, the sleeve friction follows the laboratory test quite well.

5.3 Anchor Geometry

The suction anchors installed at the Gjøa field is of cylindrical shape with a dome top. Height of the anchors varying from 15,7 to 18,5 meters, with a weight of 112 tonnes to 129 tonnes respectfully and an outer diameter of 5,5 meters. Thickness of the steel varying from 30 mm in the tip to 70 mm in the area of the pad-eye.

Availability of detailed drawings of the anchors utilised is limited. The only drawing available is the anchors from cluster SE (18,5 m), and will be the basis for some assumptions for the other anchors. End thickness for all the anchors is set to 30 mm and the average thickness[1] for the hole anchors is set to 45 mm. No ring stiffeners is observed from the drawings.

The calculations include penetration resistance from the pad-eye and the chain by simple assumptions. The pad-eye and chain is added as a square box (0,5 x 0,5 m). All the pad-eyes are assumed reaching 9,0 meters below the seabed.

5.4 Self Weight Penetration at Gjøa

The average self weight penetration were 5,6 meters with a hold back force of 70 tonnes. The different averages self weight penetrations for each cluster is presented in table 5.5.

[1]Average thickness is weighted from the anchors installed at SE cluster bases on thickness and the area that covers this thickness.

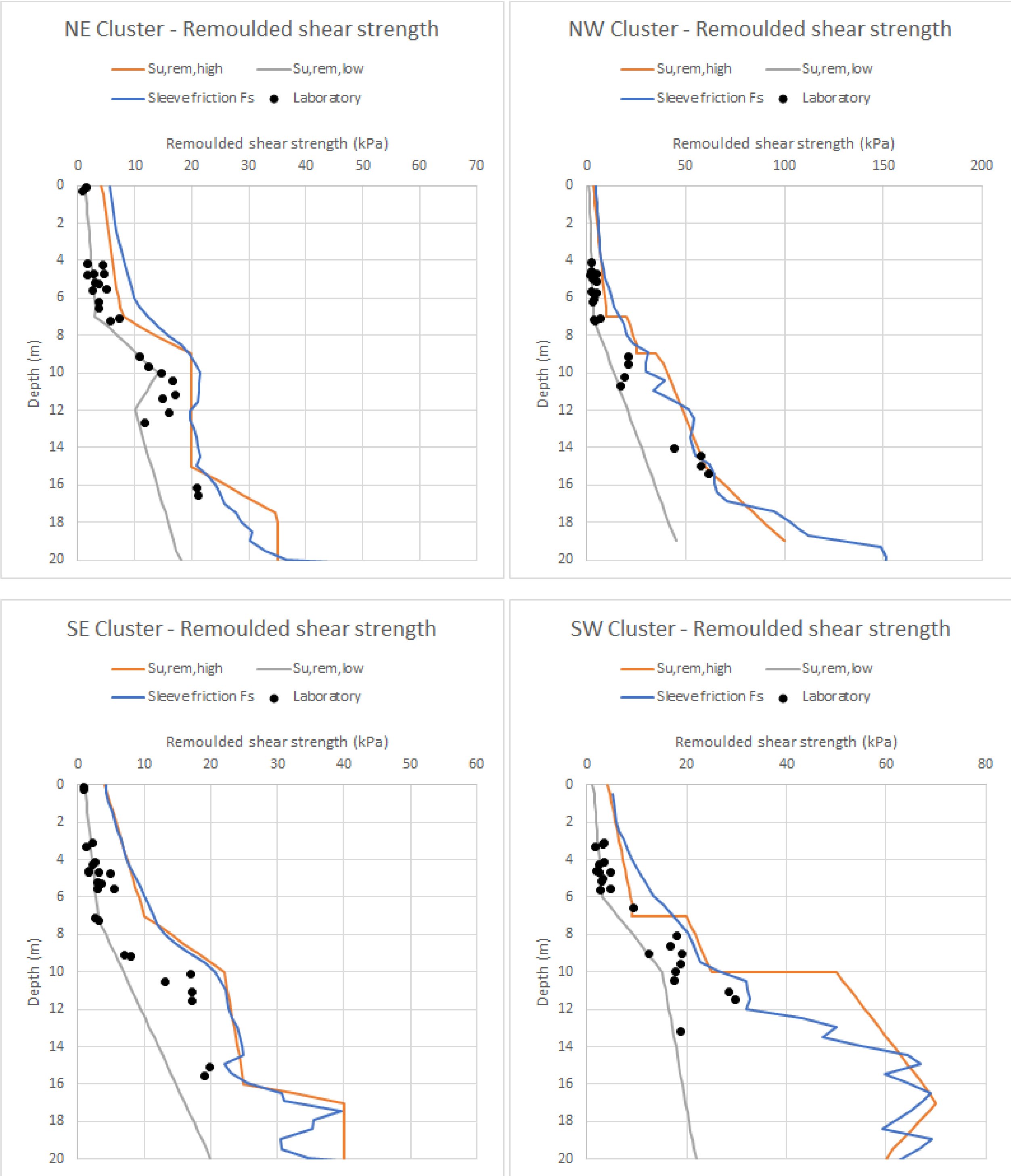


Figure 5.5: Sleeve friction f_s is plotted with laboratory values at given cluster.

5.5 Measured Suction

When installing suction anchor it's recommended by to log suction versus penetration depth. This has been done at Gjøa and the average suction needed to reach necessary penetration

Cluster	Height (m)	Weight (Tonnes)	Self weight penetration[1] (m)
SW	17,5	124	5,75
NW	17,5	124	5,83
NE	15,7	112	4,75
SE	18,5	129	6,14

[1] Average values of the four anchors at each cluster.

Table 5.5: Geometrical values and self weight penetration of the different anchor clusters at Gjøa.

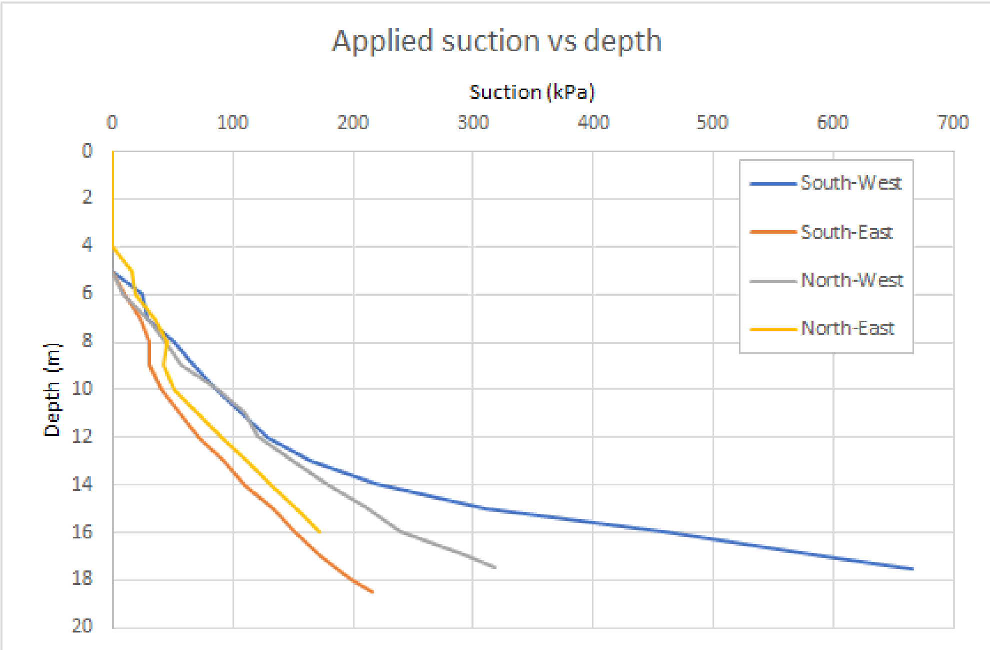


Figure 5.6: Measured suction in kPa. The suction is an average value all four anchors at each cluster.

depth is presented in figure 5.6 (detailed suction values see appendix D). Cluster SE, NW and NE has a clear fall in penetration in the range of 7-9 meters below seabed. This occurs when the crane tension is release from 70 to 30 tonnes (Eide et al., 2009). This drop is vague for SW cluster, but a drop can be seen in the range 6 to 7 meters. The suction for SE, NW and NE is quite clear from the graphs.

However, for SW cluster the measured suction increases dramatically when reaching 12 meters and below. Why this happens is challenging to explain based on the resemblance of the CPT taken for NW and SW. The anchors are installed in between the CPT for NW and SW and there is no clear indication is rapid increment in cone resistance or skirt friction below 12 meters. One argument could be that the anchor hits a large rock, but since this increment is equal for all anchors in SW cluster it has must a layer of rock in this area which seems strange. Another argument could be related to thixotropy[2] and penetration time, which also seems strange since the average penetration time for SW is 10 minutes higher than the average of all the clusters. The last and most probable explanation is that the bottom layer is higher due to change in rock topography bringing up the stiffer layer.

5.6 Possible Errors

When working in a spatial environment with a complex material with shifting characteristics based on history, depth, size, shape, and several other aspects it's then important to be aware of the possible errors. The possible errors regarding the data basis which could affect the results could be divided into three groups.

1. Soil characteristics and measurements

The laboratory procedures are standardised and would seem unreasonable to assumed large errors in the data provided by the laboratory. One major uncertainty as mentioned section 5.2.1 is the distance between the CPT measurement and the position of where the anchor were installed.

The CPT measurement are of good quality and is representative for the in-situ condition accordingly to NGI (2007).

2. Anchor geometry and weights

Regarding anchor geometry and weights might give errors. The anchor geometry is only given for SE cluster and has been the basis for SW, NW, and NE cluster. The thickness of the steel skirt shouldn't give dramatic changes in penetration resistance. Another argument for the increased measured suction for the SW cluster lays in the geometry of the anchor, if there is an increment in skirt area due to vertical plates crossing the internal structure this could result in higher resistance. Again this seems strange, since this should be visible on the required upper and lower bound suction presented in figure D.1.

Anchor weight is given as dry weight, converting this to the submerged weight will also provide some uncertainties to the final result. The anchors used at SE cluster is 129 metric tonnes,

[2]Thixotropy is when a sample has been remoulded and then gain strength back due to time (NGI, 2007).

and was assumed by Christensen and Eiksund (2009) to be 111 tonnes. Then the other anchors submerged weights is assumed with this relationship (e.g. $(111/129) \cdot 112 = 96,4$ tonnes for the 15,7 meter anchors used at NE cluster). If the chain weight is added to the dry weight is unknown and different lifting equipment weight is also unknown, providing uncertainties to the total weight. The uncertainties related to weight could have a great impact on the estimation on the self weight penetration and further penetration. As displayed in figure 5.6 a tension release of 40 tonnes results in several meters of penetration, with that in mind a offset of 10 tonnes could give deviation in the ranger of 1,0 meter more or less, especially in the first meters of penetration in the soft clay.

3. Measured values during installation

How the suction force is registered could vary from digital readings to visual readings with the use of manometers. Where the suction is registered visually in poor sight conditions the reading could be wrong. Høva (2008) puts down measured suction as an uncertainty. It is unknown which method which have been used to registered suction but it is assumed to be of good quality.

How the penetration depth is measured is also unknown. This could also be done visually, digitally with echo sounder or with barometer on-board the ROV registering change in pressure while penetrating.

Halt in penetration could lead to set-up in the clay (see section 5.3), no delay in penetration is registered in the installation log (Eide et al., 2009) and it's assumed that increased penetration resistance due to set-up is rejected.

Chapter 6

Results

In this chapter results and findings is presented regarding estimating necessary suction when installing suction anchors. Further discussion of the results is presented in chapter 7.

6.1 Input Parameters

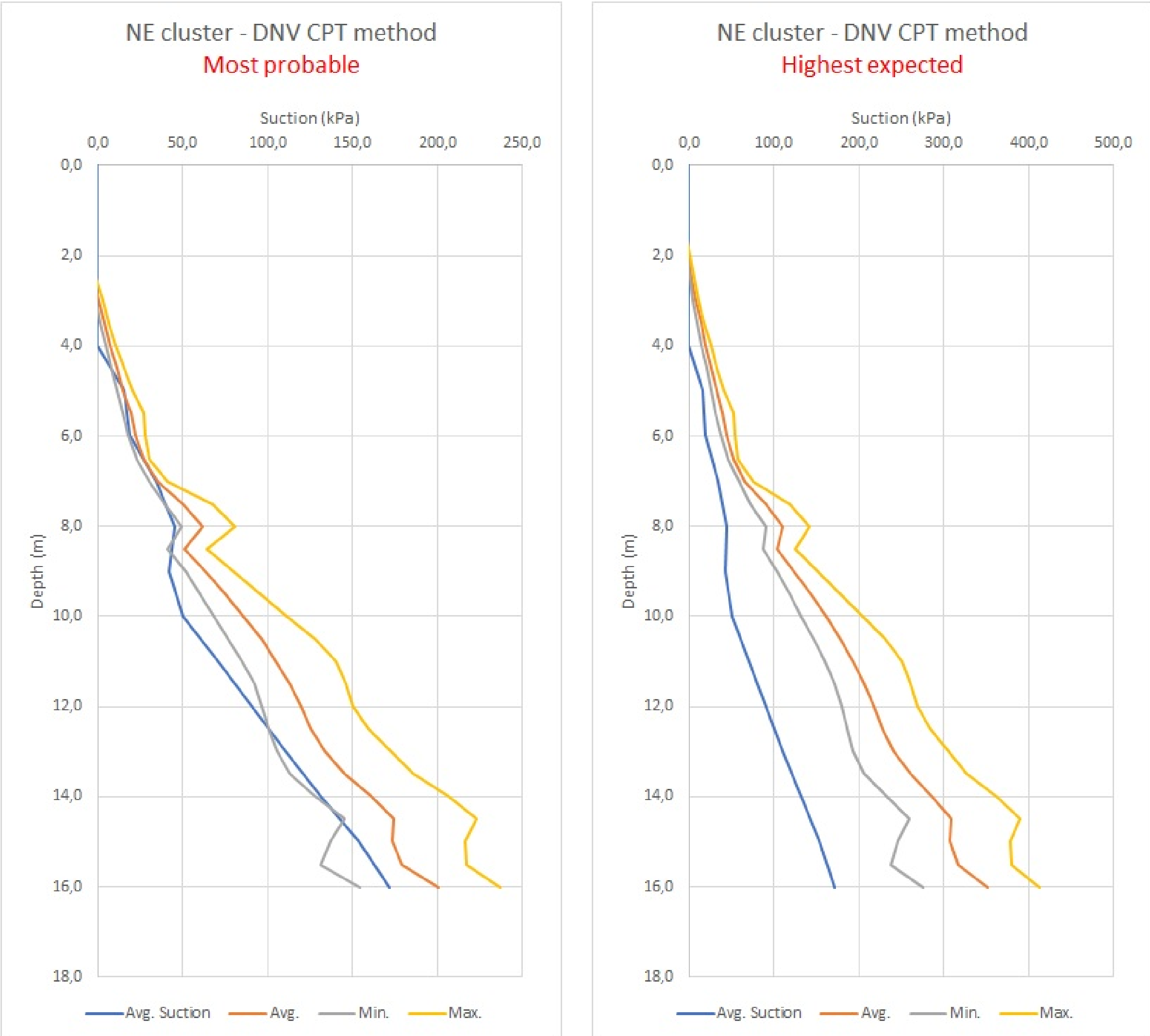
Cluster	Diameter (m)	Height (m)	Weight[1] (Tonnes)	Tension release (m)	s_u	$s_{\text{u,rem}}$	CPT	ϵ
NE	5,5	15,7	96,4	7,0	B.1	C.1	A.1	0,5
NW	5,5	17,5	106,7	7,0	B.1	C.1	A.2	0,5
SE	5,5	18,5	111,0	7,0	B.2	C.2	A.3	0,5
SW	5,5	17,5	106,7	6,0	B.2	C.2	A.4	0,5

[1] Being the submerged weight.

Table 6.1: Input parameters used for the calculations.

6.2 DNV CPT Method

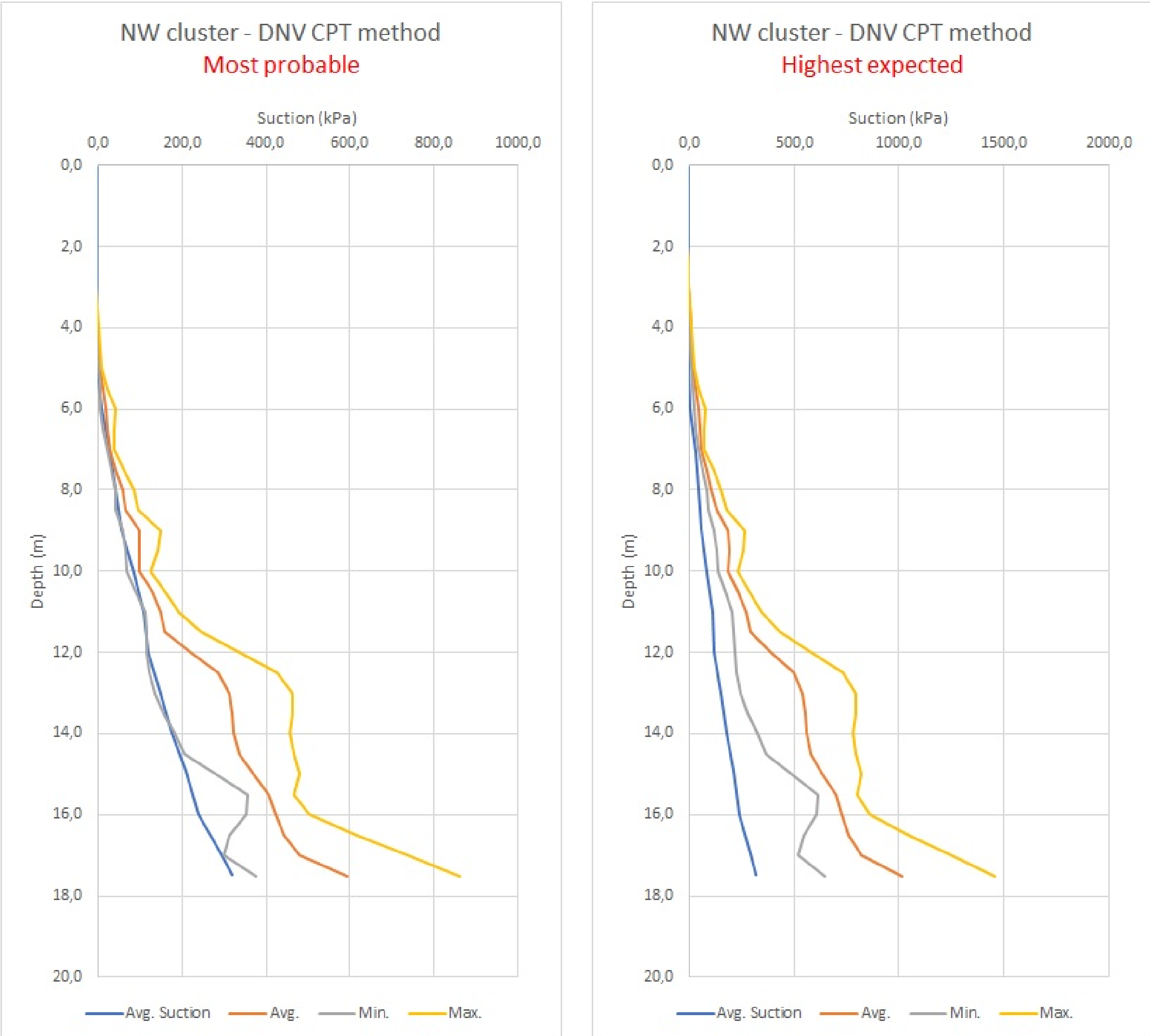
For all anchor cluster the penetration resistance has been calculated by DNV recommended CPT method given in DNV (1992). The tip and skirt factors used is equal to the one presented in table 3.1. The results are presented from figure 6.1 to 6.4.



(a) Estimated penetration resistance at NE-cluster, with the usage of most probable k_p and k_f values.

(b) Estimated penetration resistance at NE-cluster, with the usage of highest expected k_p and k_f values.

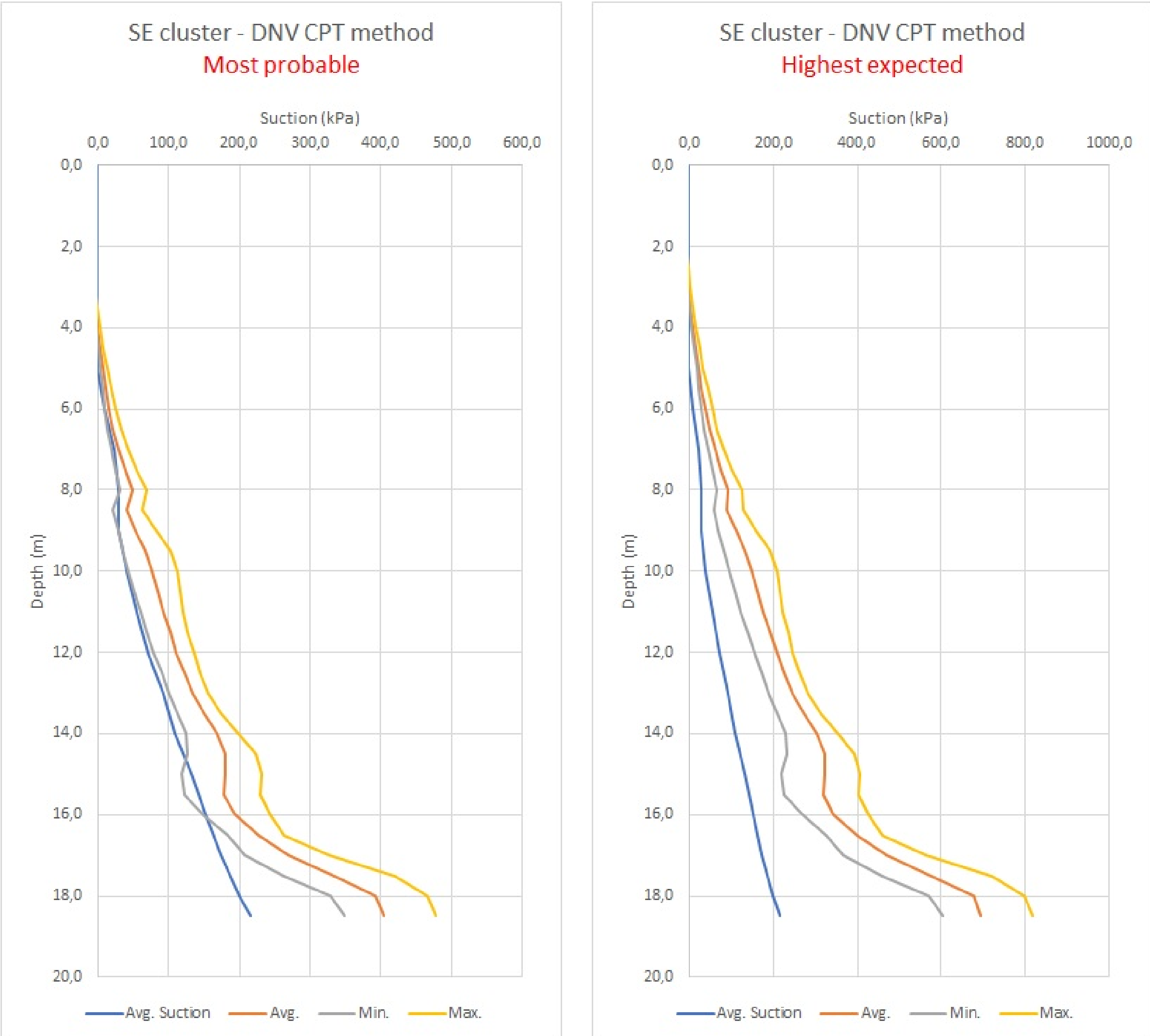
Figure 6.1: DNV CPT method for NE cluster



(a) Estimated penetration resistance at NW-cluster, with the usage of most probable k_p and k_f values.

(b) Estimated penetration resistance at NW-cluster, with the usage of highest expected k_p and k_f values.

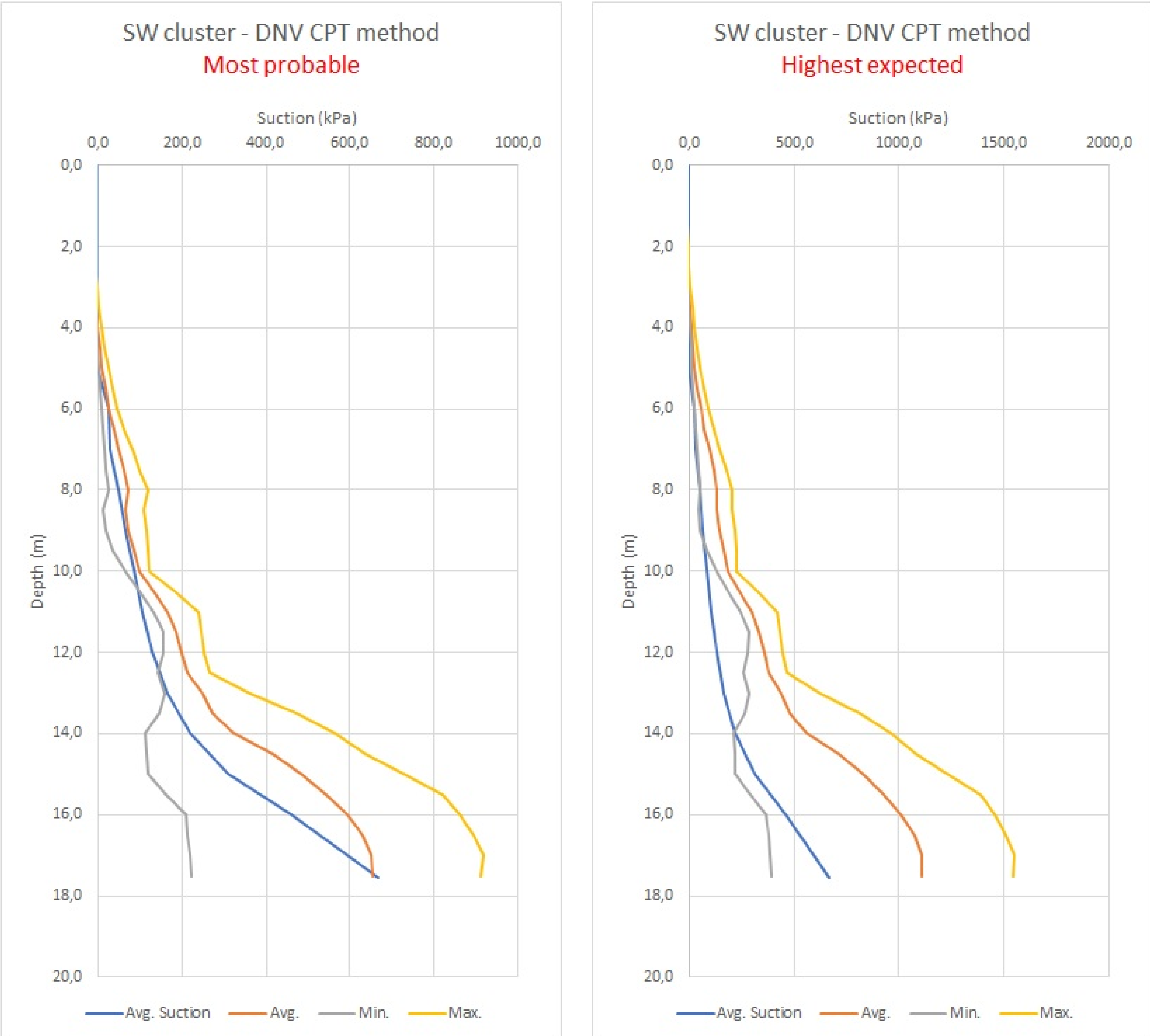
Figure 6.2: DNV CPT method for NW cluster



(a) Estimated penetration resistance at SE-cluster, with the usage of most probable k_p and k_f values.

(b) Estimated penetration resistance at SE-cluster, with the usage of highest expected k_p and k_f values.

Figure 6.3: DNV CPT method for SE cluster



(a) Estimated penetration resistance at SW-cluster, with the usage of most probable k_p and k_f values.

(b) Estimated penetration resistance at SW-cluster, with the usage of highest expected k_p and k_f values.

Figure 6.4: DNV CPT method for SW cluster

6.3 DNV RP E303 Method

The result when using recommended method by DNV (2005) described in section 3.2.2 is presented in figure 6.5 to 6.8. Here are also the recommended soil strength values used, giving an lower- and upper bound. For more detail about input values used see section 6.1. The estimations have also been conducted on measured local values, except for SW cluster due to insufficient data below 14 meters.

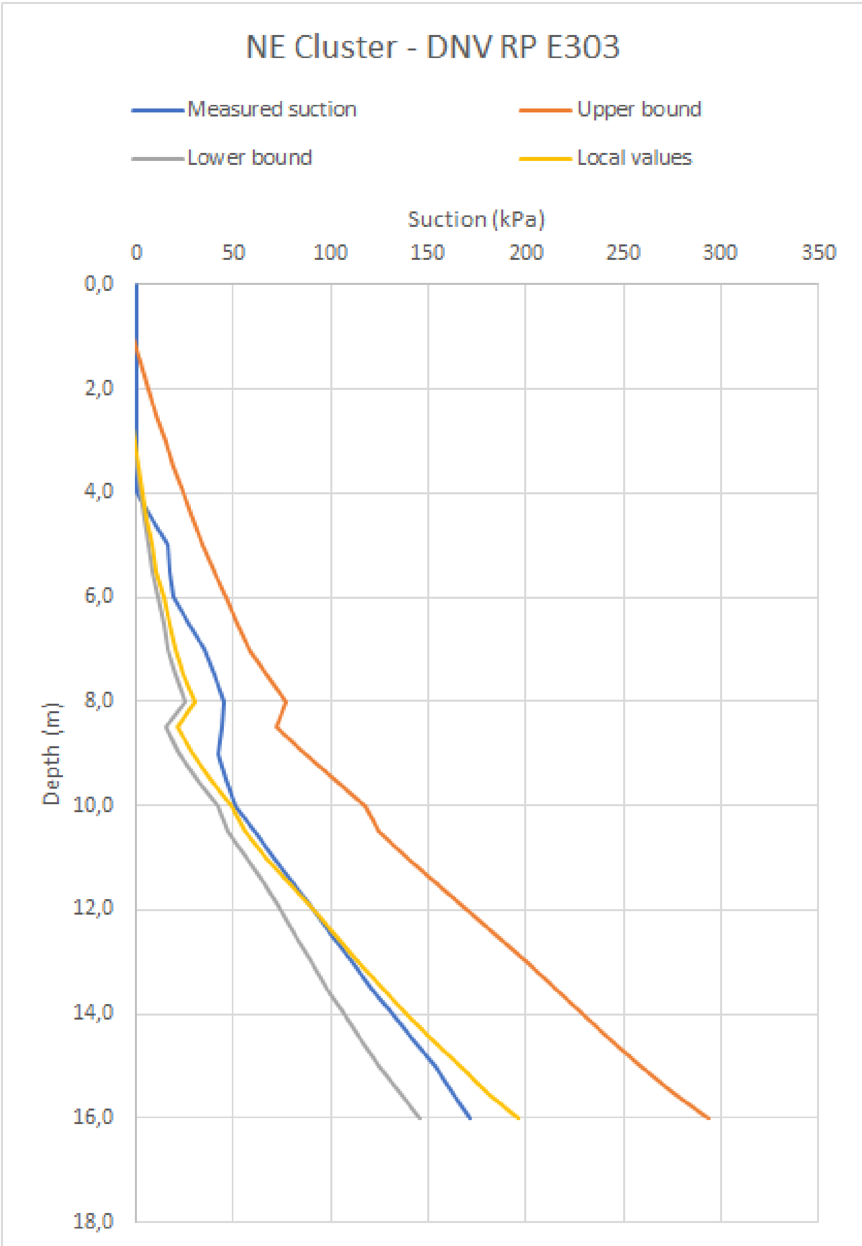


Figure 6.5: Penetration resistance estimated for NE cluster after recommendations given in section 3.2.2.

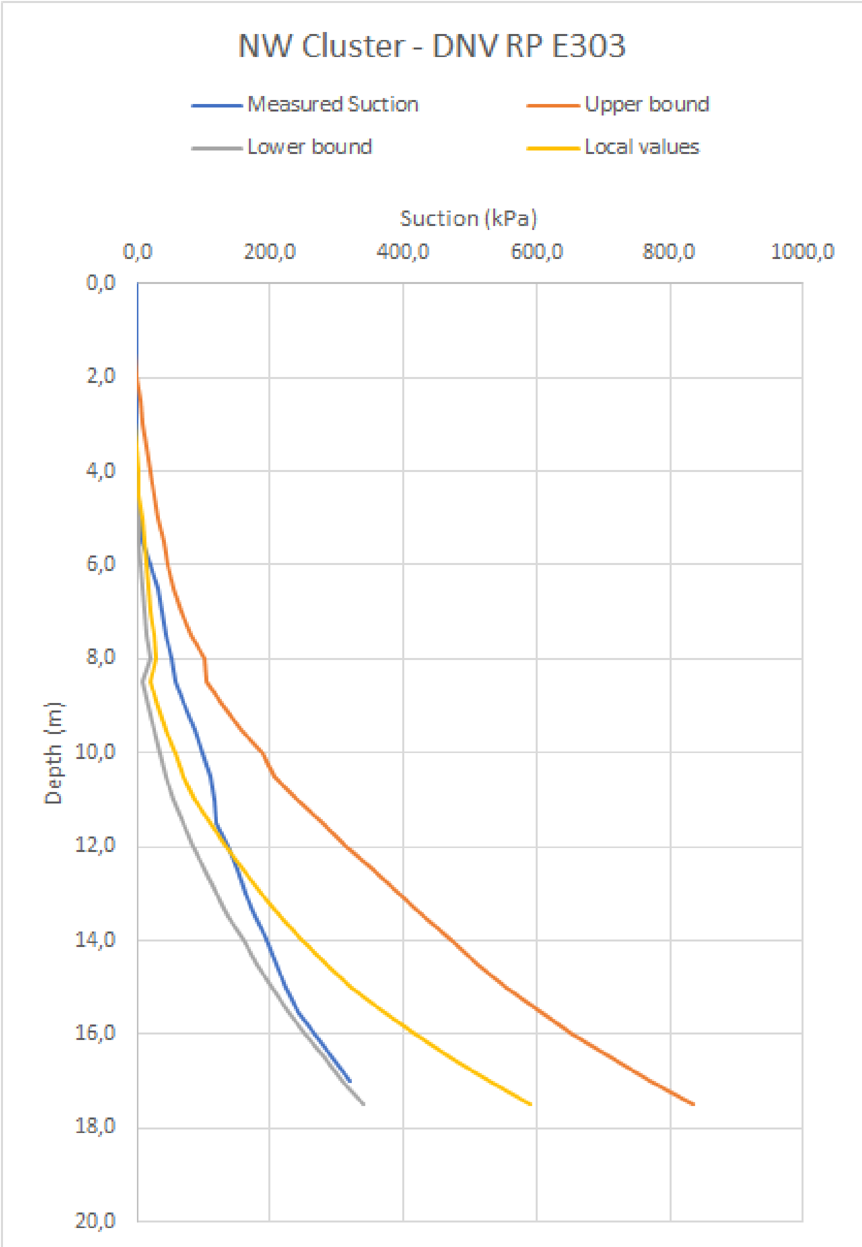


Figure 6.6: Penetration resistance estimated for NW cluster after recommendations given in section 3.2.2.

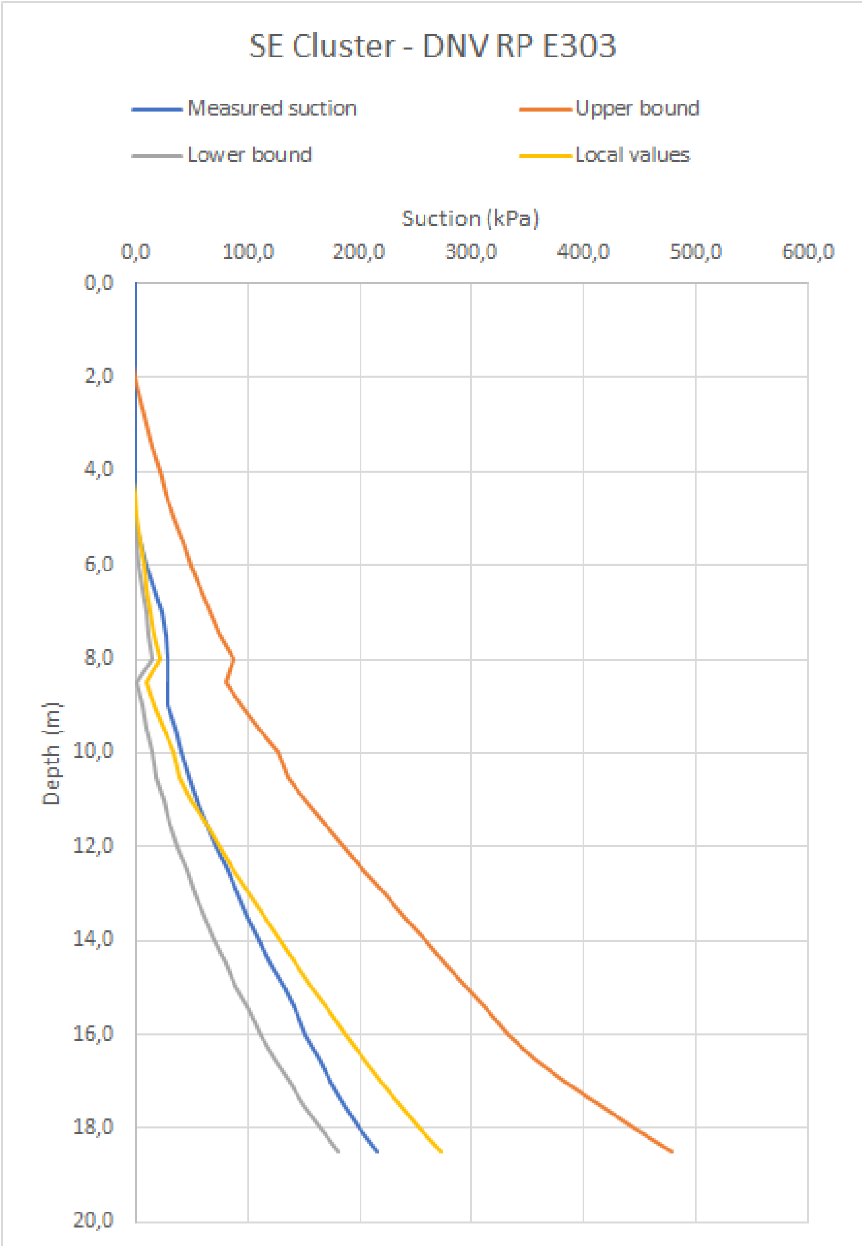


Figure 6.7: Penetration resistance estimated for SE cluster after recommendations given in section 3.2.2.

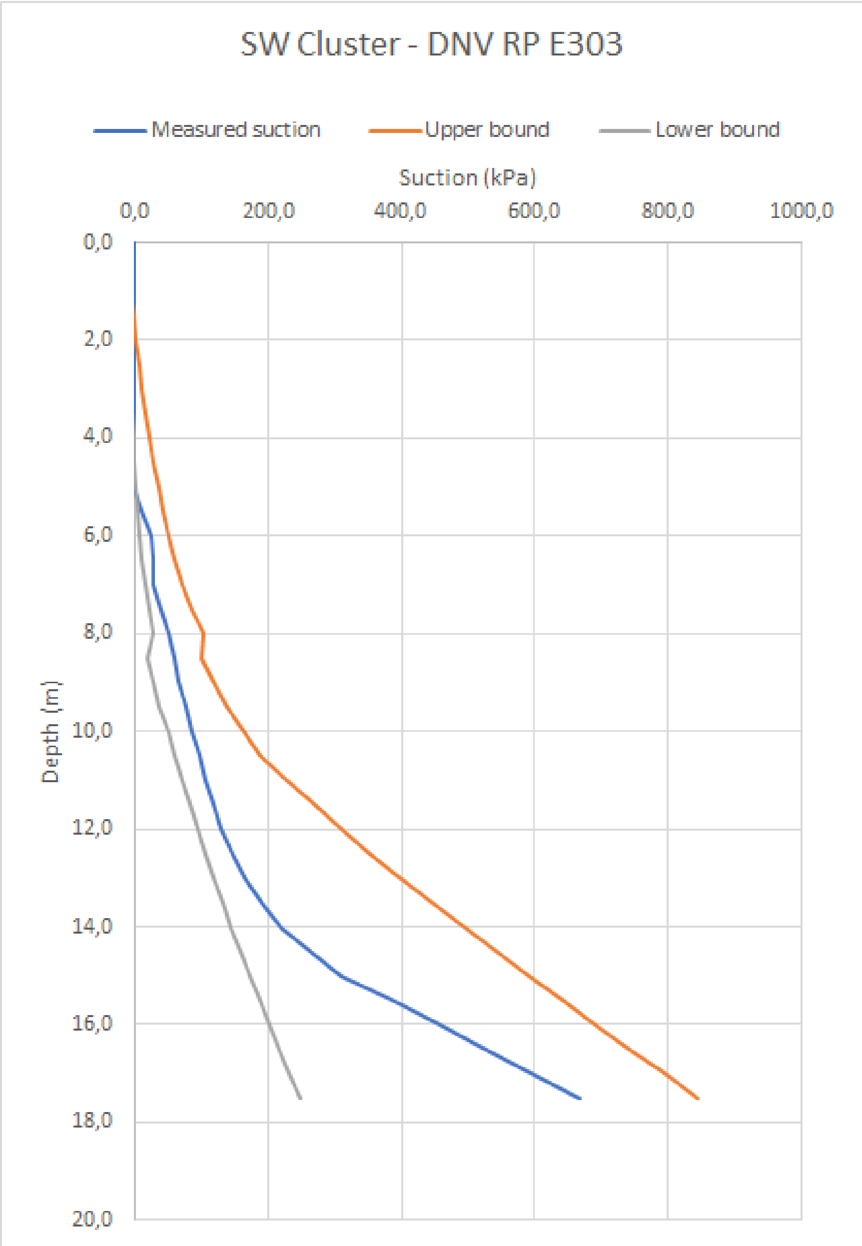


Figure 6.8: Penetration resistance estimated for SW cluster after recommendations given in section 3.2.2.

6.4 Penetration Resistance Estimation Based on Sleeve Friction

Penetration resistance estimated with proposed method in section 3.6 has been conducted for all clusters at Gjøa. The results is presented in figure 6.9 to 6.12. The chosen ϵ value is set to 0,5 for all clusters over the total depth.

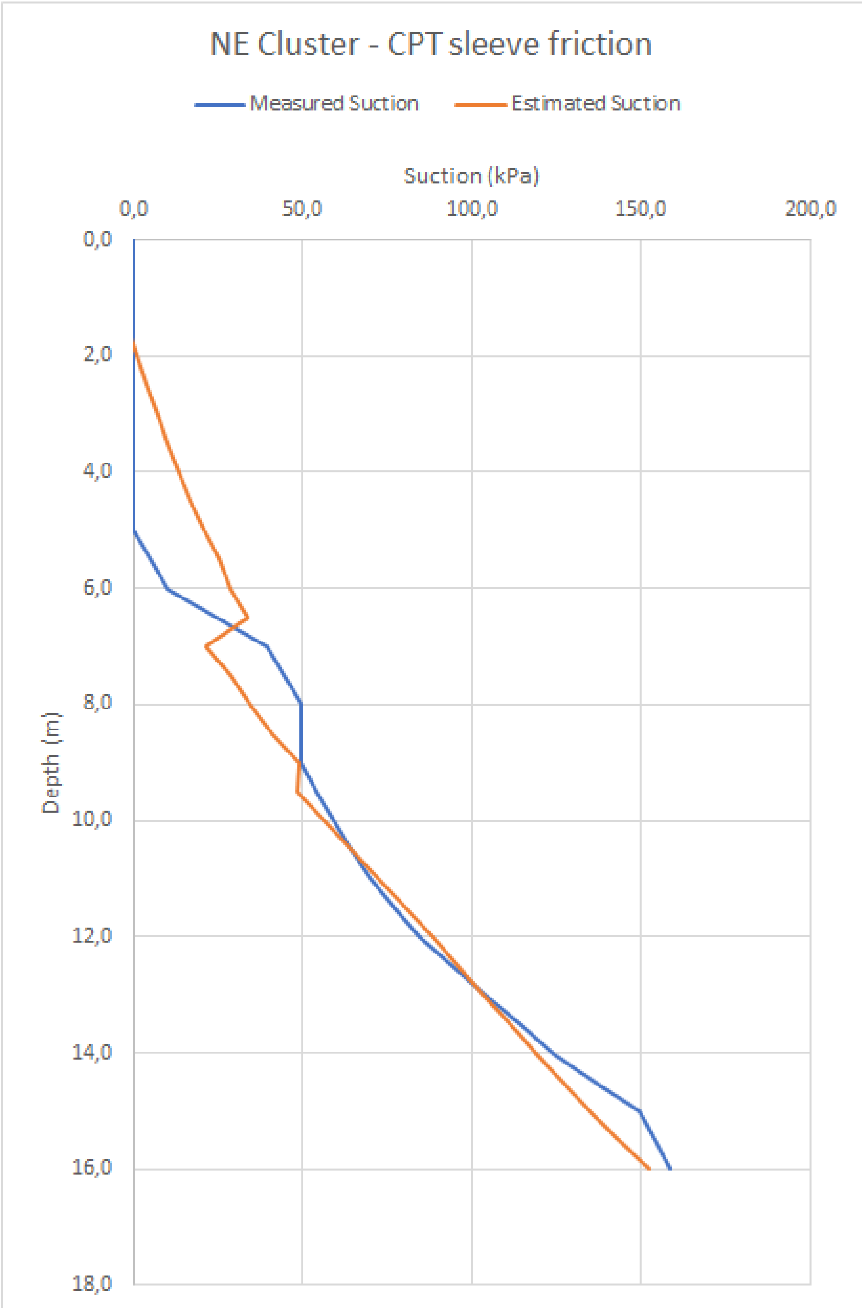


Figure 6.9: Results for estimated penetration resistance based on sleeve friction f_S for NE cluster.

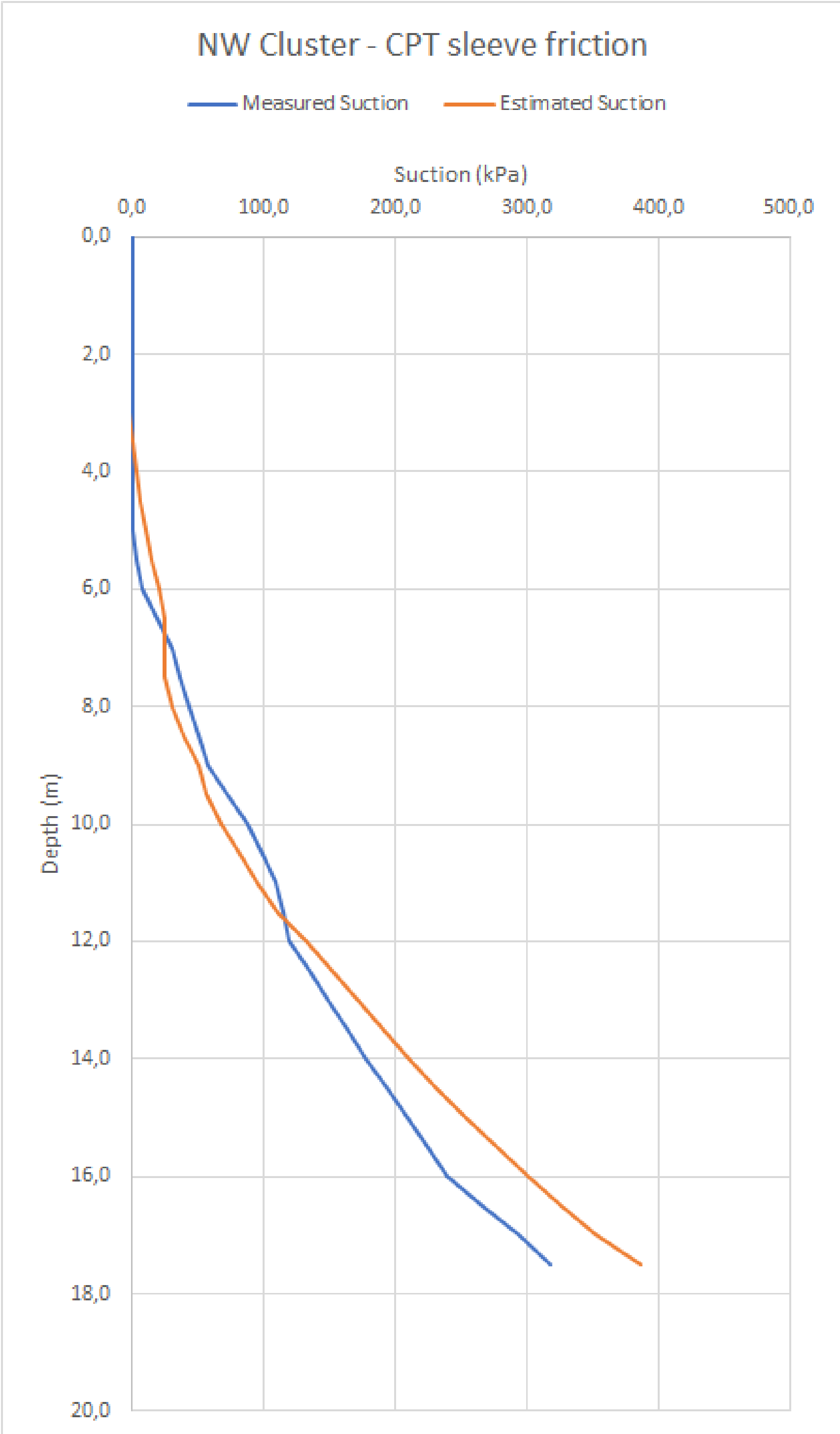


Figure 6.10: Results for estimated penetration resistance based on sleeve friction f_S for NW cluster.

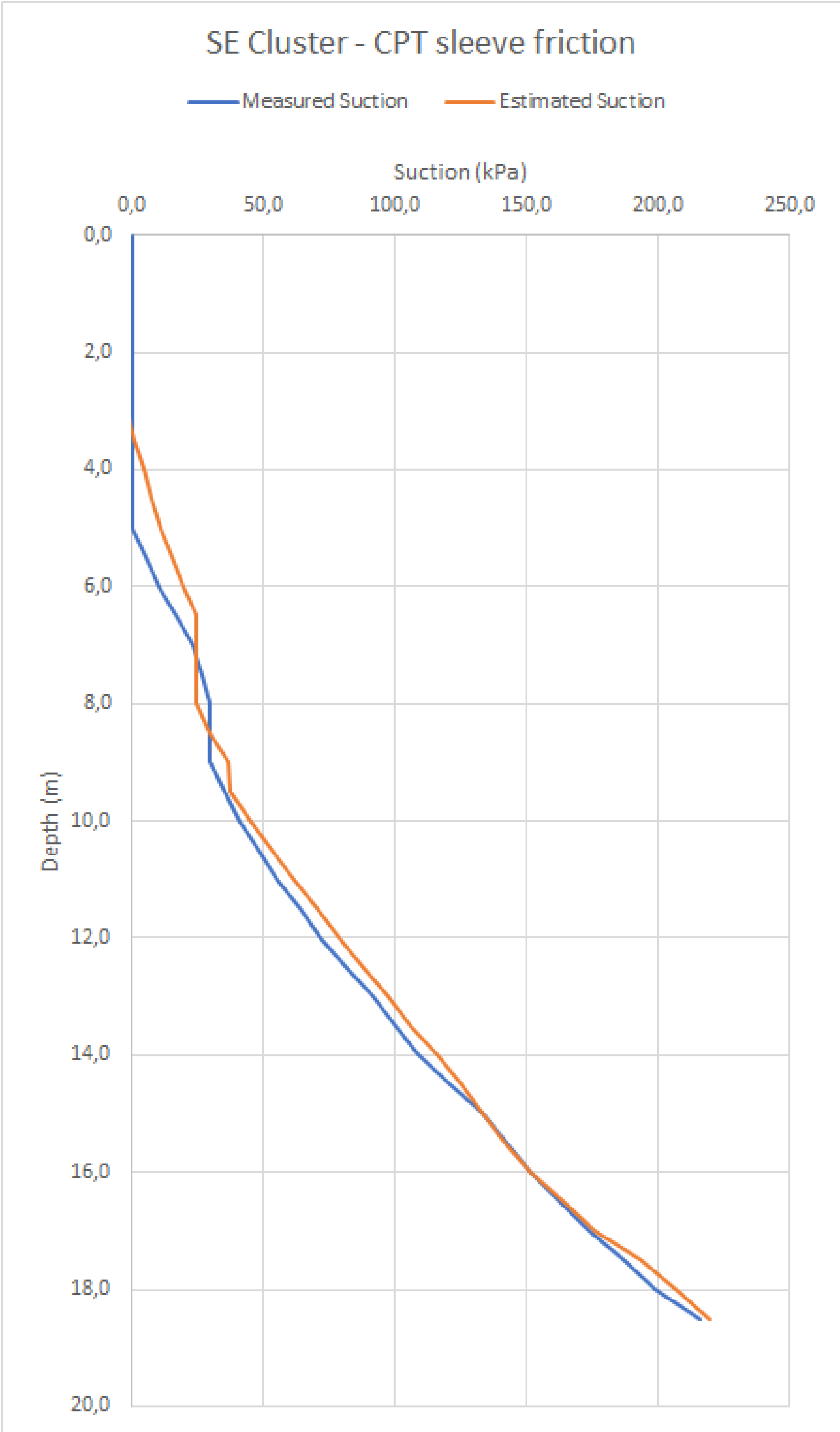


Figure 6.11: Results for estimated penetration resistance based on sleeve friction f_S for SE cluster.

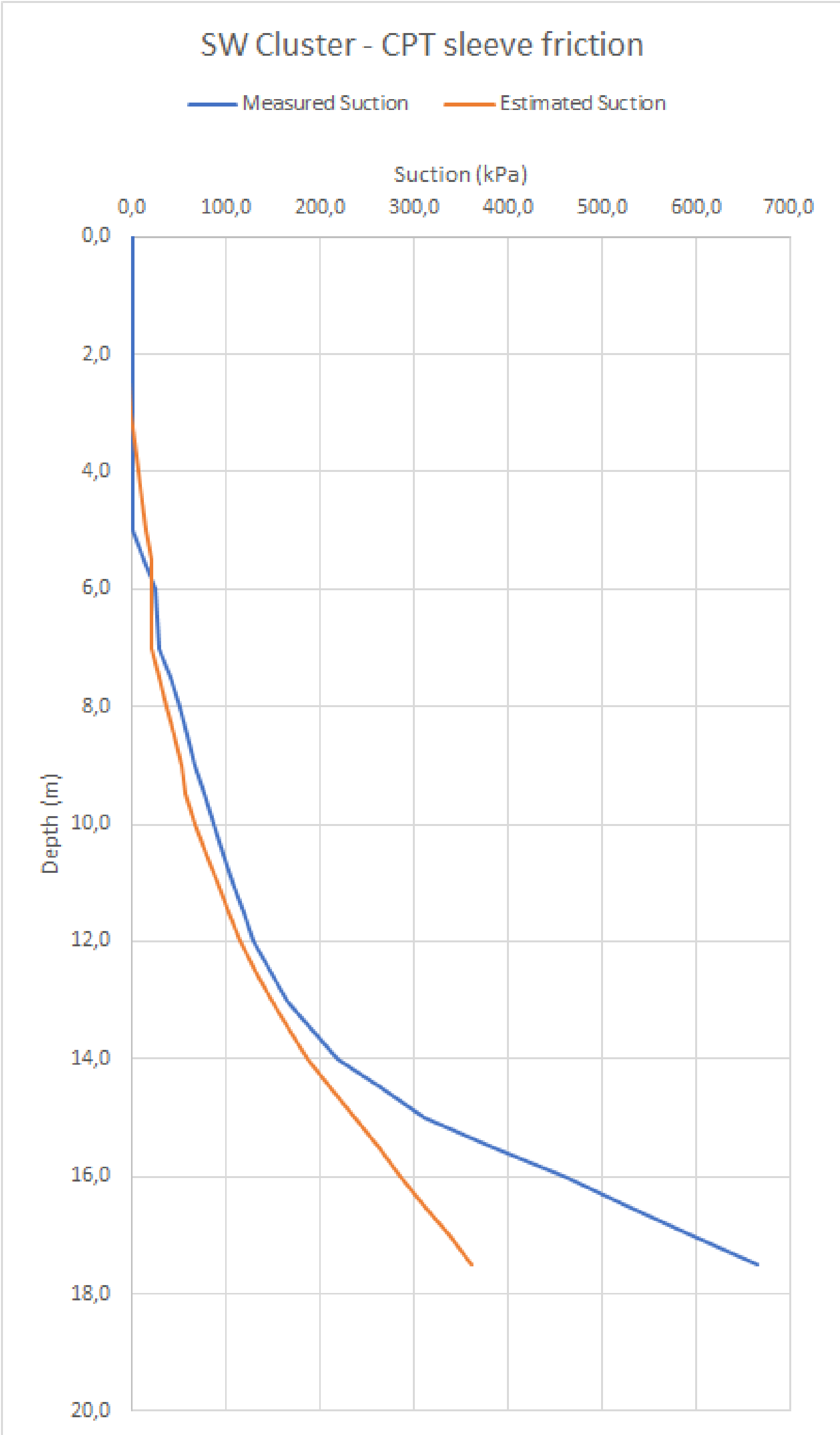


Figure 6.12: Results for estimated penetration resistance based on sleeve friction f_S for SW cluster.

6.5 Back Calculation of ϵ-value

The value ϵ as a function of depth has been back calculated with equation 6.1.

$$\epsilon(z) = \frac{Q_{\text{side,ms}}(z)}{A_s \int_0^d \overline{f_s}(z)\text{dz}} \tag{6.1}$$

Where $Q_{\text{side,ms}}$ is the skirt resistance back calculated from the measured suction. The back calculation of $Q_{\text{side,ms}}$ is showed below and are based on equation 3.4.

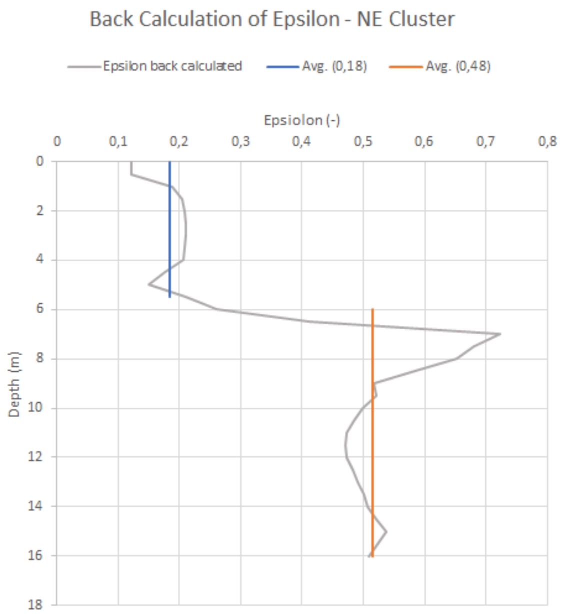
$$Q_{\text{tot}} = \text{Measured suction} \cdot A_{\text{in}} + W'$$

$$Q_{\text{tot}} = Q_{\text{tip},k_p=0,4} + Q_{\text{side,ms}} \qquad \rightarrow \qquad Q_{\text{side,ms}} = \text{Measured suction} \cdot A_{\text{in}} + W' - Q_{\text{tip},k_p=0,4}$$

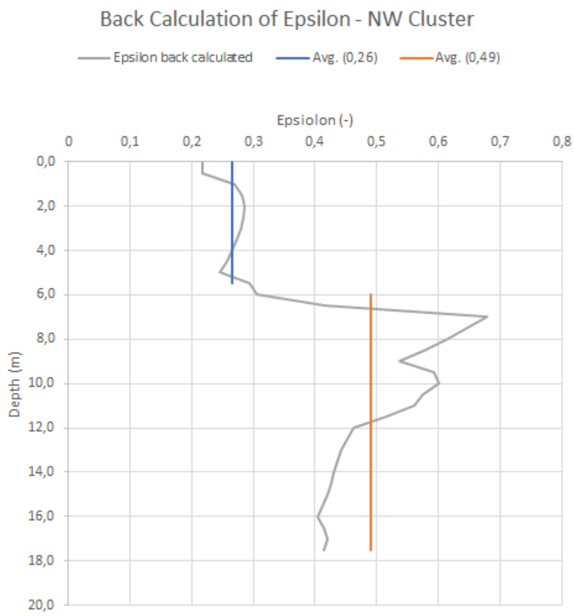
One thing to have in mind utilising this equation is the impact of Q_{tip}. Section 6.9 show the impact of tip resistance and the magnitude of k_p factor. Since the final self weight penetration is known it's possible to estimate the crane tension force which in reality is a function with depth ending at 70 tonnes. Back calculation of ϵ is depicted in figure 6.13 and 6.14, the average value of ϵ above self weight penetration and below is listed in table 6.2.

Cluster	Back calculated ϵ	
	Above self weight pen.	Below self weight pen.
NE	0,18	0,48
NW	0,26	0,49
SE	0,34	0,49
SW	0,31	0,64

Table 6.2: ϵ values above and below self weight penetration.

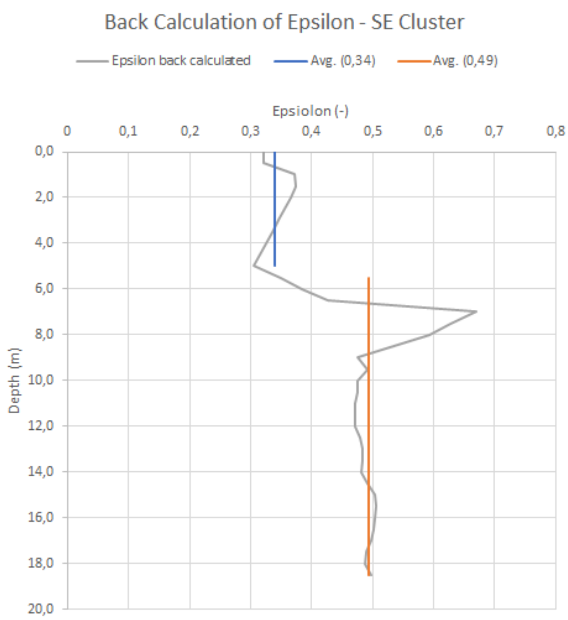


(a) NE cluster.

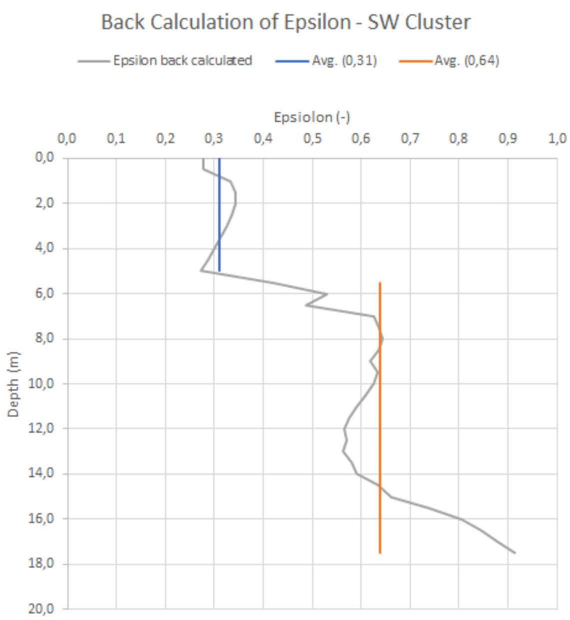


(b) NW cluster.

Figure 6.13: Back calculated ϵ with depth for NE and NW cluster.



(a) SE cluster.



(b) SW cluster.

Figure 6.14: Back calculated ϵ with depth for SE and SW cluster.

6.6 Sleeve Friction f_s with Respect to ϵ

As mentioned in section 5.2.2 the sleeve friction is quite high in respect to the measured values done in the laboratory. When multiplying the back calculated ϵ with sleeve friction f_s the assumed reduced sleeve friction follows the lower laboratory measurement, as seen in figure 6.15.

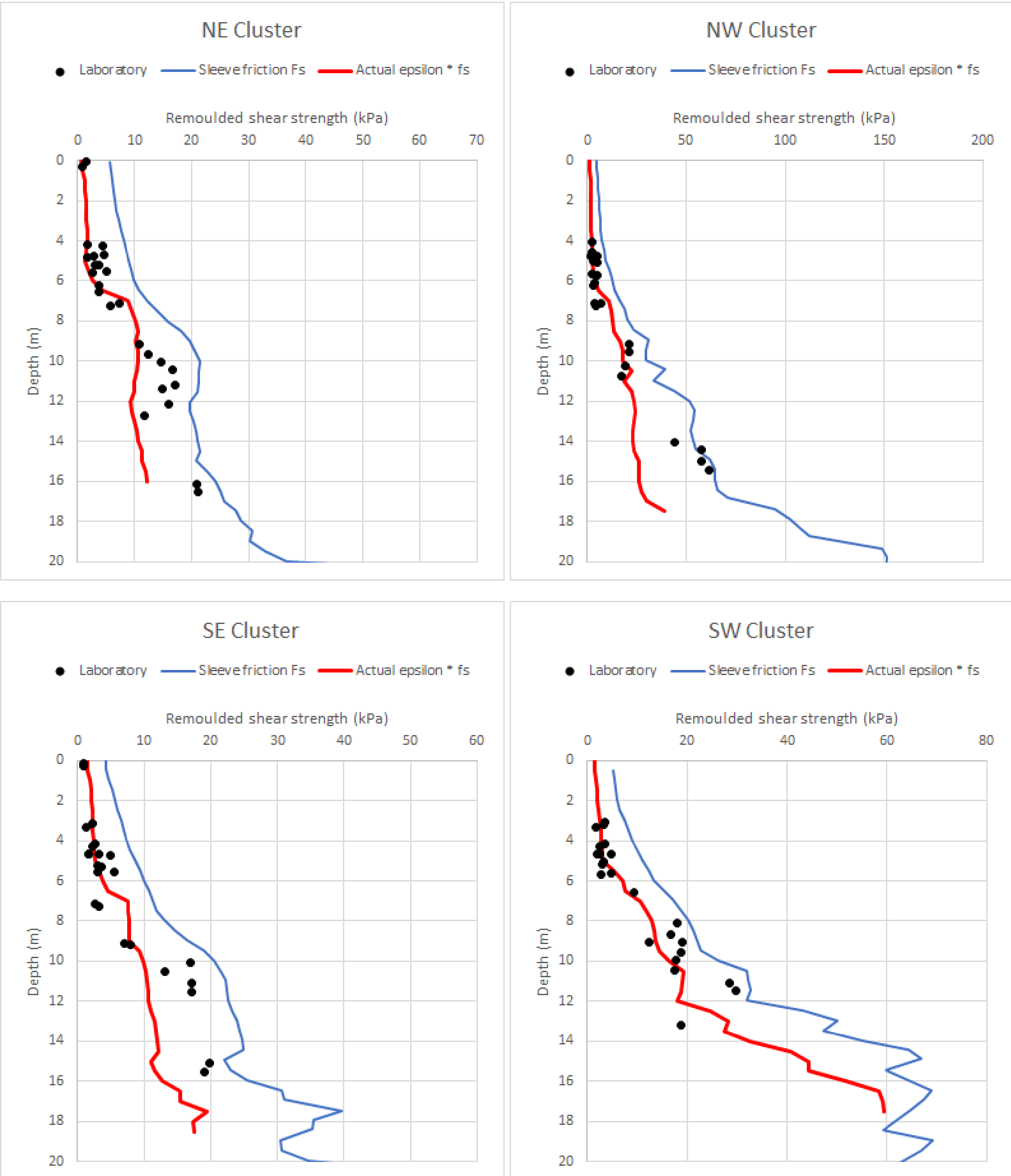


Figure 6.15: Reduced sleeve friction with ϵ.

Described in section 5.2.2, the relationship between fallcone test and sleeve friction is measured with depth. This is done to find and correlate sleeve friction towards actually measured $s_{u,rem}$. Figure 6.16 shows the measured relationships between fallcone and sleeve friction, which one could assume to be equal to ϵ. However, from the back calculated ϵ, this doesn't seem to match.

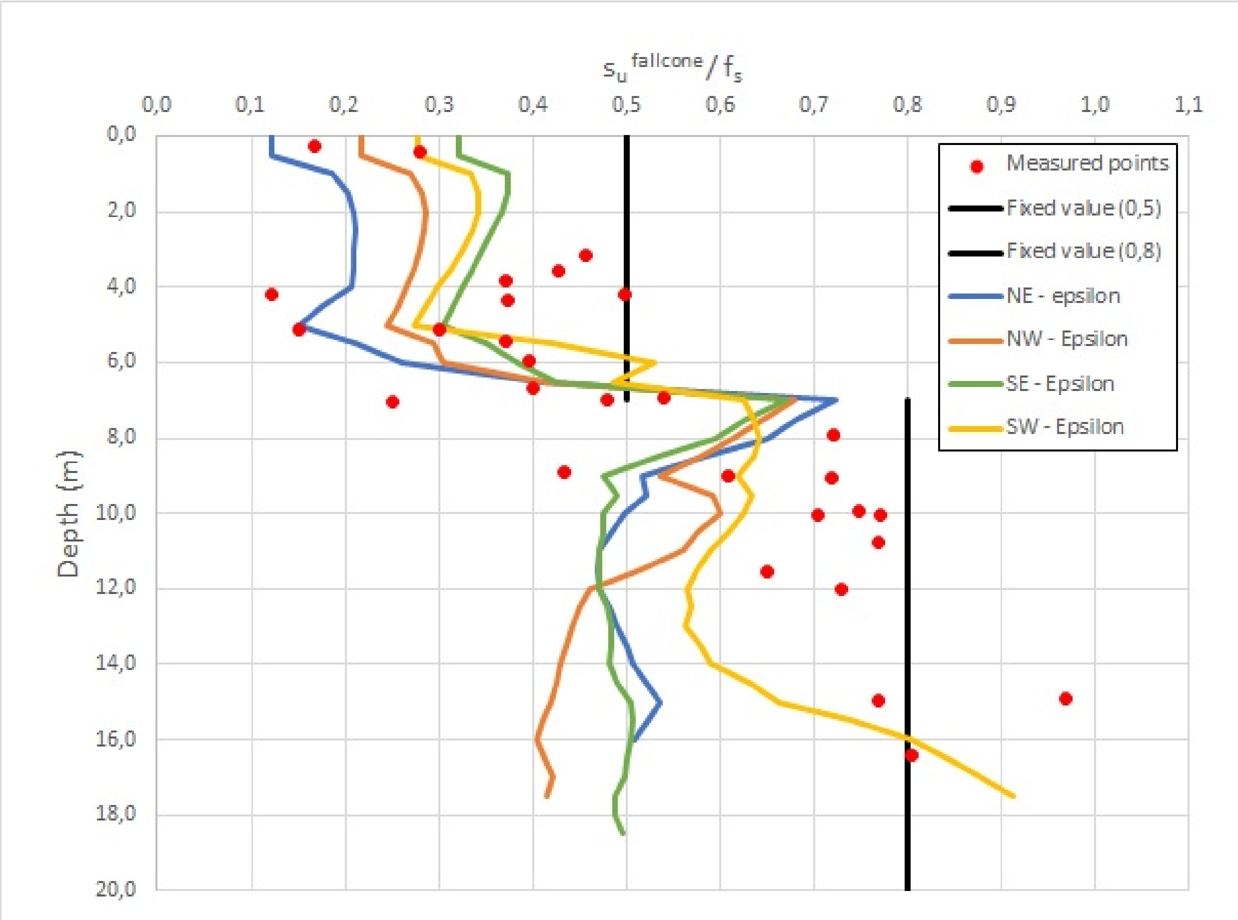


Figure 6.16: Epsilon versus fallcone test from laboratory.

6.7 Epsilon vs. Index Test

Water content usually affects the undrained shear strength of clay, and is an important index value which could tell much about the mechanical properties of the clay (Sandven et al., 2014). It would be interesting to see if ϵ is affected by standardised index test.

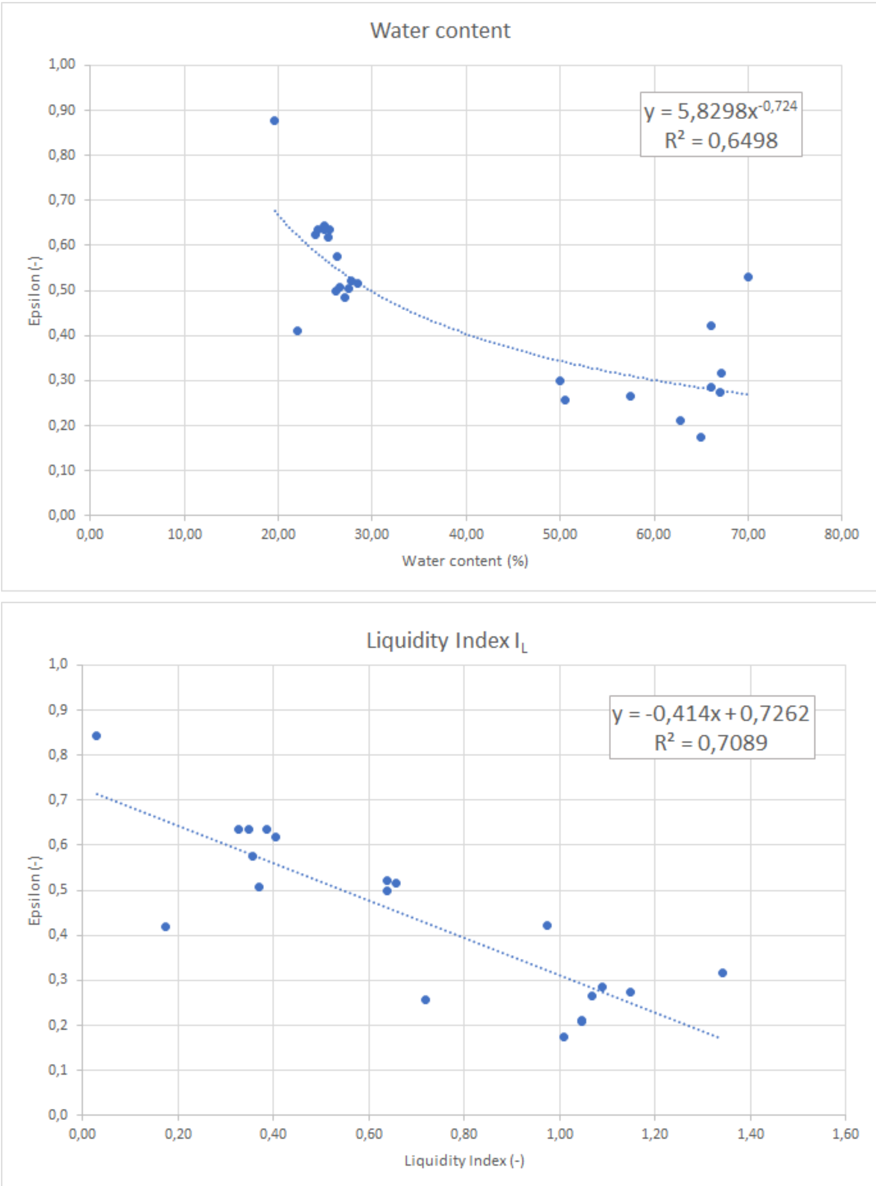


Figure 6.17: ϵ plotted with respect to water content (%) and liquidity index (-).

6.8 Self Weight Penetration

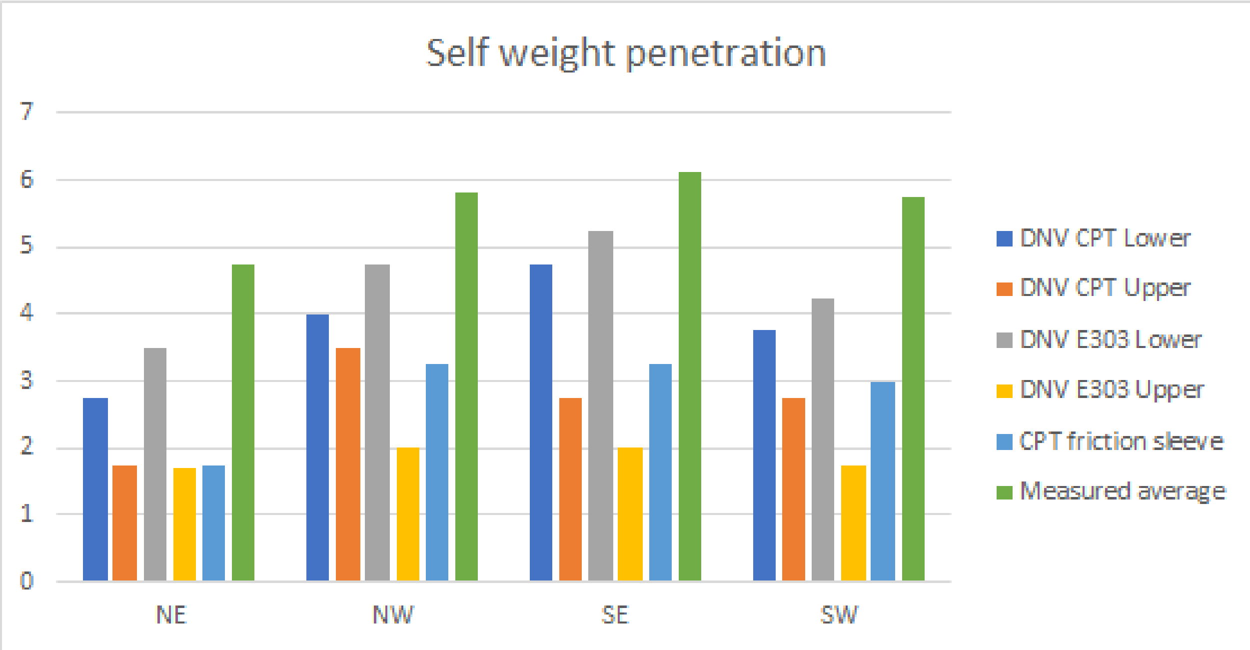


Figure 6.18: Graphical presentation of self weight penetration for the different methods.

Figure 6.18 illustrates the different values estimated for self weight penetration, and table 6.3 give the numerical self weight penetration estimated.

Cluster	DNV CPT		DNV RP E303		CPT sleeve friction
	Lower bound	Upper bound	Lower bound	Upper bound	
NE	2,75 (42%)	1,75 (63%)	3,50 (26%)	1,75 (64%)	1,75 (63%)
NW	4,00 (31%)	3,75 (40%)	4,75 (18%)	2,00 (66%)	3,25 (44%)
SE	4,75 (23%)	2,75 (55%)	5,25 (14%)	2,00 (67%)	3,25 (47%)
SW	3,75 (35%)	2,75 (52%)	4,25 (26%)	1,75 (70%)	3,00 (48%)

Table 6.3: Estimated self weight penetration for the different methods with lower- and upper bound. The values in parenthesise being the percentage different from the measured self weight penetration.

6.9 Impact of k_p

Which impact does the k_p-factor have on the total resistance? Figure 6.19 shows the impact of tip resistance with the use of values 0 , 0,4 and 1 for k_p. This calculation seems redundant since tip resistance over a small area in clay usually is quite small, and the choosing of value given by DNV (1992) is based on known theory. However, it seems useful to get the impression of the impact.

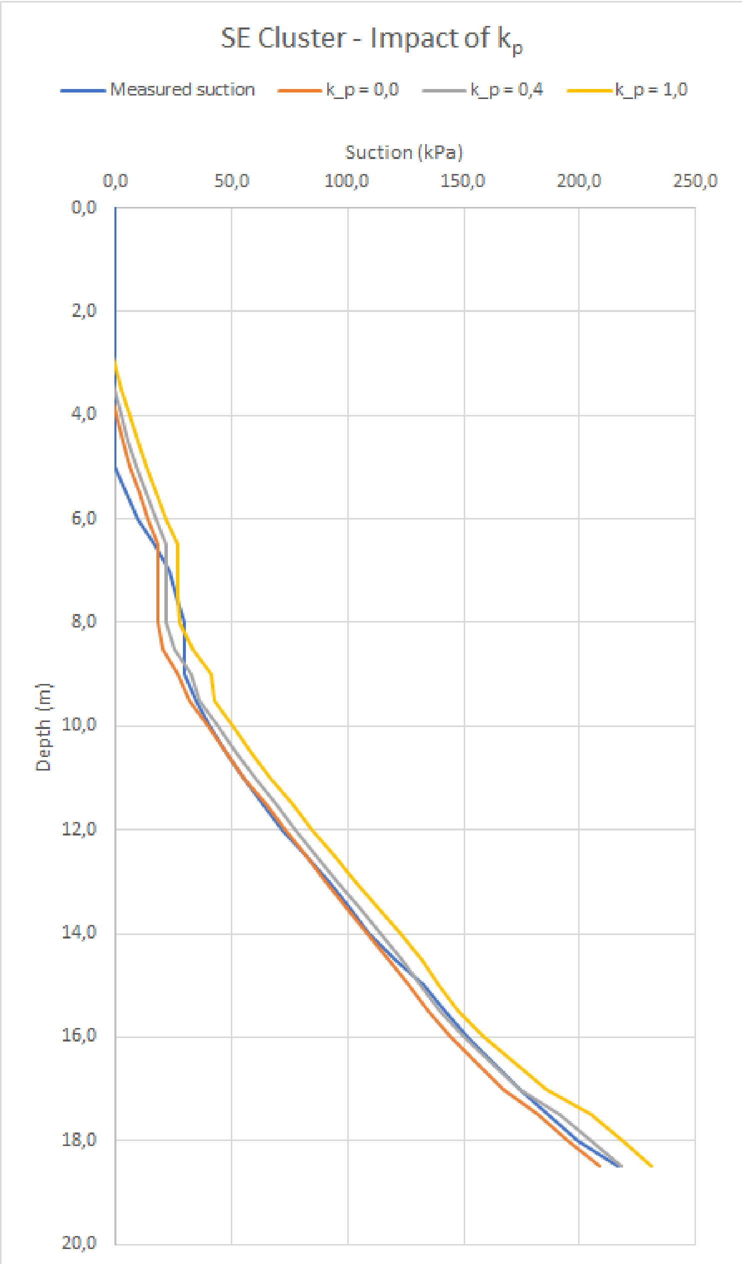


Figure 6.19: Illustration of the impact k_p-factor have on the total penetration resistance.

Chapter 7

Summary and Conclusions

7.1 Summary and Conclusions

This thesis has been looking at recommended methods to estimate suction needed to install suction anchors at depth which gives suitable bearing capacity. Necessary suction has also been estimated with a new proposed method, a method utilising measured sleeve friction from CPT. For a method to be useful it should estimate suction as close to the real suction measured, and yet be simple enough to keep investigation cost low.

All the methods utilised shows that remoulded shear strength plays a crucial role in estimating suction. The methods are based on two soil investigations techniques; values from laboratory test and CPT measurement.

The new proposed method indicates that using sleeve friction directly (not corrected for pore pressure) for Gjøa field gives estimations twice as high as what measured. When sleeve friction is reduced to fit the measured suction, it is found that a fixed value with layer and depth shows promising results. The reduction of sleeve friction shows potential correlation with liquidity index.

The main conclusion from this thesis is that estimating suction for suction anchors can't be conducted solely based on CPT measurement, and laboratory test must be conducted. With more research, this method could possible reduce the amount of costly laboratory test due to the use of index test.

7.2 Discussion

Measured suction is a valuable information for the geotechnical engineer, because it tells a lot about the soil. With the measured suction based on the equilibrium of forces one can back calculate the soil strength, which can be used to give even better estimates about the bearing capacity of the anchor.

The reason for choosing Gjøa is due to data accessibility, anchor length and limited influence of internal stiffeners. The anchor length gives more stable suction data, and will rule out "noise" in the beginning of penetration. The concern of soil behaviour due to stiffeners is excluded, which reduces the amount of uncertainty.

7.2.1 DNV Classification note No.30.4

The CPT method proposed by Det Norske Veritas in 1992 is a pragmatic method. The use of cone resistance to estimate tip resistance and skirt resistance based on fixed values gives as seen in section 6.2 large variance in estimated suction versus measured suction.

NE cluster is the only cluster where the estimated suction follows the same shape as the actual measured suction. The method also overestimates the suction for all clusters except of SW cluster. The method is quite conservative, which could be misleading. Considering the usage of steel skirted suction anchor at the time was quite limited and might be a reason for the "simple" approach. This method could be useful to some extent in the preliminary stage of planning. The limitation of this method is known by the DNV and discussed in DNV (1992).

7.2.2 DNV RP E303

The estimations done in this report based on the method proposed by DNV RP E303 is based on total stress. To use the total stress seems reasonable when penetrating under water. The results presented in section 6.3 shows that the measured suction is in between the lower and upper bound estimation. Again, the shape of the estimated suction varies from the measured suction, except for SE cluster which follow the shape quite well. The shape difference most possibly lays in the used soil strength. This problem is also discussed by Andersen et al. (2005).

The suction is also estimated with the local values for NE, NW, and SE cluster. Even with the local values the estimated suction is not a true copy of the measured suction. Reason for the deviation between the estimated suction and measured suction could again be due to the large distance between where the samples were collected and where the anchor were installed.

7.2.3 CPT sleeve friction f_s

The findings presented in section 6.4 with the use of one fixed value for ϵ shows promising results. The advantage with the use of sleeve friction is the continued measurement downwards. However, there are also several disadvantages and uncertainties with the use of sleeve friction.

As described in section 4.2, the sleeve friction measured could be assumed to be equal to the remoulded shear strength (when compared to UU compression triaxial test, in-situ vane test or fall cone probably gives better estimations of $s_{\mathrm{u,rem}}$). When studying figure 5.5 it's clear that the measured sleeve friction follows the upper limit of measured laboratory test. But why does the measured sleeve friction follow the upper limit? The reason might be:

- When penetrating with CPT the clay is exposed to large strain, and the clay passing the sleeve friction is assumed to be remoulded. But to which degree is the clay remoulded? E.g. when 16 to 18,5 meters of anchor passes a soil element in the higher region the clay could be assumed to be fully remoulded. But as close to the tip as the sleeve friction is measured it seems that the measured sleeve friction is measuring clay which hasn't been fully remoulded, hence the high the large sleeve friction.
- The used sleeve friction is f_s, and not the corrected sleeve friction f_t. As described in section 4.1 the pore pressure effect have showed to influence the measured sleeve friction with $\pm 20\%$. The size and geometry of the sleeve may vary from which supplier that delivers the CPT and should be documented to see the effect. This effect could be affecting the size of ϵ, however it seems logical to use f_s since u_3 is rarely measured.
- When penetrating with CPT the clay fails at the interface between the sleeve and the soil, which effect does this have on the measured sleeve friction values. Whereas for a vane shear test the measured momentum is measured by soil passing through soil, giving a possible better $s_{\mathrm{u,rem}}$ values.

Back calculated ϵ-values

The back calculated ϵ-values is presented in figure 6.13 and 6.14[1].

For the most part, the characteristics of ϵ is reasonable equal for all clusters. In the upper layer (Unit I, soft clay with high water content) the epsilon is in order 0,2-0,3 on average. It's important to notice that the low ϵ values are when the anchor penetrates by self weight and not suction.

After self weight penetration, the ϵ goes to almost 0,7 for NE, NW and SE cluster before it decrease down to 0,5. This peak in ϵ happens when suction is applied. One explanation for this peak behaviour could be due to set-up effect. If suction isn't applied immediately after self weight penetration has stopped, the soil might gain back some of its strength due to thixotropy.

When crane tension is released (70 tonnes to 30 tonnes) ϵ drops from 0,7 to the 0,5 on average. For NE and SE cluster the ϵ value is stable with depth throughout layer Unit IIA (medium to stiff clay).

The ϵ with depth should preferably be a fixed value with depth. Reason being if the ϵ increase or decrease linearly the back calculated friction isn't following the measured friction. Figure 7.1 illustrates this problem. Only NE and SE cluster results have fixed ϵ values, the reason lies in the uniform and stable CPT readings. NW cluster decrease linearly with depth after tension release, and SW increase linearly after 12 meters. Both NW and SW have great variation between the minimum and maximum measured sleeve friction which could cause this behaviour.
One other explanation to why the measured sleeve friction is seen to be higher than the back calculated friction, is the difference is shape and rate of speed.

There are two main differences when comparing shapes. Firstly, the CPT has a small circular shape, whereas suction anchors is assumed as a thin strip. Secondly the CPT is closed in the end, whereas suction anchors is open at the penetrating end. This difference is clear when estimating tip resistance, and also explained in section 3.2.1.

Rate of penetration during suction is 1,63 mm/s, the rate of speed of the CPT is 20 mm/s. The CPT penetrates 12 times faster than the anchor during suction. Lunne et al. (1997) refers to a tenfold in penetration speed increases cone resistance with 10-20% in stiff clays and 5-10% in

[1] Detailed list of ϵ with depth for each cluster is found in appendix E

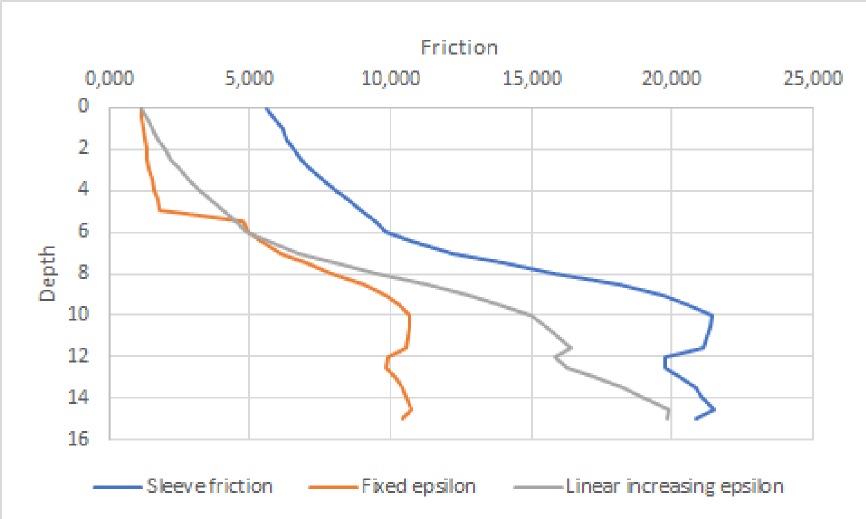


Figure 7.1: Example illustrating the problem with linearly increment of ϵ with depth.

soft clays. If the rate of penetration affects the sleeve friction is unknown, but could be a factor.

For this method to be practical there should be an approach to estimated or assume the magnitude of ϵ with depth or soil layering. As mentioned in section 6.7 water content is an important index test, and it will affect the shear strength, but will it affect ϵ? Water content and liquidity index is plotted versus ϵ at the respective depth. Figure 6.17 indicates a possibly correlation between water content and ϵ. The liquidity index versus ϵ shows an even better correlation. The liquidity seems like a reasonable index value to find a correlation for ϵ do the fact that liquidity index normalises the water content with plasticity index and liquidity limit. If the clay has an I_L value between 0 and 1 the clay will behave plastic, above 1 and the clay will act liquid. That ϵ becomes lower at higher I_L seems reasonable.

7.2.4 Remoulded shear strength

Mentioned several times throughout this report is that the remoulded shear strength is a key factor to estimate penetration resistance. When converting cone resistance with a fixed value as described in section 7.2.1 one has to assume that the relationship between cone resistance and sleeve friction is a fixed number. This isn't true for the most cases. The argument lies in friction ratio R_f, the relationship between sleeve friction and cone resistance. The friction ratio varies usually between 1 to 10 %.

Andersen et al. (2005) argues that the remoulded shear strength obtained from $\alpha \cdot s_u$ gives better estimates due to better defined s_u^C profile with depth. However, they also point out that sensitivity has some uncertainty to the parameter. If the sensitivity is measured on a disturbed sample, the sensitivity will be estimated to low. Figure 5.4 show the big variety in the measured sensitivity. This variation could be the possible reason for the result Høva got[2].

7.2.5 Self weight penetration

Great deal of uncertainties is related to estimating the final depth when only penetrating with self weight. The uncertainty could be divided into two groups. The one being soil, and the other being anchor weight.

Due to the high water content and possibility of disturbed samples in the top of unit I, giving a clear shear strength profile in the first few meters could be demanding. Crane tension release makes the anchor on average to self penetrate ≈ 2 meters at depth with higher shear strength. Thus, will uncertainties regarding weight have a great impact on the estimation of self weight penetration. Having good knowledge about weight of anchor and auxiliary weights should be reasonable and affordable uncertainty to exclude.

7.2.6 General uncertainties estimating suction

Throughout this report several uncertainties have discussed and the possible impact each uncertainty has to the final estimation.

- *Uncertainty regarding Gjøa suction anchor weight and geometry.* Does the weight given in Eide et al. (2009) include chain weight? If not, this will directly influence the estimation. Especially when trying to give the best estimated self weight penetration. Are there auxiliary weights? Since this isn't reported in the field report this question becomes impossible to answer without being present during the installation.
  There are few uncertainties regarding geometry, the only one is that there isn't detail drawing for each anchor length. The only uncertainty this will give is uncertainty when it comes to wall thickness.

- *Uncertainty regarding soil data.* Discussed in section 5.2.1, the distance from where the CPT were conducted and where the anchor were installed could contribute to uncertainty,

[2]See section 5.2.2 for Høva's research.

especially when trying to find correlation between ϵ and the index tests. Both the sleeve friction and index tests could be different from where it was measured to the place the anchor was installed.
Based on ϵ with depth it seems like the measured sleeve friction at NE and SE cluster is reliable. However, the linear increment of ϵ at NW and SW cluster indicates some uncertainties regarding used sleeve friction.

- *Uncertainty regarding measured suction and depth penetration.* During installation, the applied suction is logged alongside depth penetration. Which method used to log these values could vary, and could possible give some uncertainties.

7.3 Recommendations for Further Work

This thesis indicates that there is a possibility to utilise sleeve friction from CPT measurement to estimate suction. The correlation showed in section 6.7 should under no circumstances be used at this stage. Below shows a list of recommendations for further work based on experiences from this thesis which could contribute to a certainty of the method.

- **Recommendations regarding sleeve friction measurements:**
  - For this thesis the used sleeve friction have been the one directly measured, f_s. It would be interesting to investigate which effect the corrected sleeve friction f_t have on the back calculated ϵ.
  - The used CPT measurement should be as close as possibly to where the suction anchor is installed. This should reduce the linearly increasing or decreasing effect of ϵ. Since some soil layers have variety in pore pressure this could possible be affecting the estimated ϵ. It should be controlled if this effect has a big impact on the estimated suction.
- **Recommendations regarding soil properties:**
  - For Gjøa area figure 6.15 shows that the back calculated sleeve friction follows the lower bound of the measured $s_{\mathrm{u,rem}}$ from laboratory test. It would be interesting to check if this happens for other cases.
  - Using the proposed method in areas with different soil properties to check whether the liquidity index is a suitable index test to estimate the size of ϵ.

  - This thesis have only looked at installation in clay, however it would be interesting to check if the proposed method is suitable i coarser materials like sand.

- **Anchor geometry:**

  - The anchors estimated in this thesis stands alone which excludes rigidity forces which could accrue in ITS during suction. The anchor length has also been an advantage to rule out noise in the top layer during installation. I. It should be clarified if it's suitable method for shorter anchor and ITS.

Appendix A

CPT measurement at Gjøa

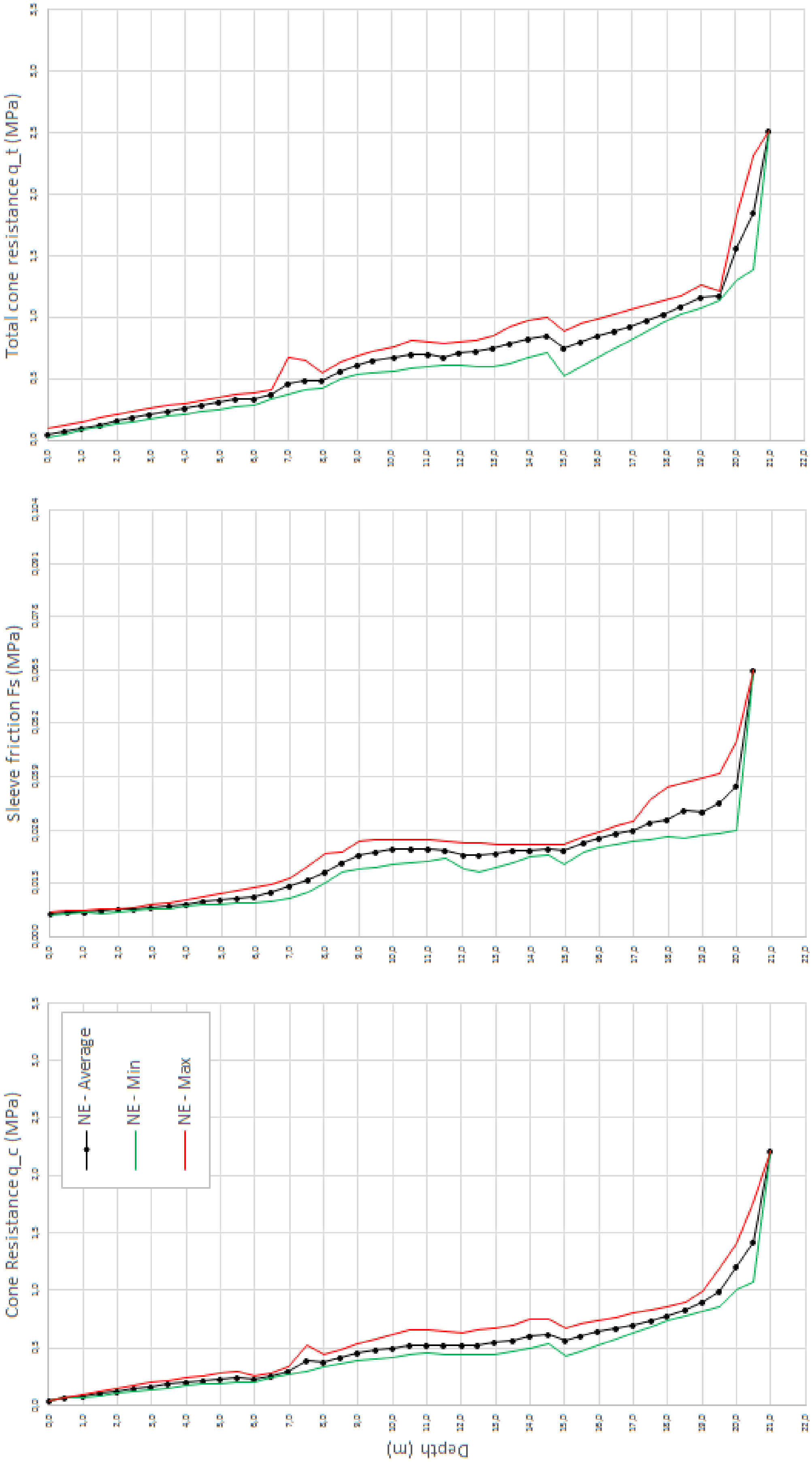


Figure A.1: CPT measurement at NE cluster.

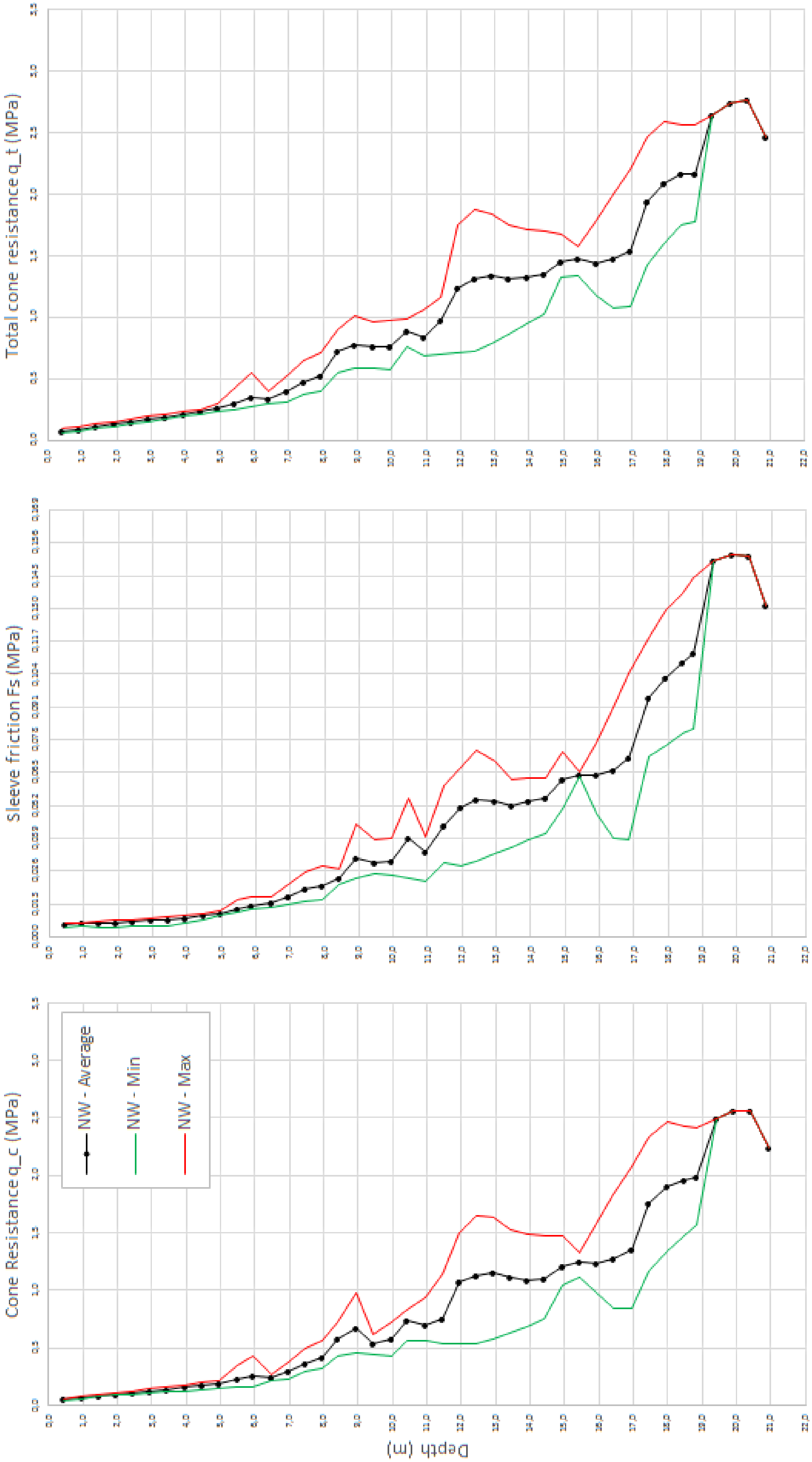


Figure A.2: CPT measurement at NW cluster.

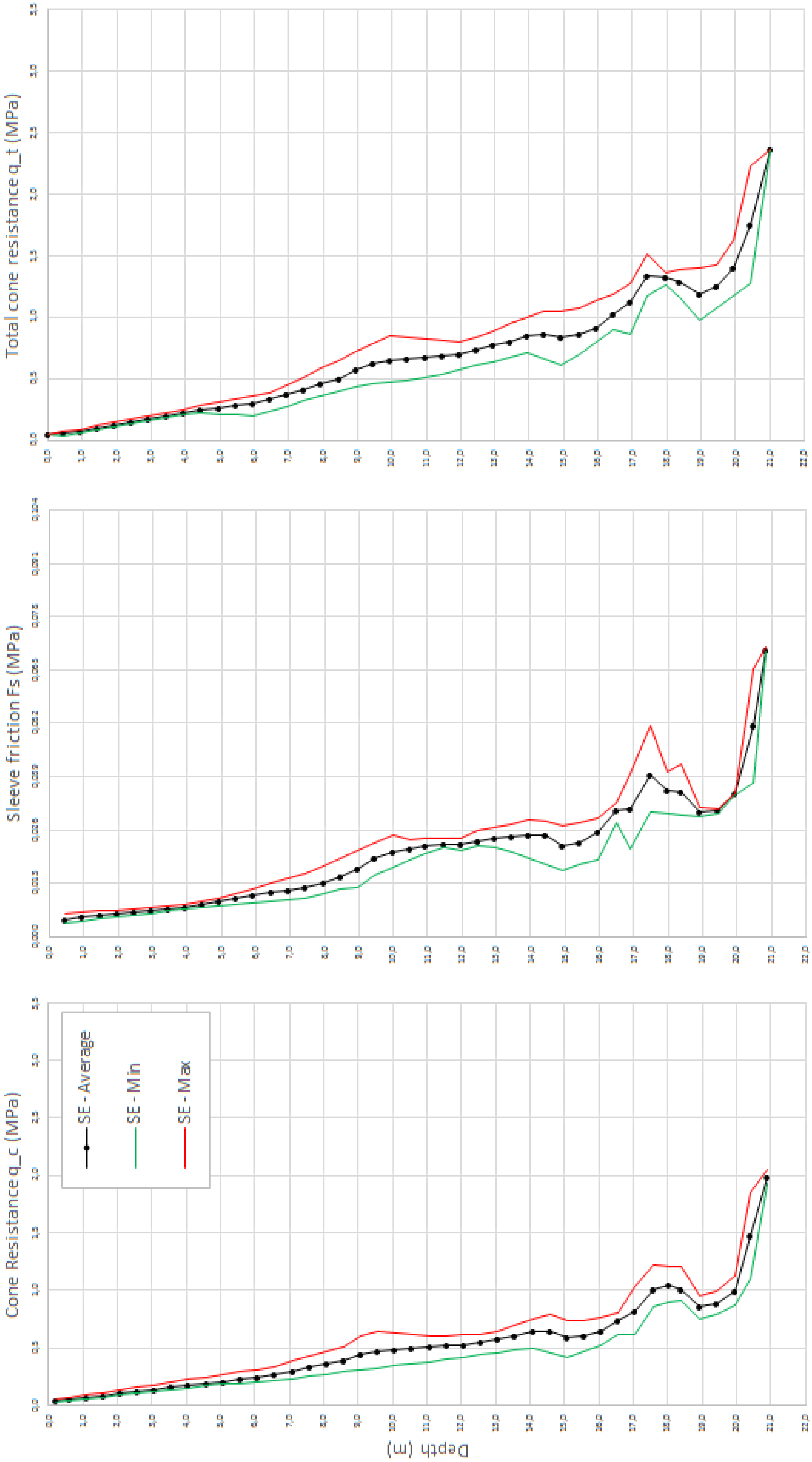


Figure A.3: CPT measurement at SE cluster.

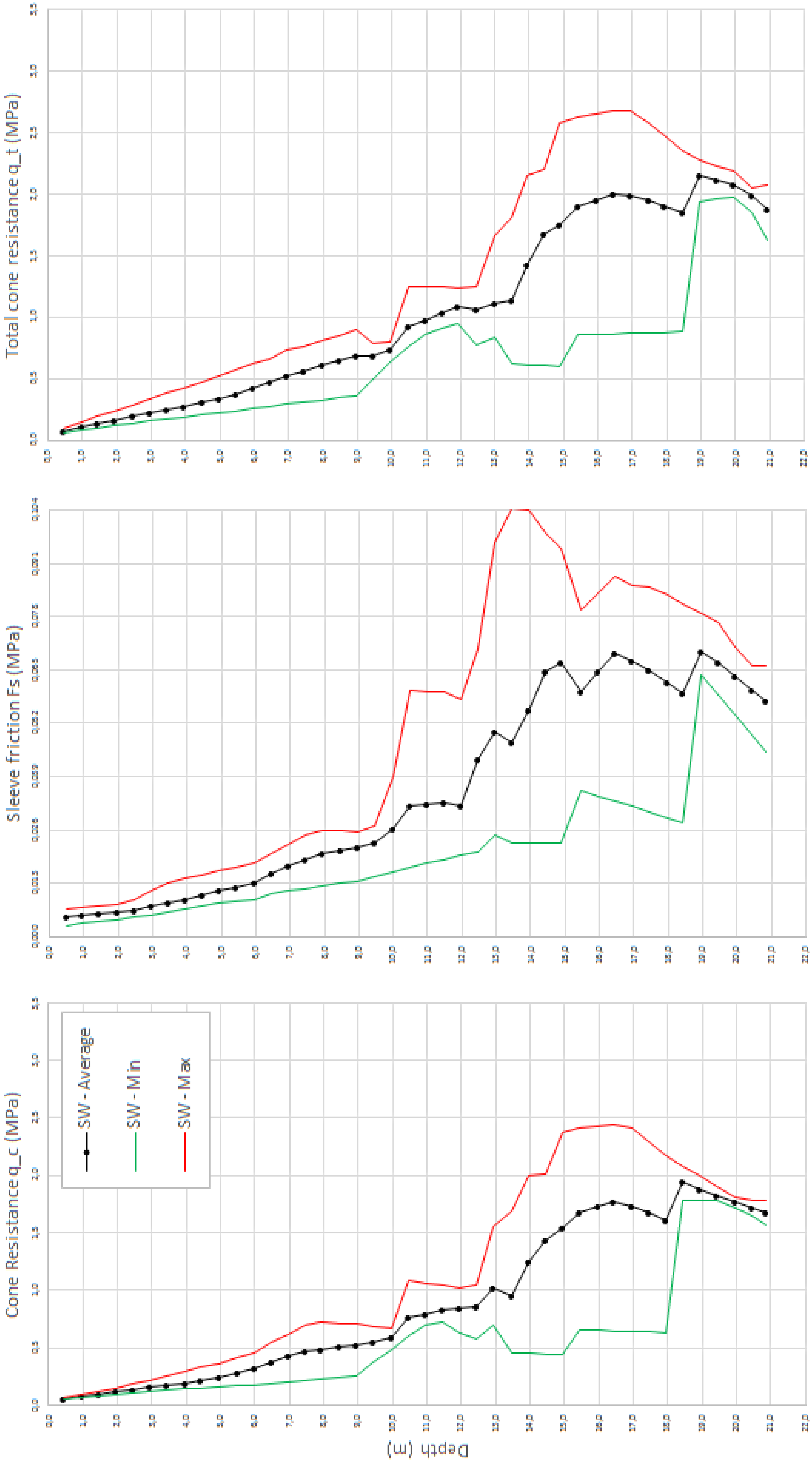
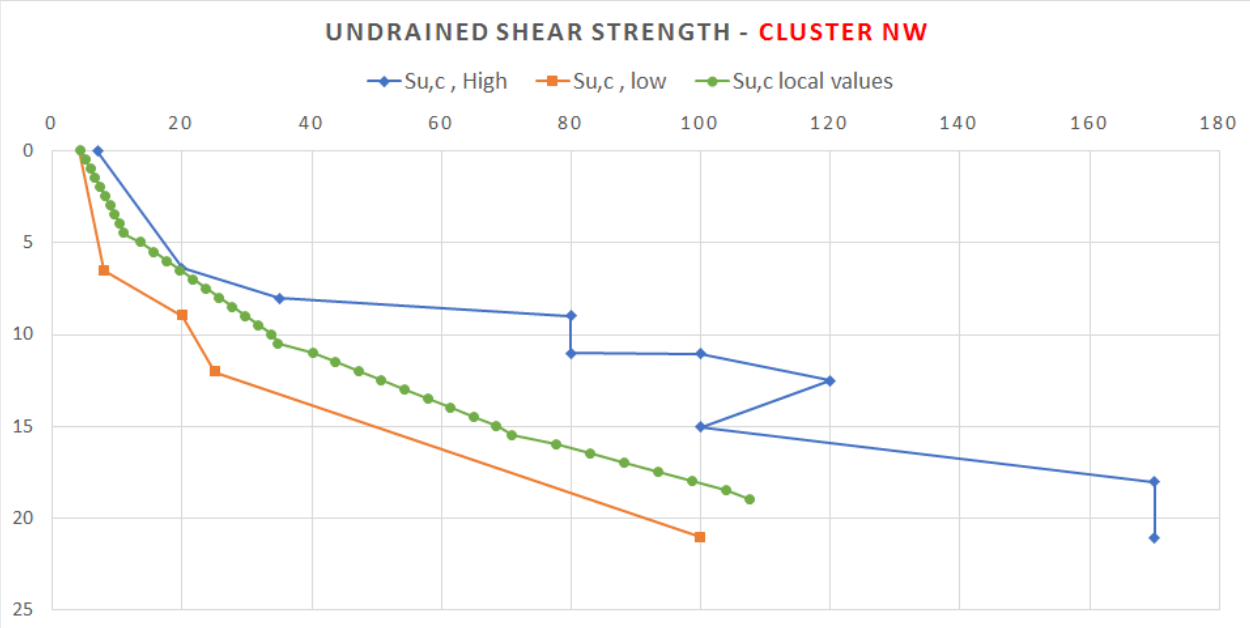


Figure A.4: CPT measurement at SW cluster.

Appendix B

Recommended shear strength s_u^c profile at Gjøa



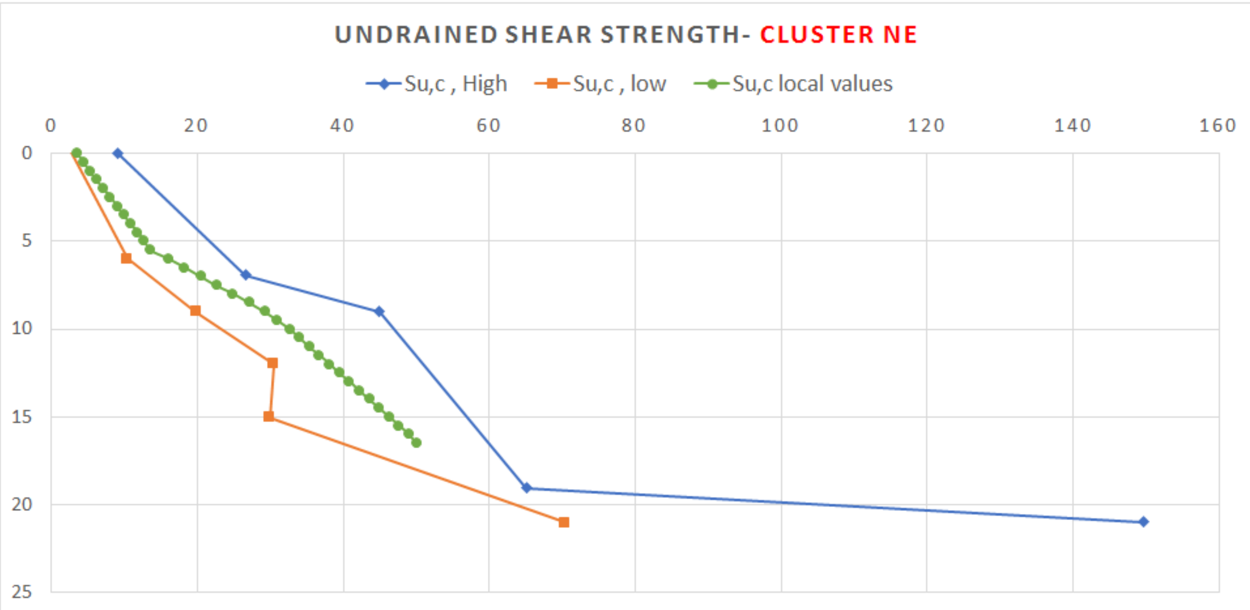
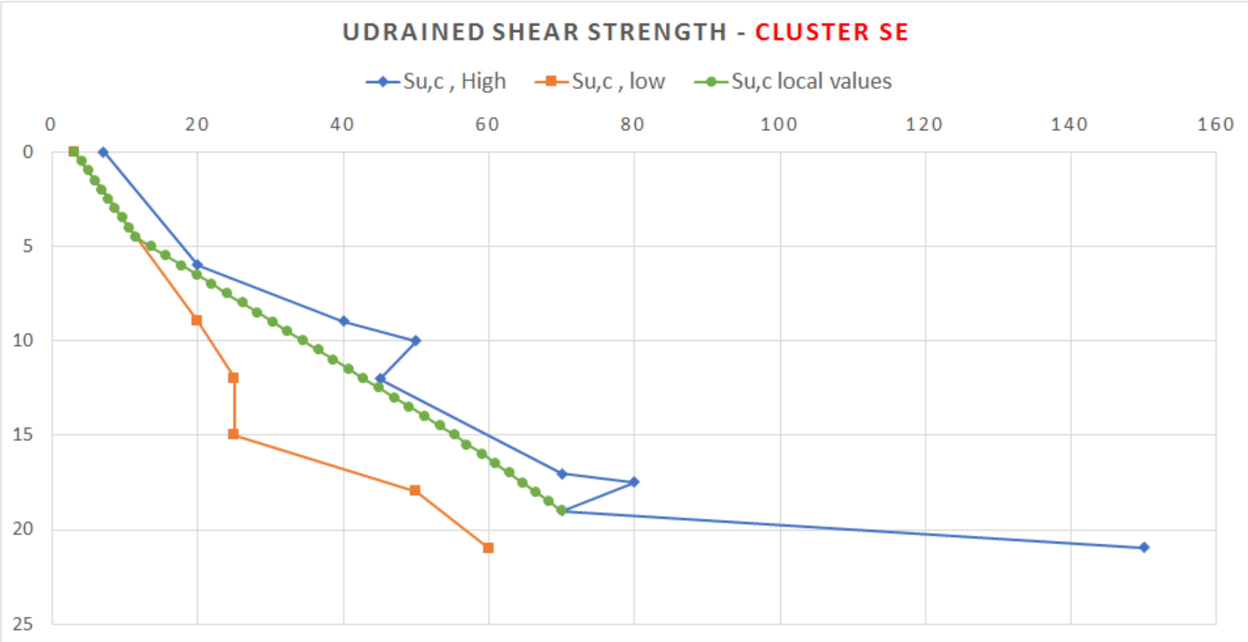


Figure B.1: Recommended undrained shear strength for NW and NE cluster (NGI, 2007).



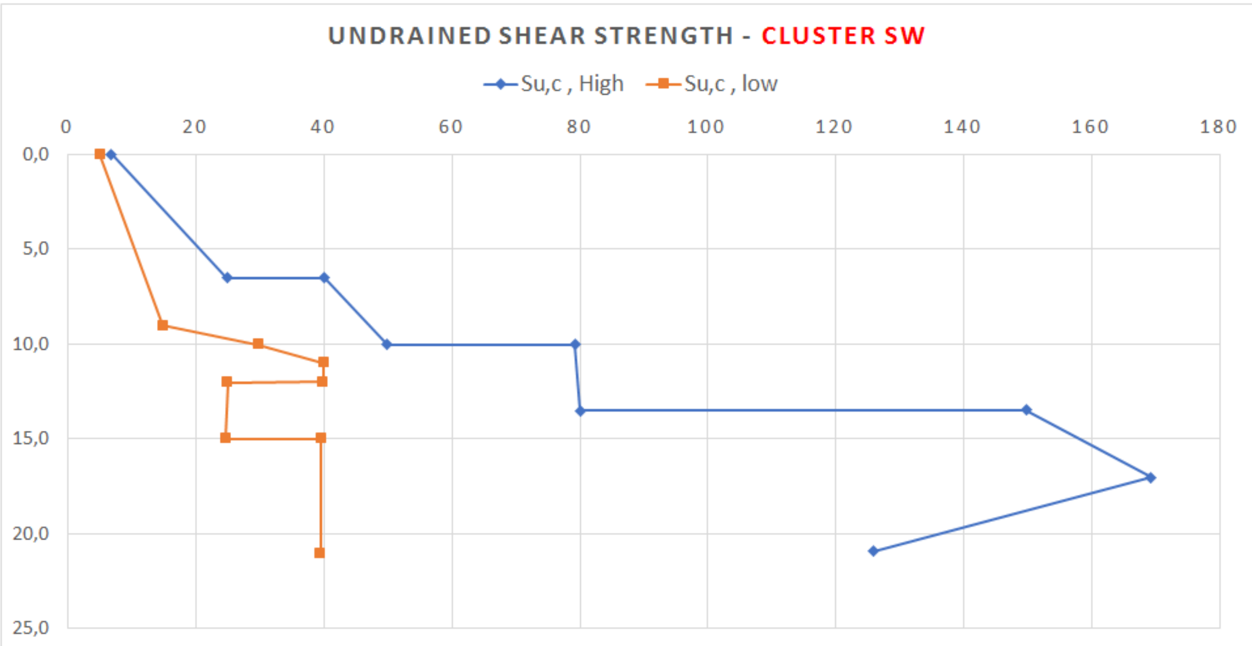
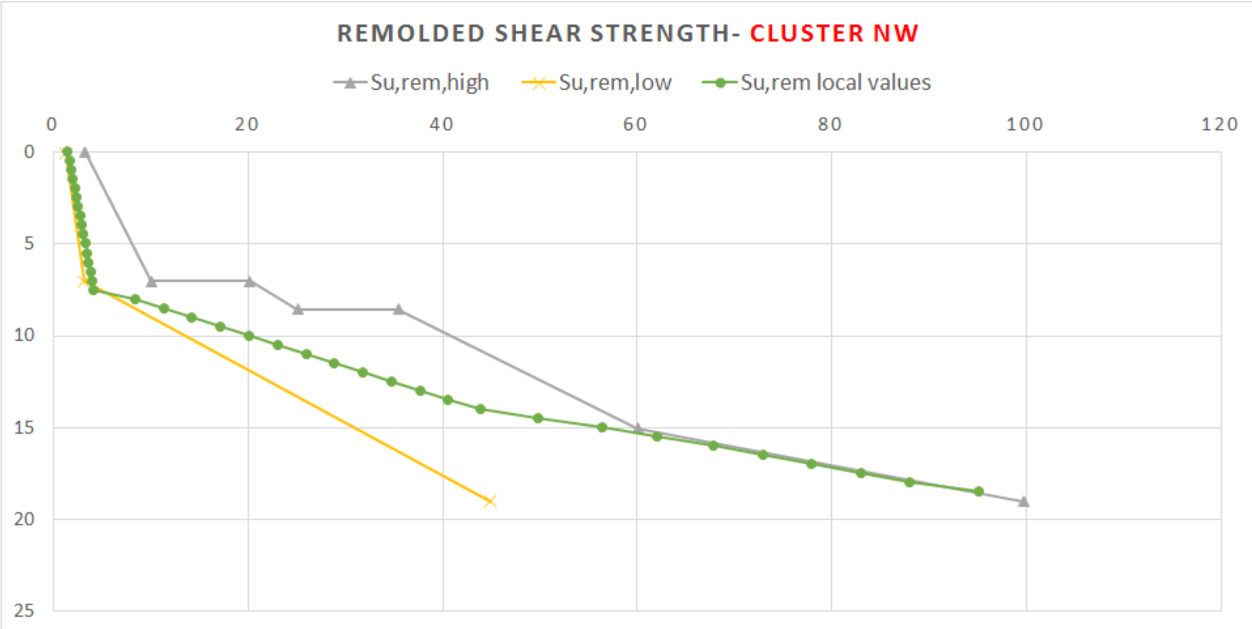


Figure B.2: Recommended undrained shear strength for SE and SW cluster (NGI, 2007).

Appendix C

Recommended remoulded shear strength $s_{u,rem}$ profile at Gjøa



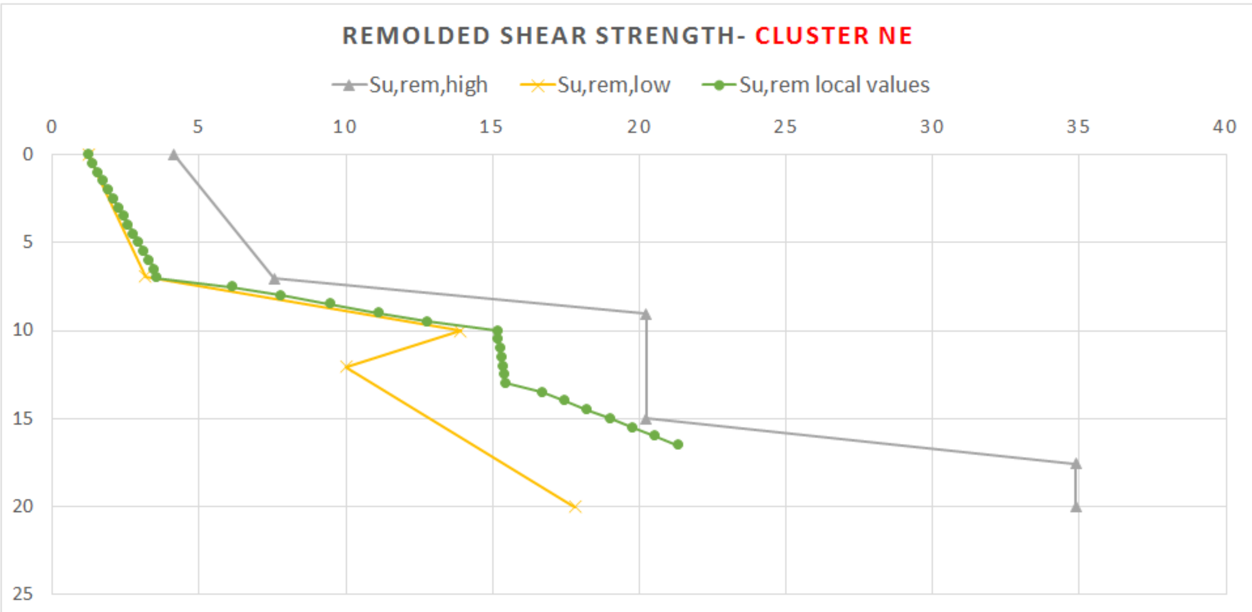


Figure C.1: Recommended remoulded shear strength for NW and NE cluster (NGI, 2007).

Figure C.2: Recommended remoulded shear strength for SE and SW cluster (NGI, 2007).

Appendix D

Suction Measurement at Gjøa

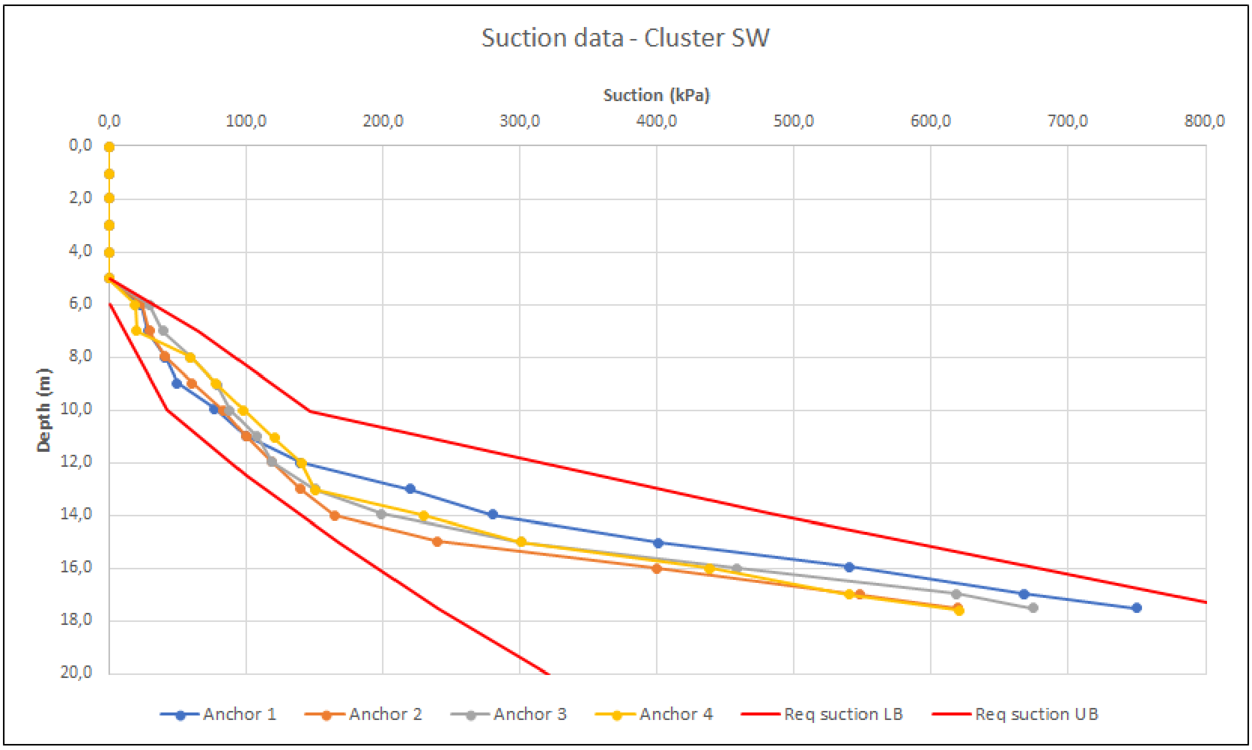


Figure D.1: Suction data from anchor cluster SW at Gjøa.

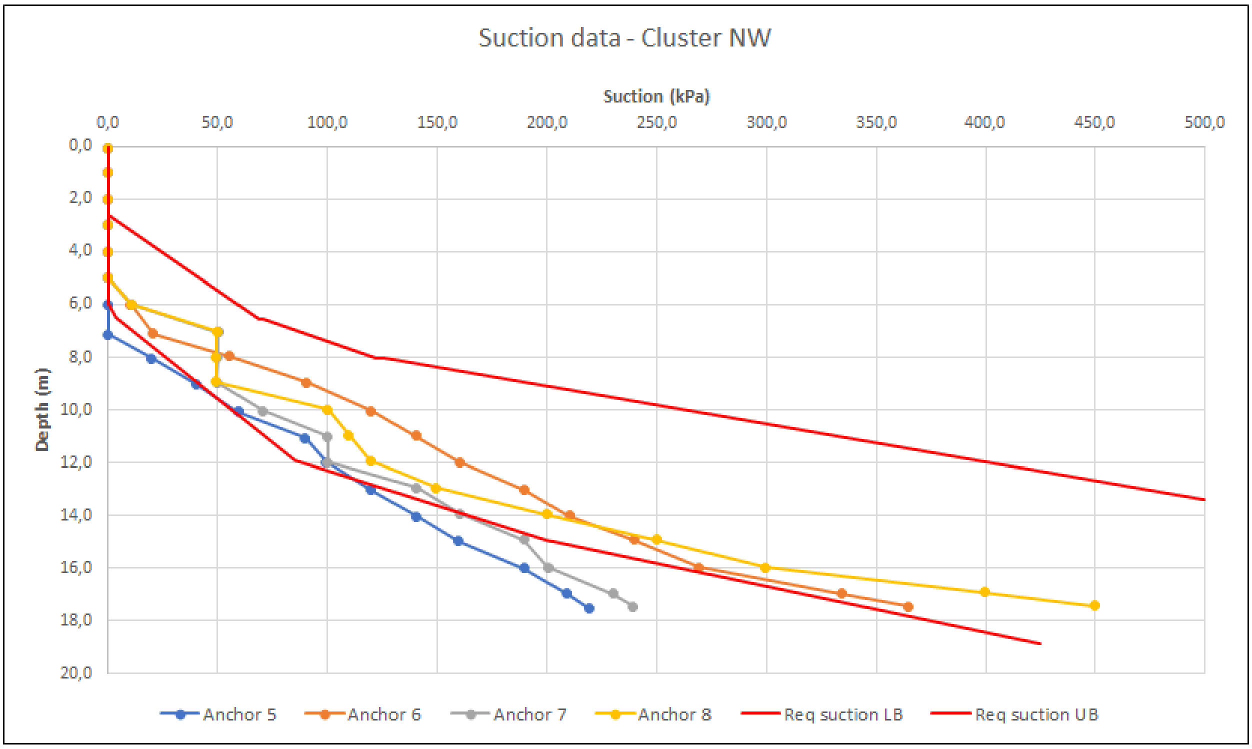


Figure D.2: Suction data from anchor cluster NW at Gjøa.

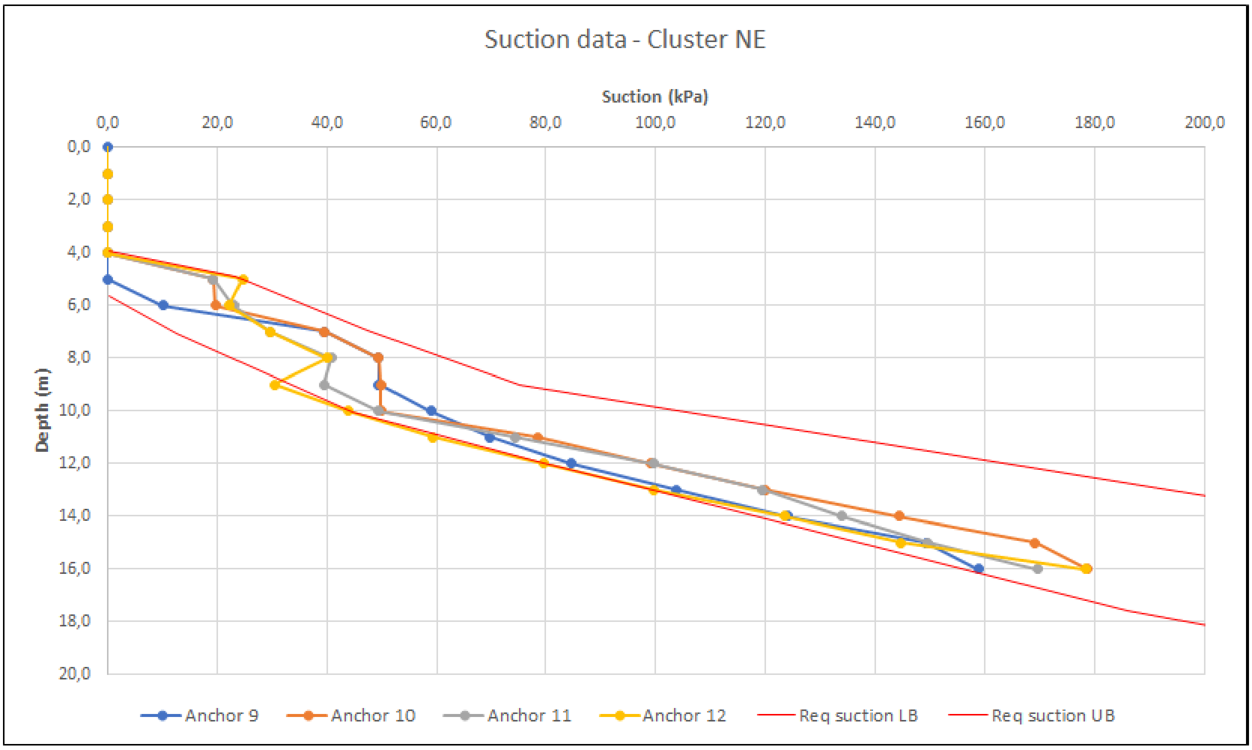


Figure D.3: Suction data from anchor cluster NE at Gjøa.

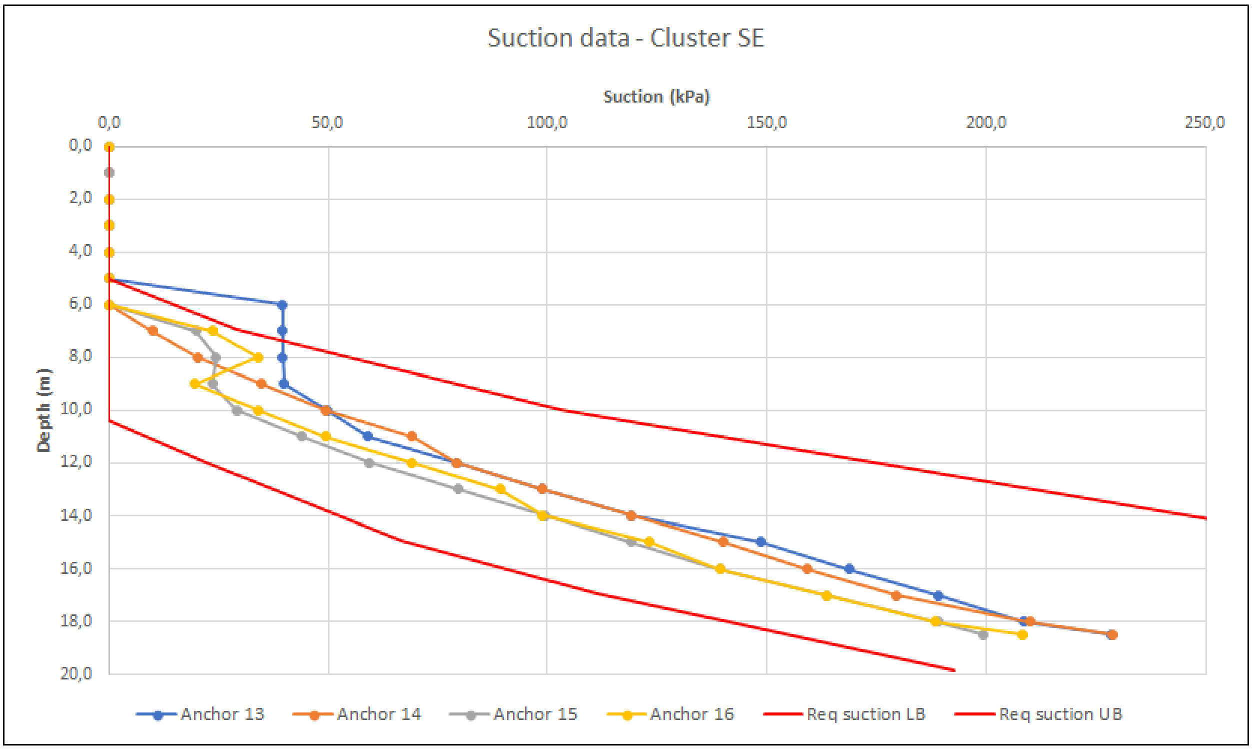


Figure D.4: Suction data from anchor cluster SE at Gjøa.

Appendix E

Back calculated epsilon values with depth

	NE	NW	SE	SW
Depth	ϵ	ϵ	ϵ	ϵ
(m)	(-)	(-)	(-)	(-)
0,0	0,12	0,22	0,32	0,28
0,5	0,12	0,22	0,32	0,28
1,0	0,19	0,27	0,37	0,33
1,5	0,20	0,28	0,37	0,34
2,0	0,21	0,29	0,37	0,34
2,5	0,21	0,28	0,36	0,34
3,0	0,21	0,28	0,35	0,33
3,5	0,21	0,27	0,34	0,31
4,0	0,21	0,27	0,33	0,30
4,5	0,18	0,26	0,32	0,29
5,0	0,15	0,25	0,31	0,27
5,5	0,21	0,29	0,35	0,42
6,0	0,26	0,31	0,38	0,53
6,5	0,41	0,42	0,43	0,49
7,0	0,72	0,68	0,67	0,63
7,5	0,68	0,65	0,63	0,64
8,0	0,65	0,61	0,59	0,64
8,5	0,58	0,58	0,53	0,64
9,0	0,52	0,54	0,48	0,62
9,5	0,52	0,59	0,49	0,63
10,0	0,50	0,60	0,48	0,62
10,5	0,48	0,58	0,48	0,61
11,0	0,47	0,56	0,47	0,59
11,5	0,47	0,51	0,47	0,57
12,0	0,47	0,46	0,47	0,57
12,5	0,48	0,45	0,48	0,57
13,0	0,49	0,44	0,48	0,56
13,5	0,50	0,44	0,48	0,58
14,0	0,51	0,43	0,48	0,59
14,5	0,52	0,43	0,49	0,63
15,0	0,54	0,42	0,50	0,66
15,5	0,52	0,41	0,51	0,74
16,0	0,51	0,40	0,50	0,80
16,5		0,41	0,50	0,84
17,0		0,42	0,50	0,88
17,5		0,41	0,49	0,91
18,0			0,49	
18,5			0,50	

Table E.1: Back calculated ϵ-values with depth for each cluster.

Bibliography

Andersen, K., Murff, J., Randolph, M., Clukey, E., Erbrich, C., Jostad, H., Hansen, B., Aubeny, C., Sharma, P., and Supachawarote, C. (2005). Suction anchors for deepwater applications. In *International Symposium on Frontiers in Offshore Geotechnics (ISFOG)*, pages 3–30. Taylor & Francis.

Andersen, K. H. and Jostad, H. P. (1999). Foundation design of skirted foundations and anchors in clay. *Offshore Technology Conference*, (OTC 10824).

Andersen, K. H. and Jostad, H. P. (2004). Shear strength along inside of suction anchor skirt wall in clay. *Offshore Technology Conference*, (OTC 16844).

API (2005). Design and analysis of stationkeeping systems for floating structures. Recommended practice API RP-2SK, American Petroleum Institute, Washington D.C.

Christensen, S. and Eiksund, G. (2009). Geotechnical design of gjøa semi sun anchors rev.08.

Colliat, J.-L. and Dendani, H. (2004). Suction anchors for deepwater moorings at nkossa and girassol in 200 and 1,400 m of water.

DNV (1992). *Foundations, Classification notes No. 30.4.* Det Norske Veritas Classification AS (DNV), Høvik, Norway.

DNV (2005). Geotechnical design and installation of suction anchors in clay. Recommended practice DNV-RP-E303, Det Norske Veritas, Høvik.

Ehlers, C. J., Young, A. G., and hwa Chen, J. (2004). Technology assessment of deepwater anchor. *Offshore Technology Conference*, (OTC 16840).

Eide, A., Huslid, C., and Håland, G. (2009). Gjøa suction anchor installation geotechnical field report. MBM-NKG-RA-00046 - statoil internal, Corporate StatoilHydro.

Eni, N. Utbyggingsløsning. http://www.eninorge.com/no/Feltutbygging/Goliat/Utbyggingslosning/ [Accessed: 25.05.2017].

Helgesen, O. K. (2010). *Statoil har overlevert Gjøa.* https://www.tu.no/artikler/statoil-har-overlevert-gjoa/252439 [Accessed: 01.03.2017].

Huang, J., Cao, J., Audibert, J. M., et al. (2003). Geotechnical design of suction caisson in clay. In *The Thirteenth International Offshore and Polar Engineering Conference*. International Society of Offshore and Polar Engineers.

Høva, E. M. (2008). Skjørtpenetrasjonsmotstand på snøhvit, kristin og Åsgard. PTT-NKG-RA 00095 - draft report, Statoil.

Larsen, V. M., Moan, T., and Odland, J. (2011). *Havromteknologi, et hav av muligheter*, volume kapittel 10 Olje- og gassutvikling til havs.

Lunne, T., Robertson, P., and Powell, J. (1997). *Cone Penetration Testing in geotechnical practice*. CRC Press.

NGI (2007). Gjøa soil investigation 2006.

Randolph, M. and Gourvenec, S. (2011). *Offshore Geotechnical engineering*. CRC Press.

Sandven, R., Senneset, K., Emdal, A., Nordal, S., Janbu, N., Grande, L., and Amundsen, H. A. (2014). *Geotechnics Field and Laboratory Investigations*. Geotechnical Division, Norwegian University of Science and Technology, Trondheim.

Smith-Solbakken, M. Alexander l. kielland-ulykken. https://snl.no/Alexander_L._Kielland-ulykken [Accessed: 24.05.2017].

Technology, O. Delta house field development, gulf of mexico, united states of america. http://www.offshore-technology.com/projects/delta-house-field-gulf-mexico/delta-house-field-gulf-mexico2.html [Accessed: 24.05.2017].

Tjelta, T. I. (2001). Suction piles: Their position and application today. *The International Society of Offshore and Polar Engineers*, 11.

Vinnem, J. E. Macondo-ulykken. https://snl.no/Macondo-ulykken [Accessed: 24.05.2017].