Elise Bergsagel

Investigation of high temperature energy storage in Norwegian quick clay

Master's thesis in Geotechnology

Master’s thesis in Civil and Environmental Engineering
Supervisor: Rao Martand Singh
Co-supervisor: Randi Kalskin Ramstad
June 2023

Master’s thesis

NTNU
Norwegian University of Science and Technology
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Abstract

The Paris Agreement binds 175 countries, including Norway, in limiting global warming below 2 degrees. To achieve this, sustainable development is crucial. Geothermal structures like thermal energy storage offer eco-friendly solutions for heating and cooling buildings. The waste heat produced during summer in Norway can be utilized during winter. Understanding how temperature affects sensitive soils, such as quick clay, is vital for safe construction near zero-emission buildings. Ground Source Heat Pumps (GSHP) are widely used, but their potential in soft clays remains untapped. Our research explores the temperature-clay relationship, revolutionizing geothermal energy storage for a greener future.

Using a modified oedometer cell with a copper coil the study investigates the effects of temperature on soil behaviour. Oedometer tests were conducted on clay specimens at 20℃ and 70℃. Calibration tests ensured accurate temperature and loading measurements. The results indicate variations in preconsolidation pressure, pore pressure, strengths and coefficient of consolidation between normal and modified oedometer tests. The thesis discusses the experimental setup, research design, and results from index testing.

The study examines the effects of thermal and mechanical loading on clay behaviour. The heating phase reveals increased strains and decreased effective stress in most clays, except for one test showing a slight stress increase. Fluctuating pore pressures during heating suggest the involvement of physico-chemical forces. Furthermore, the study compares the influence of temperature on natural clays from Canada, Sweden and Norway. Norwegian clay exhibits higher preconsolidation pressure after thermal and mechanical loading compared to Canadian and Swedish clay. The correlation between liquid limit and preconsolidation pressure is inconsistent. Pore pressures and oedometer modulus exhibit varying responses to temperature, indicating change in soil behaviour and properties. The coefficient of consolidation remains stable at room temperature but fluctuates at 70℃, highlighting the influence of elevated temperature on consolidation behaviour.

The decrease in strongly bound water with increasing temperature can result in negative pore pressure, and the plasticity of clay affects void ratio changes in response to temperature. While preconsolidation pressure generally decreases with temperature, exceptions exist. The study emphasizes the need for further experiments to explore additional factors such as strain rate. Overall, these findings contribute to a better understanding of the complex behaviour of clays under thermal and mechanical loading conditions.

Keywords: CRS test, oedometer test, thermal response of soils, quick clay, temperature effects, laboratory, saturated clay

Sammendrag

175 land, inkludert Norge, har forpliktet seg til å begrense det globale utslippet av klimagasser til under 2 grader gjennom Parisavtalen. Bærekraftig utvikling er avgjørende for å oppnå dette målet. Geotermiske konstruksjoner, som for eksempel termisk energilagring, sørger for miljøvennlige løsninger for oppvarming og kjøling av bygninger, industribygg, idrettsanlegg og lignende. I Norge kan overskuddsvarmen som produseres om sommeren, utnyttes om vinteren. Det er viktig å forstå hvordan temperaturen påvirker sensitive leirer, som for eksempel kvikkleire, for å kunne bygge i nærheten av nullutslippsbygninger eller -nabolag. Forskningen gjort i dette studiet utforsker sammenhengen mellom temperatur og leire, og på denne måten kan bidra til utvikling innen geotermisk energilagring for en mer bærekraftig fremtid.

Ved hjelp av en modifisert ødometercelle med et oppvarmet kveilet kobberrør blir leiren tilført varme opptil 70℃. Resultatene gitt i studien indikerer variasjon i prekonsolideringstrykket, poretrykket, den endimensjonale modulusen og konsolideringskoeffisienten mellom leire varmet opp til 20℃ og leire varmet opp til 70℃.

Studien undersøker hvordan effektene av termisk og mekanisk belastning påvirker oppførselen til leire. Resultatene fra oppvarmingsfasen indikerer at for de fleste leiretyper øker deformasjonen mens den effektive vertikale spenningen reduseres, bortsett fra ett forsøk hvor spenningen øker noe. Poretrykket varierer under oppvarming, noe som indikerer at tiltrekkende- og/eller frastøtende krefter er involvert i prosessen.

Studien sammenligner også innflytelsen av temperatur på naturlig leire fra Canada og Sverige med norsk leire. Det observeres at prekonsolideringstrykket etter termisk og mekanisk belastning er høyere for norsk leire sammenlignet med kanadisk og svensk leire. Flytegrensen har ikke en konsekvent sammenheng med prekonsolideringsegenskaper. Poretrykket og ødometermodulusen viser varierende respons på temperatur, noe som indikerer endringer i leirens oppførsel og egenskaper. Konsolideringskoeffisienten forblir stabil ved romtemperatur, men varierer ved 70℃, noe som tyder på at konsolideringsoppførselen påvirkes av høy temperatur. Reduksjonen av sterkt bundet vann med økende temperatur kan føre til negativt poretrykk, og leirens plastisitet påvirker endringer i porøsiteten på grunn av temperaturendring.

Nøkkelbegreper: CRS test, ødometer test, termisk respons til leire, kvikkleire, temperaturpåvirkning, laboratorium, mettet leire

Preface

I am filled with immense gratitude as I write this preface, reflecting on the journey that led me to this moment. There are countless individuals who have played a significant role in shaping my path and supporting me along the way. In particular, I would like to extend my deepest appreciation to my family, especially my mom and dad.

To my dear parents, your unwavering love, encouragement, and belief in me have been the foundation upon which I have built my dreams. Your constant presence, sending me happy thoughts during both triumphs and challenges, has been a source of strength. Your sacrifices and endless support have shaped the person I am today, and I am forever grateful for the lessons you have taught me.

I would also like to express my heartfelt thanks to my roommates, who have become more than just friends. Your companionship, understanding, and willingness to lend an ear during late-night study sessions have made this journey far more enjoyable. The laughter, camaraderie, and shared experiences have made our time together truly memorable.

Furthermore, I would like to express my deep appreciation to the workshop team. Your expertise, guidance, and patience have played a vital role in transforming my ideas into reality. I want to extend a special thank you to Bent Lervik for his meticulous application of the copper coil and the TX150 to the existing cell. I am also grateful to Tage Westrum and Torbjørn Nerland for their invaluable assistance whenever issues arose with the oedometer cell.

I would also like to acknowledge and express my gratitude to Department Engineer Lee Thomas Champe for his exceptional insights and advice on executing the experiments. Your mentorship has been a constant source of inspiration, and I feel privileged to have had the opportunity to collaborate with you. Your thoughtful contributions have significantly 0enhanced the quality and depth of my research. And, Espen Andersen Torsæter for collecting the samples used in this thesis at Flotten site.

Last but certainly not least, I would like to acknowledge and thank my supervisors, Associate Prof. Randi Kalskin Ramstad and Prof. Rao Martand Singh. Your trust in my abilities and dedication to my success has led me to create this master's.

To each and every one who has contributed to my journey from primary school, to where I am now and whether mentioned here or not, please know that your impact has been significant. The achievements I celebrate today are a testament to the collective effort and support I have received. From the bottom of my heart, thank you.

Elise Bergsagel

Trondheim, June 2023

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List of Symbols and Abbreviations

Roman letters

c	Constant dependent on tip angle	n	Porosity
c_u	Shear strength	p'_c	Preconsolidation pressure
c_{ur}	Remoulded shear strength	p'_{cr}	Referential preconsolidation pressure
c_v	Coefficient of consolidation	S_t	Sensitivity
e	Void ratio	S_u	Undisturbed sensitivity
F	Force	S_{ur}	Remoulded sensitivity
g	Gravity acceleration	t	Time
H_0	Initial height	T	Temperature
i	Average cone penetration	T_r	Reference temperature
I_p	Plasticity index	u_0	Applied backpressure
M	Oedometer modulus	u_b	Pore pressure at base
m	Mass	v	Specific volume
$m'/\,m$	Modulus number	V_p	Volume of voids
m_1	Mass of container and moist test specimen	V_s	Volume of solids
m_2	Mass of container and dried test specimen	w	Water content
m_c	Mass of container	w_L	Liquid limit
m_d	Mass of dried test specimen	w_P	Plasticity limit
m_w	Mass of water		

Greek letters

α	Viscoplastic parameter	σ	Stress
γ	Unit weight	σ'	Effective stress
γ_s	Unit weight of solid particles	σ'_0	Initial effective stress
Δ	Change	σ'_m	Mean effective stress
δ	Vertical deformation	σ'_p	Preconsolidation pressure
ε	Strain	σ'_{pr}	Referential preconsolidation pressure
ε_v	Vertical strain	σ_v	Vertical applied stress
$\dot{\varepsilon}_v$	Rate of vertical strain	σ'_v	Vertical effective applied stress
κ	Recompression index		
λ	Normal compression index		

Abbreviations

CRS	Constant Rate of Strain	NC	Normally consoldation
GDS	Global Digital Systems	NCL	Normal Compression Line
GHP	Geothermal Heat Pump	URL	Unloading-Reloading Line
GSHP	Ground Source Heat Pump	TX150	Grant Instrument Advanced Optima TX150
vs.	Versus		

1 Introduction

1.1 Motivation

The Paris Agreement is a legally binding international treaty where its goal is to limit global warming to below 2 degrees. Norway is one of 175 countries that signed the treaty that took effect 4th of November 2016 (Martin, 2016; *The Paris Agreement | UNFCCC*, n.d.). The expected heat- and cooling needs of Norway by 2030 is respectively 52.7 TWh for heating and 2 TWh for cooling according to Ramstad (2011). To reach the goal of the Paris Agreement the sustainable development is increasingly important. Geothermal structures, such as thermal energy storage, is an environmentally friendly energy provider to heat and cool buildings. Understanding the thermomechanical properties of sensitive soil, especially quick clay, is crucial for the buildings nearby due to the expanding request of zero emission buildings and neighbourhoods. It is not only a local problem in Trondheim, nor in Norway, it is a global problem.

The Ground Source Heat Pump (GSHP), also called Geothermal Heat Pump (GHP), are commonly solutions for thermal energy storage. The GSHP are the most common application of geothermal heat in the world according to Weber et al., (2017). In a GSHP system there will be running heated fluid in the heat exchanger. There are numerously of experimental studies that have investigated the influence on natural and artificial clays when exposed to temperatures up to 100˚C, majority in relation to nuclear waste disposal, e.g. Abuel-Naga et al., (2007), Bentil & Zhou (2022), Burghinoli et al., (2000), Demars & Charles (1982), Di Donna & Laloui (2015), Graham et al., (2001), Hueckel & Baldi (1990), Kuntiwattanakul et al., (1995) and Sultan et al., (2002). The results shows that a change in temperature has an affection on the pore water pressure, shear strength, stiffness as well as it might give a change in volume. A modest change on sensitive clays, e.g. quick clay, mechanical properties may lead to drastic change in the material behaviour. A change in material behaviour for quick clay can at worst have a fatal outcome.

To further investigate the temperature effect on clay, so that the GSHP can be stored in soft clays and not only in bed rock, can expand the usage of GSHP. An expanded usage of GSHP can decrease the costs for the society, not only economic but environmentally as well. The negative outcome of an increase in GSHP is if the energy needs to be connected to the telecommunications network which needs to be expanded and more transformers needs to be built. The expansion of transformers is needed do the fact that the GSHP and other geothermos produces current and power over the legally amount given in Quote 1 from NVE.

Quote 1

> *Det fremgår av energiloven §3-1 og energilovforskriften §3-1 at produksjonsanlegg med spenning på mer enn 1000 V vekselstrøm / 1500 V likestrøm trenger konsesjon etter energiloven. Slike anlegg kan ikke bygges, eies eller drives uten konsesjon etter energiloven.*
> (*Lover og regler - NVE*, 2022)
>
> *It appears from the Energy Act §3-1 and the Energy Regulations §3-1 that production facilities with voltage of more than 1000 V alternating current / 1500 V direct current require a license under the Energy Act. Such facilities cannot be built, owned or operated without a license under the Energy Act.*

1.2 Aim and Objectives

The aim of this thesis is to study the isothermal affection on sensitive clay, more specific quick clay. A modified oedometer apparatus developed for thermal affect is utilized to study the thermomechanical properties of the sensitive clay. The temperature-controlled oedometer is being affected by temperatures up to 70˚C. From the oedometer test the aim is to study the volumetric strain, excess pore pressure and the stress-strain curve during a CRS test under isothermal loading. To achieve the goal of this master's thesis, the following actions need to be taken:

- Develop and construct a temperature-modified oedometer cell
- Carry out drained heating consolidation tests for investigation of thermally induced volumetric deformation and pore pressure
- Carry out drained loading for a constant temperature to investigate the temperature effect on volumetric deformation ε_V, preconsolidation stress p'_c, modulus number m' and coefficient of consolidation c_v

1.3 Thesis chapter structure

1. Introduction: gives an introduction on why the study is relevant for the society
2. Quick clay: gives background information about quick clay through definition of quick clay and its deposits
3. Thermal behaviour of natural clays: a literature study on relevant theories, and previous studies related to the study
4. Experimental background: gives founding knowledge for the experiment by explaining the key concepts and theories with the experiment
5. Experimental methodology: presents how the experiments are executed and justifies the choices made during the process.
6. Oedometer test: presents, analyse and reflects over the research done with the thermal loading with constant load and from the isothermal loading with mechanical loading
7. Conclusions: summarizes the main findings of the study. Discuss the implications of the study, its contributions to the field, and recommendations for future research

Part Ⅰ

Theory

2 Quick clay

Quick clay is found in large quantities in Norway, especially in areas around Trondheim and Oslo. Quick clay is a sensitive type of clay and is described in the following chapter, (*Kva Er Kvikkleire Og Kvikkleireskred? - NVE*, 2022).

2.1 Formation

The formation of quick clay originating from when the glacier covered the land areas that we know today. The kilometres thick glaciers covering the land areas crushed mountains and the bedrock. This resulted in microscopic grains and clay particles. As the glacier melted the rivers transported the particles into the ocean. As the clay particles got in contact with the ocean, the clay particles flocculated with each other while the salt ions in the ocean connected the particles together and created a card-house structure. With the glaciers' retreat, land areas that previously had been submerged below sea level and subsequently uplifted due to isostatic rebound, and are referred to as marine clay, the structure of the marine clay is shown in Figure 2-1 (a). In Norway, marine clay is found up to 160 meters above today's water level, and its distribution is demarked by the marine level.

The uplifted marine clay was exposed to freshwater flow, e.g., precipitation and groundwater flow. When fresh water flowed through the marine clay, the salt ions and salt particles vanished. This left an unstable structure, that is called quick clay, shown in Figure 2-1 (b). The quick clay has pore water in the interstices between the clay particles. If the contact between the particles becomes insufficient due to subjected load or shear deformation, the quick clay structure will collapse. The clay particles have no strength, and the abundant pore water confers a liquid consistency to the clay.

(a) Marine clay with saline porewater: stable structure, large attractive forces between particles edges and planes. There are small repulsive forces between particle planes

(b) Quick clay: unstable structure, there are small attractive forces between particle edges and planes. There are large repulsive forces between particle planes

*Figure 2-1: Card-house structure of marine- and quick clay, (*Kva Er Kvikkleire Og Kvikkleireskred? - NVE*, 2022)*

2.2 Geotechnical Definition

A quick clay is defined by a remoulded shear strength, $c_{ur} < 0.33\ kPa$ according to ISO 17892-6. Some other characteristics, (*Geotechnics Field and Laboratory Investigations*, 2017):

- Water content is greater than the liquid limit, $w > w_L$
- A high sensitivity, above 30
- Salinity below 0.5%

The Atterberg limit is a characteristic of remoulded soil in which the original structure has been disrupted. Consequently, the in-situ water content of the soil can exceed the liquid limit, even though the soil in-situ is not in a fully liquid state. The liquid limit represents the threshold at which the soil transitions from a plastic to liquid state. The determination of the liquid limit can be accomplished through either the Casagrande method or by the fall cone method.

Sensitivity, S_t, is calculated as the ratio of the initial shear strength, c_u, to the remoulded shear strength, c_{ur}, of a given soil. The classification of soils is established based on their sensitivity, as detailed in Table 2-1. The sensitivity is determined through the falling cone test, which involved the examination of both undisturbed and remoulded soil samples. In the case of a soil exhibiting low remoulded strength but relatively higher undisturbed shear strength, the soil will be very sensitive. Additionally, the classification of shear strength will be correspondingly high.

Table 2-1: Classification of soil strength, *(*Geotechnics Field and Laboratory Investigations*, 2017)*

Classification soil type	**Classification shear strength**	**S_t (-)**
Low sensitive	Low	< 8
Medium sensitive	Medium	8-30
Very sensitive	High	> 30

The salinity test is a pertinent analytical technique when assessing marine clays. In particular, a clay exhibiting low salt content in its pore water is indicative of sensitive or quick clay. The determination of salinity is accomplished through measuring the conductivity of the pore water. A salinity value below 0.5% suggest the potential presence of quick clay.

2.3 Geological Deposits

Quick clay was formed during the ice age, and in Scandinavia it can be found below the marine level. Quick clay is primarily found in Scandinavia, North of America, and Russia, (*NGI - What Is Quick Clay?*, 2023). For Scandinavia, there are found quick clay primarily in Norway and Sweden. In Norway, quick clay is found in areas around Trondheim and Oslo, shown with red rectangles in Figure 2-2. While, for Sweden, there are possibilities for quick clay at the same latitude as Oslo on 59˚, as well as small possibilities at the eastern side of the country, which is marked as yellow and blue areas in Figure 2-3. In Canada quick clay occur commonly in the valleys of St. Lawrence and Ottawa Rivers, shown in the shaded areas on Figure 2-4. The glacier that covered Scandinavia, called Scandinavian Ice Sheet, retreated for 9500 years ago, while the glacier that covered Canada, called Laurentide Ice Sheet, retreated 6000 years ago, (Mangerud, 2022). The Swedish and Norwegian clay is predicted to have more of the same properties than the east Canadian clay, due to the timing of the glaciers' retreat.

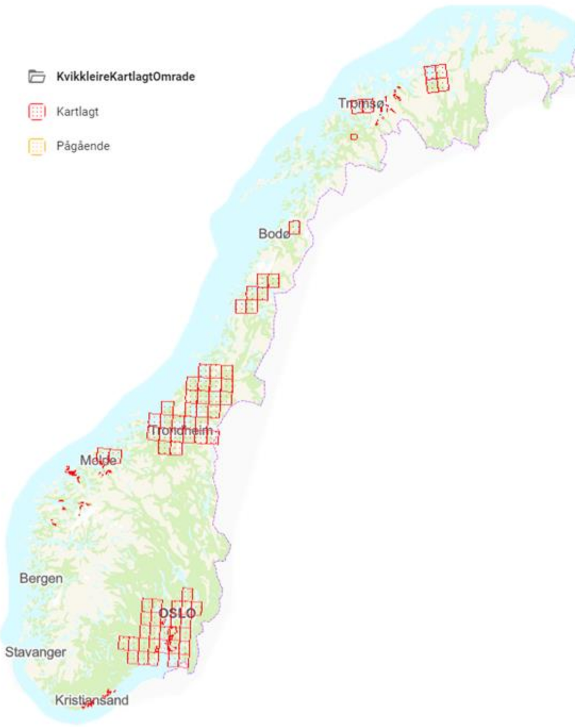


*Figure 2-2: Charted quick clay deposits in Norway, (*NVE Temakart*, n.d.)*

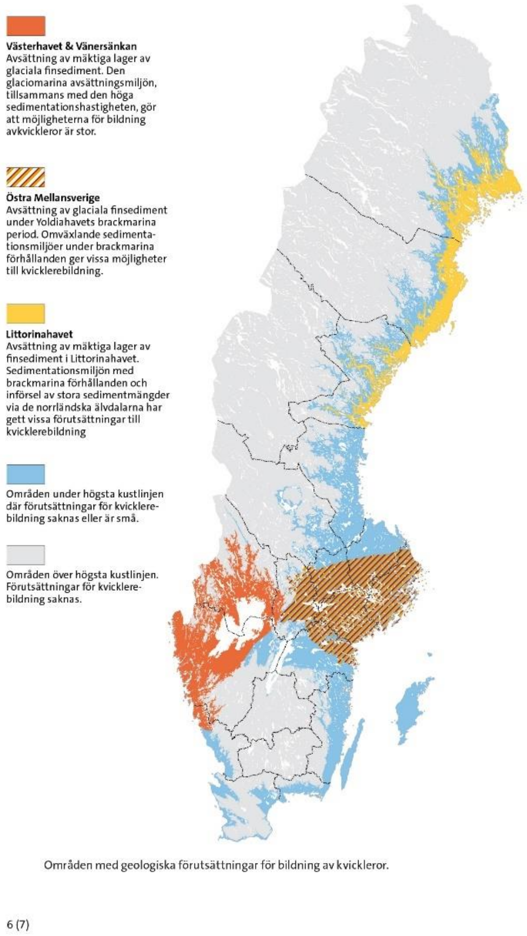


*Figure 2-3: Possibility for quick clay deposits in Sweden, (*Kvickleror*, n.d.)*

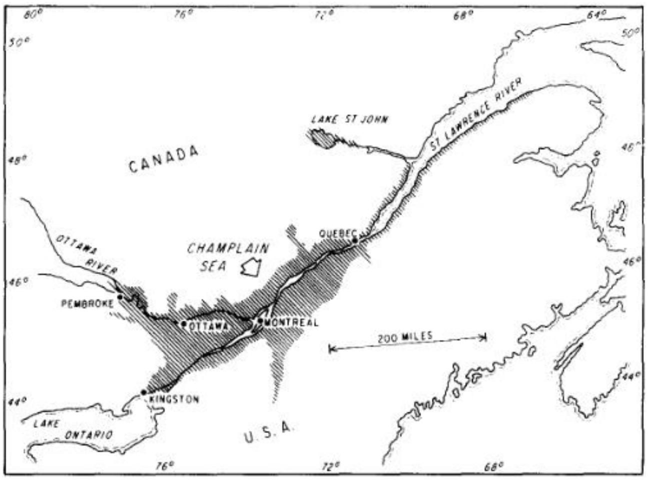


Figure 2-4: Quick clay deposit of eastern Canada, (Crawford, 1968)

2.4 Summary

This chapter presents structural formation of quick clay, the geotechnical definition and its deposits. Quick clay, a sensitive type of clay, was abundant in Norway, particularly around Trondheim and Oslo. It was formed during the ice age when glaciers crushed mountains and bedrock, creating microscopic clay particles. As glaciers retreated, the marine clay was exposed to freshwater, losing its stability and becoming quick clay. Geotechnically, quick clay is characterized by low remoulded shear strength, water content greater than the liquid limit, high sensitivity, and low salinity. Deposits of quick clay are found in Scandinavia, North America and Russia.

3 Thermal Behaviour of Natural Clays

3.1 Background

There have been raised questions regarding the storage of nuclear waste in the seabed. The storage of nuclear waste continued to generate heat from the "spent" fuel, leading to a focus on heat storage. Soil temperatures up to 100˚C can cause heating, shrinkage, pressure changes, and possible alterations in soil strength and stiffness (Graham et al., 2001). As the Paris Agreement's two-degree goal by 2050 is a way of adapting the impact on climate change. As a result, there has been raised question how to efficient save and use energy. One of the ideas is geothermal structures, such as geothermos or energy piles to store energy. According to Graham et al., (2001), changes in temperature can lead to alterations in the structure, thickness, and viscosity of adsorbed water. As a result, noticeable variations in compressibility and strength can be observed. Both heating and cooling have the potential to induce these effects. These changes can have significant implications for both serviceability limit state and ultimate limit state. Consequently, understanding the thermo-mechanical properties and behaviour of clays is crucial in various engineering applications, including geothermal structures, radioactive waste disposal, and buried high-voltage cables.

The thermo-mechanical behaviour of soft soils is a complex phenomenon that is closely related to the responses of the microstructure. When saturated clay is subjected to temperature change, the soil particles and the pore fluid are the two components that respond to the change in temperature. Campanella and Mitchell (1968) conducted a thorough investigation of the effects of temperature on the volume change and pore water pressure of clay. They found that an increase in temperature would activate rearrangement of clay particles, inducing expansion of the clay particles and causing the expansion of water in the clay pores. The difference in the thermal expansion coefficient of water and the soil solids generates excess pore water pressure under thermal loading. The thermal expansion coefficient of water is approximately 15 times larger than that of solids (Sultan et al., 2002). The magnitude of pore water pressure change depends on several factors such as hydraulic conductivity, the rate of thermal loading, and the drainage conditions (Li, 2019).

According to Abuel-Naga et al., (2007), the volumetric strain induced by thermal effects can be decamped into two components. One of these components is a reversible expansion strain, while other is an irreversible contraction strain. The reversible component can further be decomposed into two parts. The first component is attributed to the thermal expansion of clay minerals, while the second component is related to the temperature effects on the interparticle physico-chemical forces that exhibit reversible behaviour.

In order to investigate the potential change in properties of natural clays due to temperature change, several modified standard tests have been conducted, with the aid of temperature-controlled equipment. The test used is primarily oedometer tests (e.g., Abuel-Naga et al., (2007; Bentil & Zhou, 2022; Hu et al., 2022)), Constant Rate of Strain (CRS) tests (e.g., (Boudali et al., 1994; Chen et al., 2023; Hu et al., 2022; Moritz, 1995; Tidfors & Sällfors, 1989)) and triaxial test (e.g., Abuel-Naga et al., (2007; Burghignoli et al., 2000; Demars & Charles, 1982; Graham et al., 2001)) as well as it is possible to use a direct shear test.

3.2 Response of Clays to Temperature Change

Robinet et al. (1996) highlighted the difficulty in distinguishing between the effects of thermal and mechanical loading on soil behaviour. Thus, it is necessary to decouple these mechanisms by maintaining constant the mechanical loading during temperature change. The thermo-mechanical properties of soils are influenced by various factors such as the overconsolidation ratio, the plasticity index, the type of specimen (normally- or overconsolidated), the loading condition (mechanical or thermal). Moreover, these factors interact with each other, making it challenging to develop general equations that can be applied to all types of soil.

3.2.1 Mechanical Loading Under Constant Temperature

The objective of mechanical loading under constant temperature is to elicit specific mechanical and hydraulic responses, such as normal compression and recompression indexes (λ and κ), drained and undrained shear strength (c_u) and stiffness. These properties are important indicators of the material's ability to withstand external forces and deformation.

The normal compression and recompression indexes, denoted λ and κ respectively, are fundamental parameters in the study of the mechanical behaviour of soils subjected to isotropic compression. The indexes are functions of effective stress and specific volume of the given material, as shown in Figure 3-1. λ, also known as the flexibility coefficient, describes the compression in the normal consolidation (NC) range after yielding along the normal compression line (NCL).

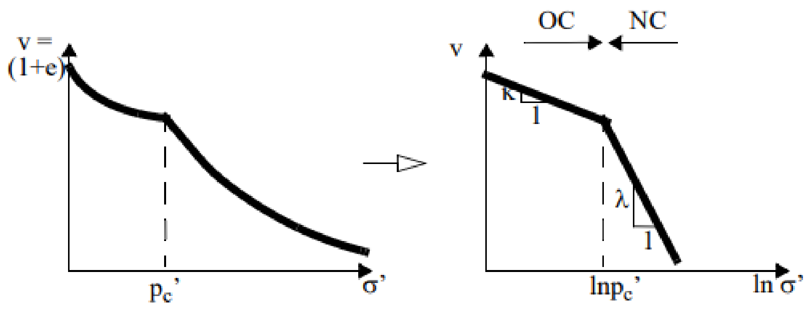


Figure 3-1: The normal compression and recompression indexes, λ and κ, (Nordal, 2020).

On the other hand, κ, also called swelling index, represents the elastic compression behaviour of soils during unloading-reloading cycles on the unloading-reloading line (URL), ignoring visco-plastic strain. While some studies (Campanella & Mitchell, 1968; Chen et al., 2023; Robinet et al., 1996; Tidfors & Sällfors, 1989) have reported that λ and κ are independent of temperature, others have shown that these indexes exhibit temperature sensitivity. Plum &

Esrig (1969), Eriksson (1989), Burghignoli et al. (2000) and Graham et al. (2001) have reported that λ and κ are influenced by temperature in varying degrees, see Figure 3-2 and Figure 3-3. Plum & Esrig (1969) found a slight expansion of the material, still the volume change is too small to be shown in Figure 3-2. On the other hand, Eriksson (1989) and Graham et al. (2001) (see Figure 3-3) have observed an increase in κ with rising temperature, while Eriksson (1989) has found no significant change in λ with temperature.

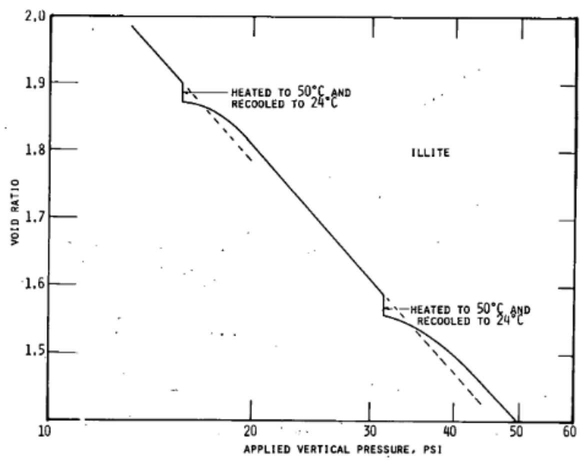


Figure 3-2: Applied vertical pressure vs. void ratio (Plum & Esrig, 1969)

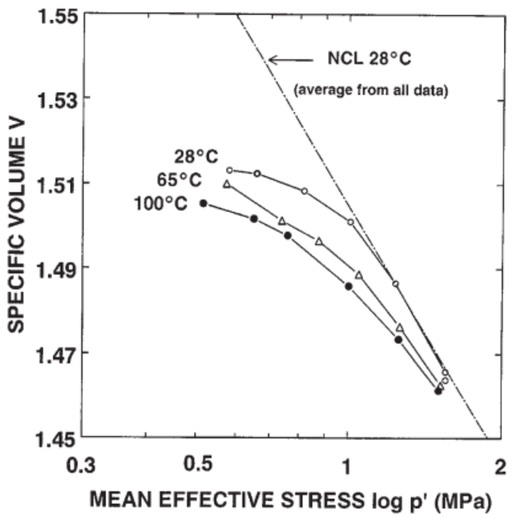


Figure 3-3: Mean effective stress vs. specific volume (Graham et al., 2001)

An increase of the normal compression index, λ, will give the soil greater settlements under the same load compared to a soil with a smaller λ, this is because the change in void ratio in the NC range will become bigger, see equation 3.1. The soil is more compressible with a greater λ. This can be problematic with regards to the structural challenges on e.g., uneven settlement of foundations. With a smaller λ there is experienced less settlements which is beneficial. However, soil with small λ might make it hard to excavate or penetrate due to the compacted soil. It takes more effective stress applied to reach the same deformations for a small λ rather than for a greater λ.

A larger recompression index, κ, indicates that a soil will continue to compress after the primary consolidation at a relatively faster rate compared to a soil with a smaller normal recompression index. Looking at equation 3.2 the change in void ratio becomes bigger with a high κ. In other words, the compression in the soil becomes larger for a large κ. If the overtime settlements are not accounted for it can cause long-term damage. On the other hand, a smaller normal recompression index can result in more stable and durable foundations.

NC range

$$\Delta v = -\lambda \Delta \ln \sigma' = -\lambda \ln \frac{\sigma'}{\sigma'_0} \qquad 3.1$$

OC range $$\Delta v = -\kappa \Delta \ln \sigma' = -\kappa \ln \frac{\sigma'}{\sigma'_0} \qquad 3.2$$

The principles of the "Soft Soil Creep" model can be effectively demonstrated through oedometer testing. In this context, modified compression indexes (as given in equation 3.3 and 3.4) are used as a function of the oedometer modulus (M) and applied stress (σ) (František Havel, 2004). The oedometer modulus and shear strength are key parameters that influence a soil's resistance to failure, with a higher modulus number, m, indicating a stiffer material. Nilmar Janbu (1989) has suggested that a normal consolidated specimen would have a greater oedometer modulus number if the average stress exceeds $100\ kPa$ as compared to an overconsolidated specimen.

$$\lambda^* = 1/m_{nc} \qquad 3.3$$

$$\kappa^* = 1/m_{oc} \qquad 3.4$$

The strength of clay is influenced by several factors, such as whether the specimen is normal consolidated or overconsolidated and whether the test is conducted under drained or undrained conditions. Notably, an increase in temperature can lead to a significant increase in shear strength and stiffness for NC specimens due to a decrease in pore water pressure caused by heating (Kuntiwattanakul et al., 1995). However, Kuntiwattanakul et al. (1995) have found that the strength of OC samples under drained heating is not sensitive to temperature. Similarly, Robinet et al. (1996) have conducted tests on clays under drained and undrained conditions and reported that the strength of clays is not sensitive to temperature arising under drained conditions, while it is sensitive to temperature arising under undrained conditions.

3.2.2 Thermal Loading Under Constant Pressure

Under drained thermal loading that acts at a constant stress level, the interparticle physico-chemical forces will increase and as a result the repulsive forces will increase (Abuel-Naga et al., (2007)). An increase in repulsive forces can make it more difficult for the clay particles and its structure to stick together. If this were to happen in the ground, in or around a neighbourhood, the increase in repulsive forces might have severe or even fatal consequences, e.g., by causing landslides or other forms for ground failure.

The impact of thermal loading on the properties of soft soils, such as void ratio and maximum stresses, has been widely investigated by several researchers. Thermal loading can be applied in the form of a thermal cycle or as a constant increasing or decreasing thermal load. The thermal behaviour of soils is dependent on the thermal time history-deviation of heating phase, according to Burghignoli et al. (2000). Moreover, it has been observed that there are distinct responses for NC and OC clays. In the case of thermal cyclic loading applied in the drained condition tests conducted by Burghignoli et al. (2000), the void ratio at increasing temperature was found to decrease for NC clay, while it increased for OC clays. During cooling in the thermal cycle, the void ratio was found to be insensitive to temperature changes. Several researchers have reported that the void ratio is sensitive to temperature changes, particularly above the yield surface (Campanella & Mitchell, 1968; Chen et al., 2023; Demars & Charles, 1982; Graham et al., 2001; Hu et al., 2022; Kuntiwattanakul et al., 1995; Plum & Esrig, 1969; Robinet et al., 1996; Sultan et al., 2002; Towhata et al., 1993). Abuel-Naga et al. (2007) have stated that the change in void ratio is independent of the stress level for both normally- and overconsolidated specimens.

The impact of temperature change on the maximum stresses of soil is dependent on the consolidation state of the specimen. Specifically, for normally consolidated clay, a rise in temperature leads to a change in the maximum deviator stress, whereas for overconsolidated clay, there is no change in value. However, if NC clay is first cooled and then sheared, the maximum deviator stress will remain unchanged, similar to an overconsolidated clay. (Kuntiwattanakul et al., 1995). The preconsolidation stress is also influenced by the temperature change and stress history. For higher temperatures, the preconsolidation stress is lower than for lower temperatures (Chen et al., 2023; Eriksson, 1989; Hueckel & Borsetto, 1990; Robinet et al., 1996; Sultan et al., 2002; Tidfors & Sällfors, 1989). Hueckel & Borsetto (1990) and Robinet et al., (1996) explains this as the preconsolidation stress depends on the mechanical volumetric strains as well as the thermal volumetric irreversible strains. Furthermore, the yield surface of soil is explicitly temperature dependent, and for the cooling in the thermal cycle the two different research groups differ in their conclusion. Hueckel & Borsetto (1990) consider that the yield surface can change infinitely with temperature when no irreversible strains are produced. While, Robinet et al. (1996) concluded that during cooling the preconsolidation stress in room temperature is higher and the stress-point is slightly decreased due to the reversible contractive strains. So not only is the preconsolidation stress dependent on the stress history, but it seems like the thermal history can have affected the p_c.

3.3 Summary

This chapter presents a literature review by researchers throughout the century. Questions have arisen about storing nuclear waste in the seabed, particularly due to the heat generated by the spent fuel. To address the challenge of heat storage, researchers have explored geothermal structures like geothermos or energy piles for efficient energy storage. The temperature changes in clay due to heating or cooling can significantly impact its properties, including volume change, pore water pressure, compressibility, and strength. These changes have implications for engineering applications such as geothermal structures, radioactive waste disposal, and buried cables. Various tests, including oedometer, Constant Rate of Strain (CRS), and triaxial tests, have been conducted to study the effects of temperature change on clay properties. The behaviour of clay under temperature change is influenced by factors such as the overconsolidation ratio, plasticity index, specimen type, and loading condition. The thermal loading can affect the interparticle forces and the strength of clays. Understanding the thermo-mechanical properties of clay is crucial for assessing its behaviour in different scenarios and designing sustainable structures.

4 Experimental Background

The following chapter is crucial for understanding and further analysing of following chapters. Experimental Background establishes the foundational concepts and context that are necessary for comprehending the subsequent material. By providing essential information on how to do the index testing and the outcoming parameters in both index testing and the CRS testing.

4.1 Index Testing

4.1.1 Remoulded Shear Strength and Sensitivity

The fall cone test is a way to determine the undrained shear strength of undisturbed and remoulded specimens. The shear strengths are calculated by equation 4.1, according to ISO 17892-6. The classification of the soil type and shear strength based on the undisturbed shear strength is shown in Table 4-1.

$$c_u \ (or \ c_{ur}) = c \ g \frac{m}{i^2} \tag{4.1}$$

where

c_u	is the undrained shear strength, undisturbed
c_{ur}	is the undrained shear strength, remoulded
c	is a constant, dependent on the tip angle of the cone $c = 0.8$ for cones with $30°$ tip $c = 0.27$ for cones with $60°$ tip
g	is the acceleration due to gravity at free fall, usually taken as a value of $9.81 \ m/s^2$
m	is the mass of the cone [g]
i	is the average cone penetration [mm]

Table 4-1: Classification of soil type and shear strength based on undisturbed shear strength

Classification soil type	**Classification shear strength**	c_u **[kPa]**
Very soft	Very low	< 12.5
Soft	Low	$12.5 - 25$
Medium soft	Medium high	$25 - 50$
Stiff	High	$50 - 100$
Very stiff	Very high	> 100

Sensitivity is calculated as shown in equation 4.2, using the shear strength calculated in the equation above.

$$S_t = \frac{c_u}{c_{ur}} = \frac{S_u}{S_{ur}} \tag{4.2}$$

A high sensitivity corresponds a larger undisturbed shear strength than a remoulded. And, for a quick clay the remoulded shear strength is very low due to the loss of strength when the card house structure is being destroyed.

4.1.2 Atterberg Limits

"The fall cone method provides results with higher repeatability, and is the preferred method However, there is a long history of use of the Casagrande method and its use is equally permitted."– ISO 17892-12. The four-point test is preferred from the fall cone method. The purpose of the Atterberg limits is to find the liquid limit and plastic limit. The fall cone method used to determine the Atterberg limits is executed according to ISO 17892-12. The liquid limit, w_L, is the water content of the limit where the soil changes from plastic to liquid state. The liquid limit is found at 10mm penetration for the fall cone test, shown in Figure 4-1. The plastic limit, w_P, of a soil is the water content at which a soil ceases from crumbling and change to plastic phase. The plasticity index, I_P, can be found by equation 4.3 and the classification of the soil's plasticity is given in Table 4-2.

$$I_P = w_L - w_P \tag{4.3}$$

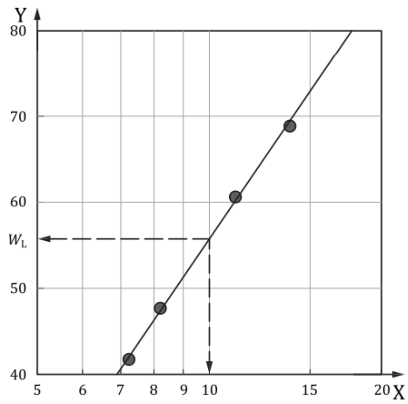


Figure 4-1: Where X is cone penetration [mm] and Y is the water content [%]

Table 4-2: Classification and plasticity based on plasticity index

Classification	**Plasticity**	I_P
Low plastic	Low plasticity	< 10
Medium plastic	Medium plasticity	$10 - 20$
High plastic	High plasticity	> 20

4.1.3 Water Content

The determination of water content is done according to ISO 17892-1. moist test specimen is heated in an oven that holds a temperature of 105 to 110˚C and is taken out of the oven when the mass becomes constant and is now considered as dry. The water content is calculated according to equation 4.4.

$$w = \frac{m_1 - m_2}{m_2 - m_c} x100 = \frac{m_w}{m_d} x100 \qquad 4.4$$

where

w	is the water content [%]
m_1	is the mass of container (and lid if used) and moist test specimen [g]
m_2	is the mass of container (and lid if used) and dried test specimen [g]
m_c	is the mass of container (and lid if used) [g]
m_w	is the mass of water [g]
m_d	is the mass of dried test specimen [g]

Typical values of water content for Norwegian clays are $20 - 50$ %.

4.1.4 Salinity

A low salt content in the pore water may be associated with sensitive or quick clay behaviour. By measuring the electric conductivity (or specific resistivity) of pore water, a measure of the salt content may be obtained. Usually, marine clays will exhibit quick behaviour if the salt content in the pore water drops below approximately $5\ g/L$ pore water ($S{\sim}0.5$ %).

4.2 Modified Oedometer

The oedometer measures settlements due to soil displacement, which is one of the most central subjects in geotechnical engineering. The purpose of the oedometer test is to find deformation and consolidation parameters of the soil. The Constant Rate of Strain (CRS) test will be used in this thesis. The strain is constantly increased in a constant rate, hence the loading is adjusted based on the strain.

4.2.1 Stress History of Soils

The behaviour of soil is complex and dependent on various factors, including its current stress level and stress history, stress history refers to the past stresses that the soil has been subjected to, and it can be categorized as normally consolidated (NC) or overconsolidated (OC). A soil material that is normally consolidated will experience small deformations until it reaches a stress limit. This limit corresponds to the stress level caused by the weight of the soil itself. After reaching the stress limit, the deformations will increase significantly. On the other hand, an overconsolidated material has been exposed to additional stress, causing the weaker parts of the soil to break. As a result, the deformations will be small until the soil reaches the preconsolidation stress level, p_c', which is the maximum stress the soil has experienced before.

When a load is applied to the in-situ soil, the water within it will gradually dissipate, causing the pore pressure to decrease and the effective stresses to increase. The primary consolidation settlement refers to the initial settlements that occur until the pore water is fully dissipated. However, even if the loading remains constant, settlements will continue to increase over time due to creep, leading to a period called secondary consolidation. This phenomenon occurs due to the slow reorganization of the soil structure under sustained loading, resulting in further compression and settlement.

4.2.2 CRS

The constant rate of strain, CRS, measures time (t), load (F), vertical deformation (δ), vertical stress (σ), and pore pressure at the sample base (u_b). The deformation and consolidation parameters are calculated based on these data.

Stress vs. strain

When subjecting the test to an elevated temperature Boudali et al. (1994) performed a CRS oedometer test on Canadian Berthierville clay at different temperatures and found that the mean vertical stress should be after equation 4.5, where σ_v is the applied vertical stress, u_b is the pore pressure at the base of the specimen and u_0 is the applied backpressure. Boudali et al. (1994) observed that at a given strain rate, the influence of temperature on the vertical effective stress of a soil can be significant. It was observed that higher temperatures result in smaller vertical effective stress.

$$\sigma_v' = \sigma_m' = \sigma_v - \frac{2}{3}(u_b - u_0) \tag{4.5}$$

The vertical strain rate is described as a linear function in a logarithm of the preconsolidation pressure, $\Delta\dot{\varepsilon}_1 = m'\Delta\sigma_p'$. Where m', the modulus number, is described as the inclination of the average stress vs. oedometer modulus, and classifies the soil as described in Table 4-3.

Table 4-3: Classification of soil according to modulus number, m

Classification	m
Soft	< 10
Medium	$10 < m < 20$
Stiff	> 20

Average stress vs. oedometer modulus

The oedometer modulus is an important parameter that describes the stiffness of a soil sample under oedometer conditions. Oedometer conditions occur when a soil sample is loaded axially without any lateral constraints or strains. Such conditions are typically found in practical situations where the depth of the soil layer is small compared to the extent of the loading on the soil

A high oedometer modulus corresponds to a stiff material, while a low modulus indicates a more compressible or weaker material. The oedometer modulus can be used to predict the amount of settlement that will occur in a soil layer under a given load. The average stress, σ'_m, and the oedometer modulus, M, can be calculated using the formulas below.

$$\sigma'_m = \sigma - \frac{2}{3}u_b \tag{4.6}$$

$$M = \frac{d\sigma'}{d\varepsilon} \tag{4.7}$$

Coefficient of consolidation vs. average stress

The coefficient of consolidation, c_v, indicates how soft or firm the soil is. A low c_v corresponds to a soft soil, while a high c_v corresponds to a firm soil. Equation 4.8 represents the formula for calculating the coefficient of consolidation for a CRS test.

$$c_v = \frac{d\sigma'}{dt} \cdot \frac{[H_0(1-\varepsilon)]^2}{2u_b} \tag{4.8}$$

where H_0 is the initial sample height. The initial sample height for the GDS CRS is $22\ mm$.

Determination of the preconsolidation

For the determination of the preconsolidation, p'_c, an interpretation of the curves created from the results ($\sigma' - \varepsilon, \sigma' - u_b, \sigma'_m - M$ and $\sigma'_m - c_v$) need to be done. p'_c is expected to be located where the pore pressure and rate of strain increase. When the soil is subjected to an elevated temperature an elasto-thermo-viscoplastic constitutive model for clay derived by Yashima et al. (1998) describes the preconsolidation pressure as a function of temperature by equation 4.9.

$$\frac{p'_c}{p'_{cr}} = \frac{\sigma'_p}{\sigma'_{pr}} = \left[\frac{T_r}{T}\right]^{\alpha} \tag{4.9}$$

where T_r is the referential temperature, $p'_{cr} = \sigma'_{pr}$ is the referential preconsolidation pressure and α is a viscoplastic parameter.

Void ratio

Void ratio is related to the particle deformations. A high void ratio indicates that the particles contract, while a low void ratio indicates dilating particles. The void ratio relates to the porosity and the specific volume given by equation 4.10.

$$e = \frac{V_p}{V_s} = \frac{\gamma_s(1+w)}{\gamma} = \frac{n}{1-n} = 1 - v \qquad 4.10$$

where V_p is the volume of voids, V_s is the volume of solids, v is the specific volume and n is the porosity. The specific volume can is easily be converted into volumetric strains. Moreover, the volumetric strains can calculate settlements and plastic deformation.

Part Ⅱ

Methodology

5 Experimental Methodology

5.1 Introduction

This thesis explored the effect of temperature on soil behaviour, using a modified GDS low-pressure cell to evaluate the temperature effect. The modified GDS cell involves adding heat to the cell through heated silicone oil circulating in a copper coil. The silicone oil is heated by the Grant Instrument Advanced Optima TX150. The copper coil is efficient at transferring heat due to its high thermal conductivity of approximately $400\ W/m \cdot K$. The thesis investigates the insulation of the cell, which is necessary to prevent the glass from transferring heat from the cell to the test room, which has a temperature of approximately 22°C. The limitations of the thesis are also discussed, including time limitations, which prevented the study of secondary consolidation settlements and the effects of thermal cycles. The research is more qualitative than quantitative, and the approximated parameters found in the thesis can be helpful for future researchers to perform more quantitative research.

5.2 Set up Parameters

The oedometer tests were run at Constant Rate of Strain, CRS, on specimens with a height of $22\ mm$ and a diameter of $50\ mm$. The specimens were trimmed at room temperature of approximately 20°C immediately after the specimens was taken out of the refrigerator. The specimens were then mounted in the oedometer cell, at room temperature of approximately 20°C, and subjected to a vertical stress of $0\ kPa$ and applied back pressure of $0\ kPa$.

The normal oedometer cell was located in the main laboratory. The oedometer ring had a height of $20\ mm$ and a area of $20\ cm^2$. The specimen was trimmed in a self-made construction made by the staff at NTNU. Where the oedometer ring is lubricated with silicone oil and fastened to the construction. The construction guides the ring down towards the specimen and penetrated halfway through the specimen. The specimen is then trimmed above, below and around the ring. This ensures that the specimen has the same shape as the oedometer, leaving no traces of clay outside the oedometer ring.

The clay was tested under two temperatures, 20°C and 70°C. The test at 20°C was executed in the main laboratory in a drained and saturated oedometer cell at a constant rate of strain. The test runed at 70°C was executed in a modified GDS low pressure cell, further elaborated in chapter 5.3. For all of the tests the strain rate of $108\ micrometer/hr$ and $0.54\ \%/hr$, for the normal oedometer cell and for the modified GDS respectively. According to NS8018 the strain rate with a $20\ mm$ high test should be around $0.25\ to\ 0.75\ \%/hr$ for clays. A strain rate of $0.54\ \%/hr$ corresponds to Boudali et al., (1994) strain rate of $1.5\ x\ 10^{-6}\ s^{-1}$.

5.3 Research Design

The focus of this thesis is to study the effect of temperature on soils behaviour. To evaluate the temperature effect a modified GDS low pressure cell is being developed. The modification involves adding heat in the cell. This was the only consolidation cell available for this kind of modification due to its free space between the glass and the plastic frame. A schematic drawing of the modified GDS is presented in Figure 5-1. In the free space between the glass and plastic frame made it possible to insert a copper coil with a diameter of $8\ mm$ inside the cell. The copper coil is manufactured in a cylindrical form with the intention to spread and transfer the heat to the water cell and the specimen. The heat is transferred from the silicone oil running through the copper coil. The silicone oil has a flash point at above $200°C$. The Grant Instrument Advanced Optima TX150, further on called TX150, heats the silicone oil in its tank. The TX150 properties are given in Table 5-1.

Table 5-1: Product performance and electrical details

Stability (DIN 12876)	$\pm 0.01°C$
Uniformity (DIN 12876)	$\pm 0.05°C$
Max pump head pressure	$310\ mBar$
Max pump flow pump	$18\ L/min$
Max current consumption	$9\ A$
Heater power	$1.8\ kW$
Mains supply	$220 - 240\ V\ @50/60Hz$

The coil used in the cell is a copper coil due to its high thermal conductivity of approximately $400\ W/m \cdot K$ which is eight times higher value than the conductivity of steel, and 1.6 times higher than for aluminium. The high thermal conductivity indicates that the copper is very efficient at transferring heat. By means that the copper is able to transfer heat to or from surroundings very quickly. In this project the copper will transfer heat from the silicone oil, heated in the TX150, to the water in the cell. The water in the cell will further on heat the clay inside the oedometer ring placed inside the plastic frame.

The copper coil is placed close to the glass for the cell. An ordinary glass has a thermal conductivity of $0.8\ W/m \cdot K$, (*Thermal Conductivity*, n.d.). To prevent the glass from transferring heat from the cell to the test room, that has a temperature of approximately $20°C$, it is desired to insulate the cell as shown in Figure 5-2. The test room consists of other apparatus, these apparatuses need a stable temperature to give reliable results. As well as the water tank in cell is dependent on a constant heat so that the plastic frame, shown in Figure 5-3, can reach the same temperature and further heat the sample inside the oedometer ring. The temperature calibration is further elaborated chapter 5.3.2.

When testing the copper coil for leakage a defrosting liquid was used, due to its pink colour that was easily detected without any use of external equipment. A leakage was detected in the connection between the hose and the copper coil, as shown with the red ring in Figure 5-4. The leakage was avoided by tighten the nut approximately 45 to $90°$.

During the first test run when the defrosting liquid was replaced with silicone oil, the soldering flux used to attach the two independent copper coils together to maximize heat transfer, reacted with the water or with the silicone oil. The reaction is shown in Figure 5-5. The solder is only used on the outside of the copper coil and because of the temperature of the incoming silicone oil from the TX150 and the outcoming oil is approximately the same, the reaction should not affect the silicone oil's flow. The water flow should not be affected by the reaction due to the small affected area. Therefore, the reaction shown in Figure 5-5 is not of a problem running the test.

Silicone oil is used due to its high flash point compared to the defrosting liquid. The silicone oil has a density of $0.963\ g/mL$ and a flash point of $> 270℃$, (*Silicone Oil for Oil Baths – 40…+200°C*, n.d.). The good thermal characteristics makes the silicone oil remain its temperature throughout the copper coil loop. And, due to insulation of the tubes to and from the TX150 the temperature of the silicone oil does not have a great variation whether it is on its way from or to the TX150.

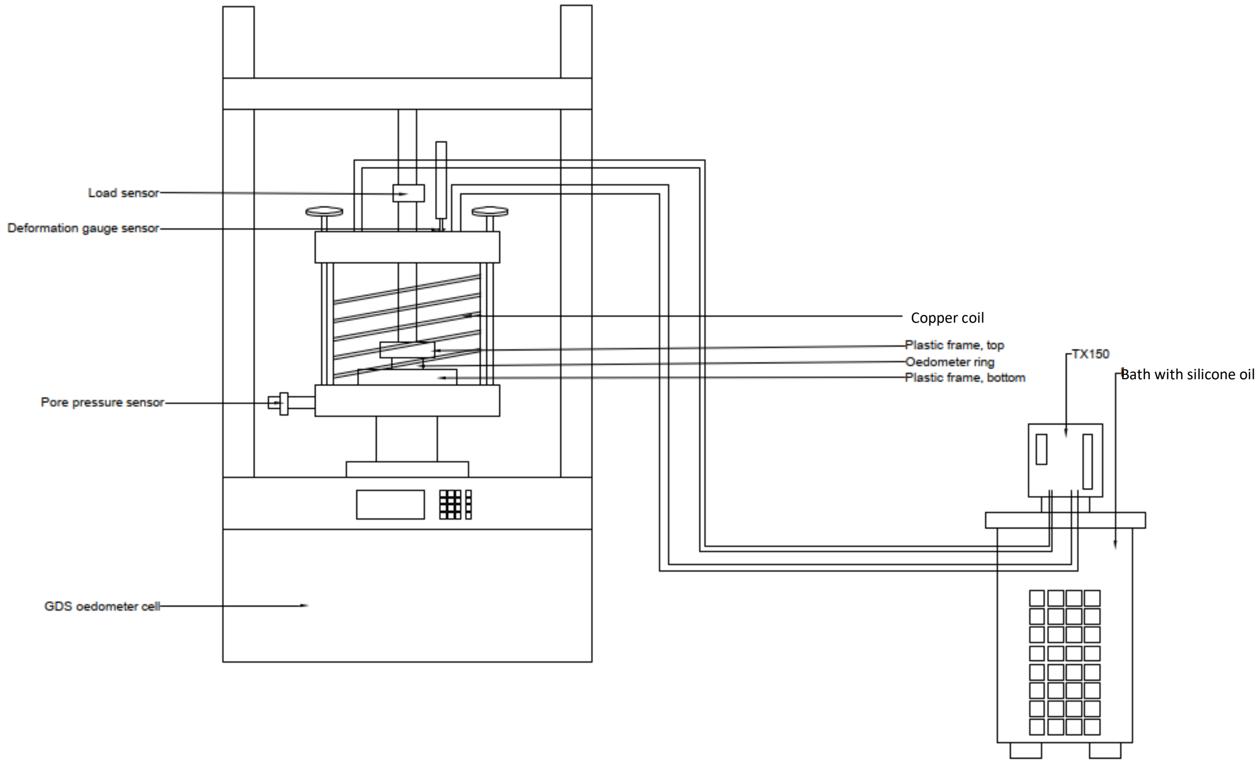


Figure 5-1: Sketch of the GDS setup

Figure 5-2: Insulated cell and tubes

Figure 5-3: Plastic frame

Figure 5-4: Connection between the hose and the copper coil

Figure 5-5: Flux used in the soldering

5.3.1 Index Testing

The results from the index testing are presented in Table 5-2, as well as remoulded shear strength, water content, liquid limit and Atterberg limits are presented in Appendix 1.

The resistivity values obtained during the index testing provided inaccurate results, leading to a lack of meaningful interpretation when converting $\Omega\ cm$ to %. Consequently, there is no specific numerical value for the salinity of the clay samples. Although it is worth measuring that the $\Omega\ cm$ values is approximately the same as fresh water. Based on other discernible characteristics, it becomes evident that the tested clay samples are classified as quick clay. Despite the absence of salinity data, the observed properties, and behaviours of the clay point towards its classification as quick clay.

Table 5-2: Geotechnical properties of tested clays

	Depth	Undisturbed shear strength c_u	Remoulded shear strength c_{ur}	Sensitivity S_t	
282	7-8 m	43.45	0.41	104.99	
V121	8-9 m	28.47	0.11	258.27	
708	7-8 m	64.07	0.12	544.22	
V124	9-10 m	38.76	0.25	153.73	
508	10-11 m	148.60	0.15	1022.49	
	Depth	**Liquid limit**	**Plastic limit**	**Plasticity index**	**Water content**
282	7-8 m	34.54	28.48	6.05	44.04
V121	8-9 m	30.71	26.96	3.75	45.63
708	7-8 m	33.50	29.96	3.54	40.56
V124	9-10 m	32.97	27.98	4.99	43.31
508	10-11 m	33.57	26.71	6.86	40.91

5.3.2 Calibraion test, Temperature

To ensure that the thermocouples used for temperature registration is somewhat calibrated, the thermocouples was placed in an ice bath. The registered temperatures from the three thermocouples at two different predominated temperature baths are shown in Table 5-3 The thermocouples have an accuracy $\pm 0.5°C$, which means that the warm bath of 30°C most likely was a degree less.

Table 5-3: Calibration of thermocouples

	Temp 0	Temp 1	Temp 2
Ice bath	$-0.16°C$	$-0.14°C$	$-0.17°C$
Warm bath at 30°C	29.11°C	29.10°C	29.19°C

For the next calibration test the thermocouples are placed at different locations in and around the cell. The location of the thermocouples' measurements is described in Table 5-4. Ceramic clay body is used for testing the temperature. Initial temperature of the model clay out is approximately 17°C. A ceramic clay body is a mixture of clay or clays and other earthy mineral substances, which are blended to achieve a specific ceramic purpose. The ceramic clay body is a good indication on how the clay will be affected by temperature.

Table 5-4: Description of data labels

Name	Location
Temp 0	Sample
Temp 1	Water tank of the cell
Temp 2	TX150

To evaluate whether the TX150 has the ability to heat the clay sample up to a desired temperature there was executed a calibration test. From Figure 5-6 the TX150 was set to heat the silicone oil to $70°C$ and as illustrated, the clay is heated up to only $56°C$ which indicates that the temperature difference is $15 - 16°C$. It is also important to note that the water tank in the cell keeps a temperature of around $65°C$, which means that the water temperature is not a good representative for the temperature in the clay.

As previously described, there is a temperature difference between the water tank and the sample, illustrated in Figure 5-6. The plastic frame in Figure 5-3 has a trapezoid shape seen from the side, this is shown in Figure 5-7. The distance between the two frame components is $6\ mm$. The plastic has a low heat conductivity and will not transfer the heat as well as e.g., the copper. However, the issue might be at the base of the cell. The temperature of the piston underneath the sample remains cooler than the water in the cell. The base of the cell does not warm up to the same temperature as the top of the sample, and some heat loss occurs.

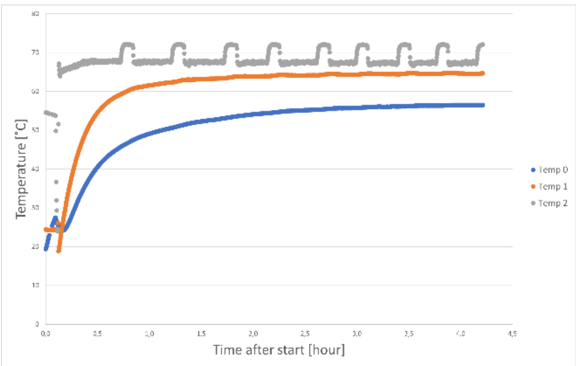


Figure 5-6: Temperature calibration test I

Figure 5-7: Schematic drawing of the plastic frame

Temperature calibration test II is done to evaluate how quick the clay responds to a change in temperature. The clay and water respond to change in temperature and stabilize after the temperature gradually has been increased. Figure 5-8 shows a clear trend that the temperature in the water holds a temperature of 15 degrees warmer than the temperature in the clay.

Figure 5-9 shows temperature calibration test on model clay as well. Figure 5-8 indicates that the water tank temperature in the cell is not a good representation of the temperature in the clay. To evaluate where to place the thermocouple to get the best fit with the temperature of the clay, Temp 0 is still placed in the clay, but now Temp 1 is placed inside the plastic frame as shown in Figure 5-10. Temp 2 is placed in the same place as the other tests, inside the silicone oil bath in the TX150. Looking at Figure 5-9 the temperature difference between Temp 0 and Temp 1 is very small, approximately 5℃. According to Figure 5-9 it takes five hours to stabilize and heat the clay to 70℃, therefor the test will be set to have a "saturation stage" of 6 hours to be conservative.

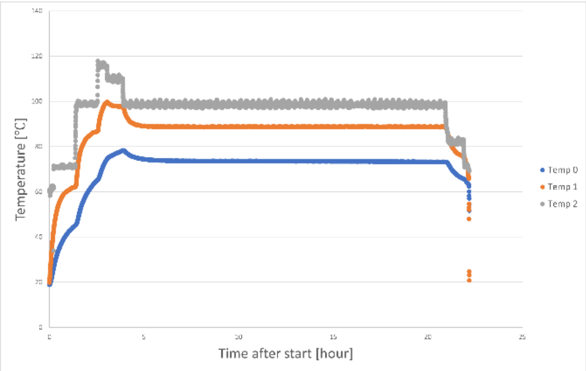


Figure 5-8: Temperature calibration test II

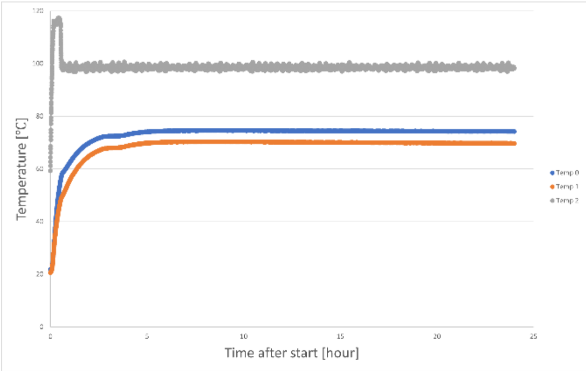


Figure 5-9: Temperature calibration test III

Figure 5-10: Placement of the thermocouple Temp 1, temperature calibration test III

To ensure that the deformation gauge or cell was not affected by expanding or contracting during change in temperature, the test was subjected under heating as shown in Figure 5-11. The result from the test is illustrated in Figure 5-12, and shows that the deformation gauge is neglectable affected by the temperature change, with a registered deformation of $0.098\ mm$ after around 5 hours with a temperature ranging from $70 - 100$℃ in the TX150 tank.

The pore pressure has been tested for calibration of four different temperatures. The pore pressure has been measured in the pore pressure sensor in the GDS and at the same time measured in FLUKE 719Pro300G. The pore pressure differs at most at $2.88\ kPa$ which can be neglectable. The pore pressure registration is presented in Figure 5-13.

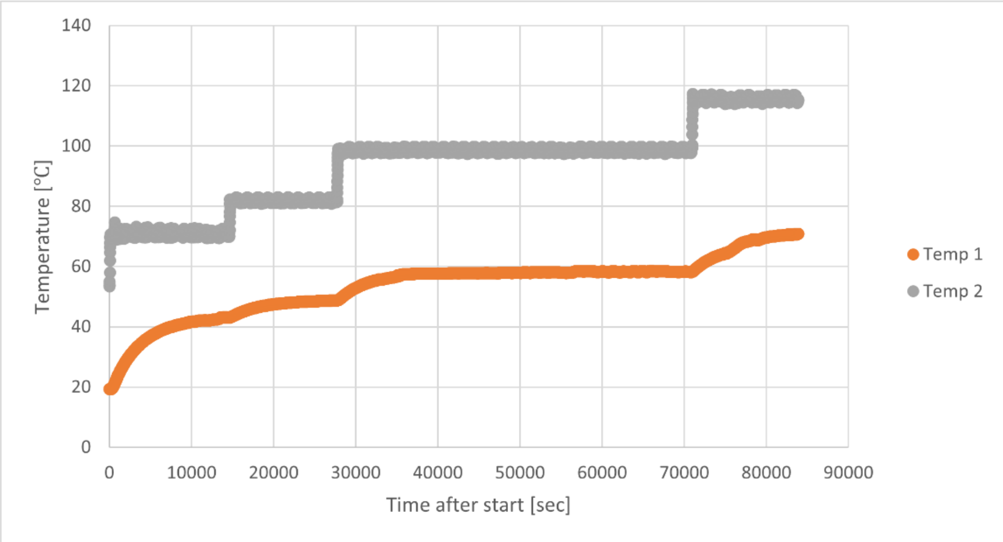


Figure 5-11: Temperature, calibration of deformation gauge

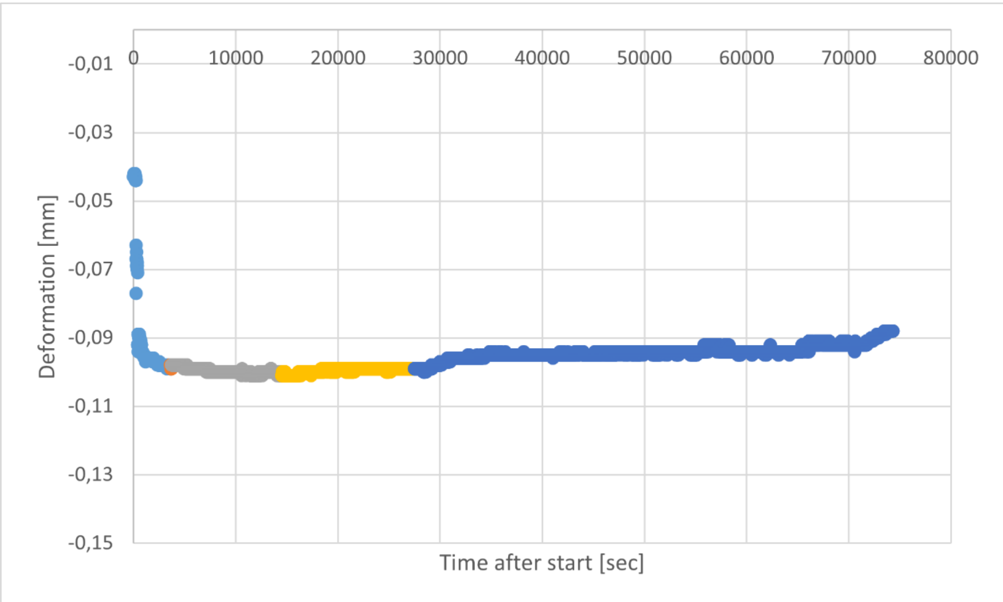


Figure 5-12: Calibration of deformation gauge

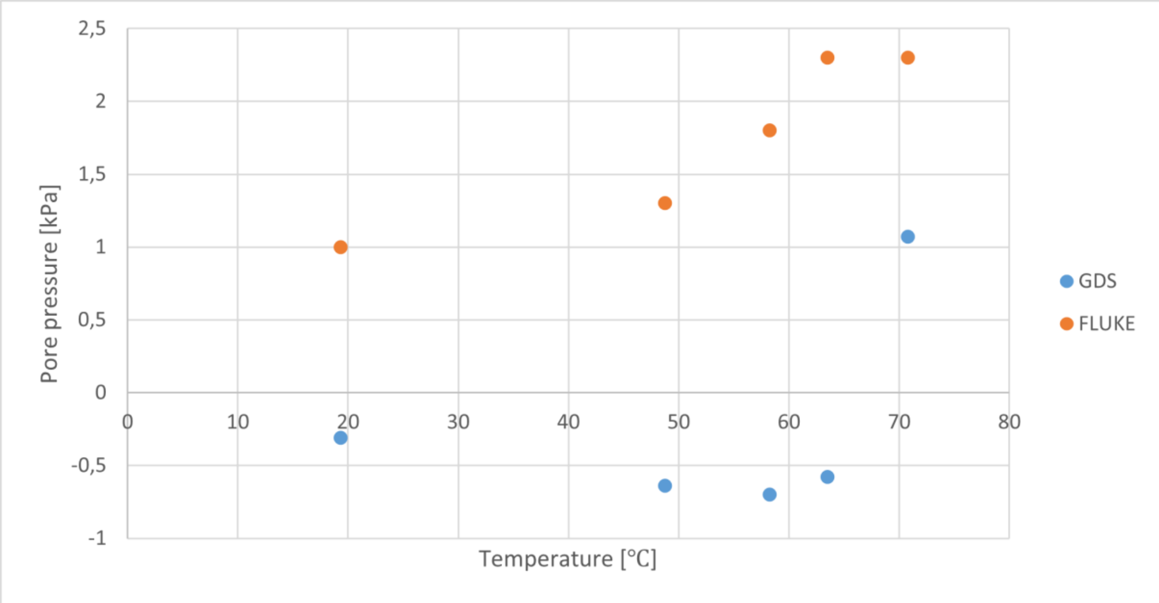


Figure 5-13: Temperature vs. measured pore pressure from GDS and FLUKE

The thermal affected tests were performed in the order given in Table 5-5.

Table 5-5: Temperature affected tests done in the GDSCRS

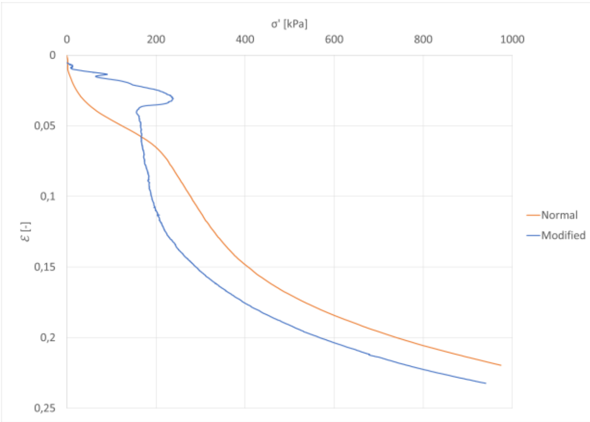
Test	Date
Calibration, temperature	29.03
Calibration, temperature	12.04
Calibration, temperature	29.04
508 70°C	29.04
V121 70°C	06.05
708 70°C	09.05
282 70°C	13.05
Calibration, pore pressure	15.05
708 50°C	16.05
708 50°C (7.55-7.6 m)	19.05
V124 70°C, temperature calibration	21.05
V124 50°C	23.05

5.3.3 Calibration test, Loading

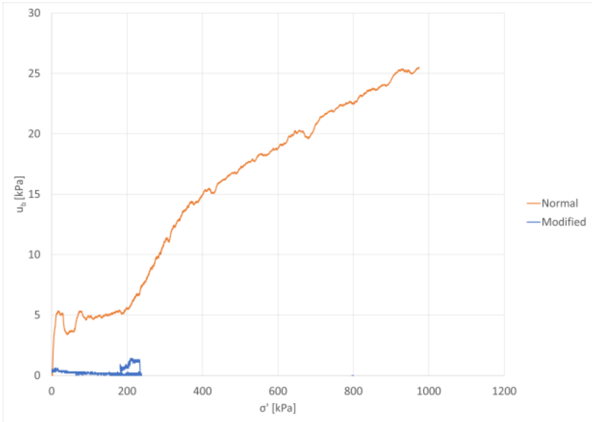
First step of calibration

For the loading calibration test, the rest of the 282 test was used from the depths 7.57-7.67 m. The results are presented in Figure 5-14 (a), (b), (c) and (d). The results from the normal oedometer and the modified oedometer varies a lot which might indicate that there is something wrong with either of the tests. Due to the fact that the modified oedometer last was calibrated in 2018 and has only been used for a master's thesis immediately after calibration. Whereas the normal oedometer in the main laboratory is used constantly by e.g., people writing their master's thesis and courses in geotechnical laboratory. Because the normal oedometer is used constantly the possibility that the modified oedometer has e.g., air in the pore pressure sensor is more likely than there being air in the pore pressure sensors in the modified oedometer.

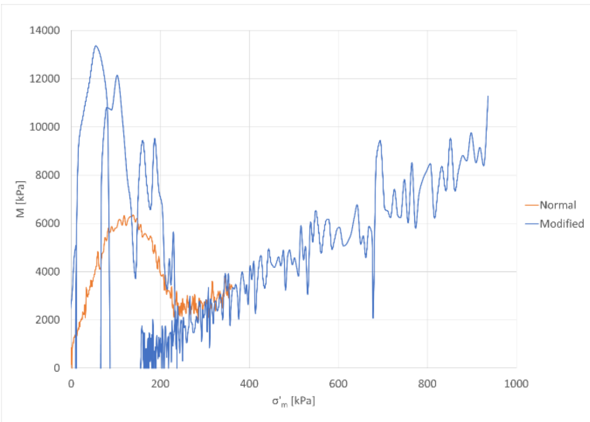
Figure 5-14: Calibration sample 282



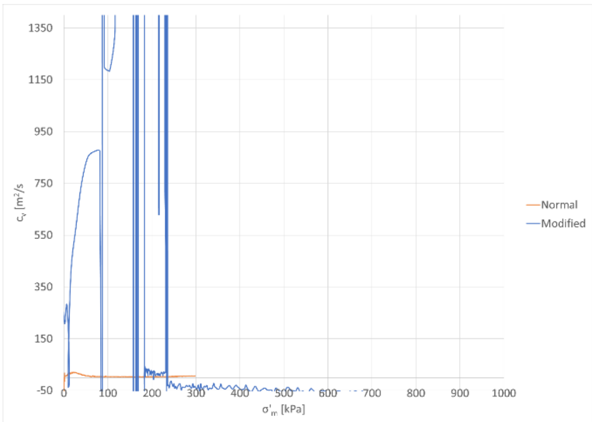
(a) Effective stress vs. strain



(b) Effective stress vs. pore pressure



(c) Mean effective stress vs. oedometer modulus



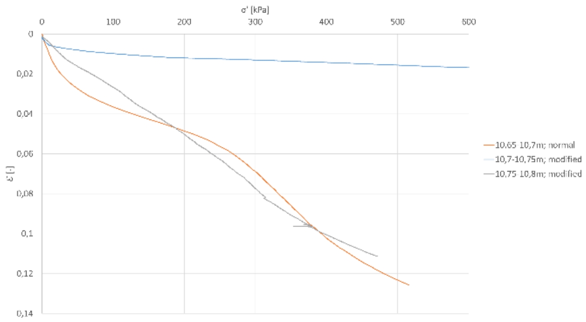
(d) Mean effective stress vs. coefficient of consolidation

Second step of calibration

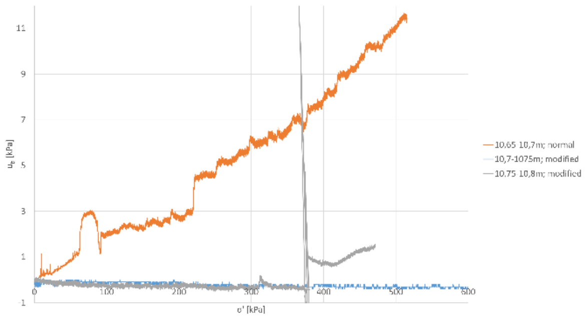
To ensure that the results from 282 was not only a one-off case for the modified oedometer the test was carried out on the rest from the sample 508 test, with the depths 10.65-10.75 m. The specimen was divided into to two parts, one for the normal oedometer and one for the modified oedometer, respectively in the depths 10.65-10.7 m and 10.7-10.75 m. The results are given in the Figure 5-15 (a), (b), (c) and (d).

The results given are indicating that the normal and modified oedometer gives different preconsolidation pressure, pore pressures, strengths, and coefficient of consolidation.

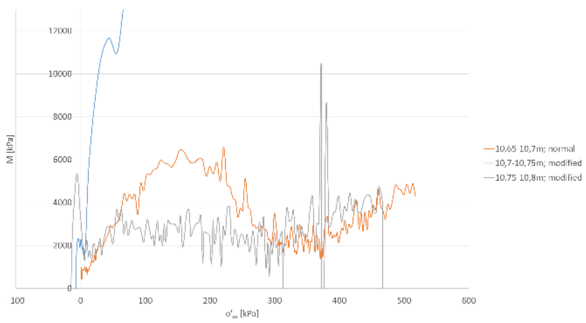
Figure 5-15: Calibration sample 508



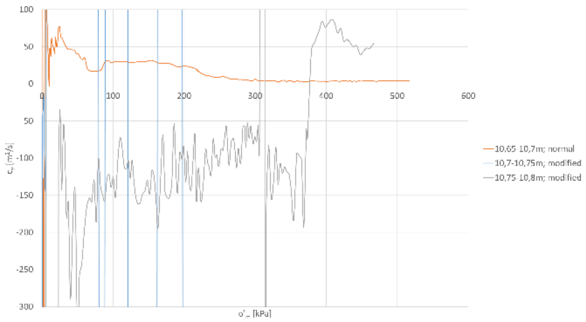
(a) Effective stress vs. strain



(b) Effective stress vs. pore pressure



(c) Mean effective stress vs. oedometer modulus



(d) Mean effective stress vs. coefficient of consolidation

The need of calibration of the modified oedometer equipment was crucial to ensure that the modified oedometer had the correct properties, such as calibration factors. The results from the troubleshooting are given in Table 5-6. Where the calibration factors after troubleshooting is needs to be multiplied with the previous values given in the testing.

Table 5-6: Results from troubleshooting in loading calibration

	Calibration factor	**Calibration factor, after troubleshooting**
Load cell pressure	1	0.97
Displacement needle	1	0.3681047
Pore pressure sensor	1	10.001

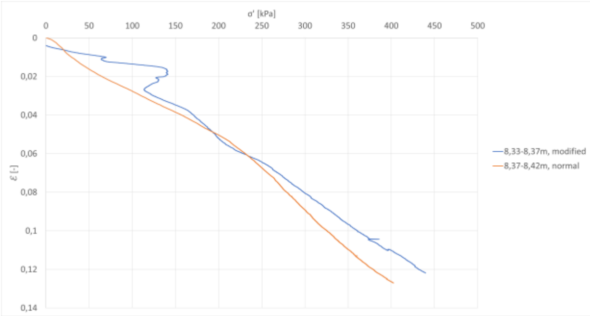
After the calibration factors were set in orders there was done two more tests from 5 cm further down in the ground, from 10.75-10.8 m. The results are given in Figure 5-15 (a), (b),

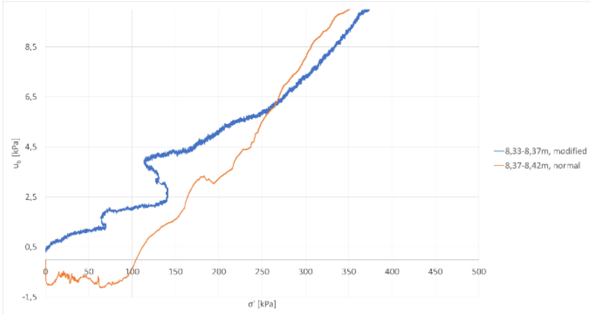
(c) and (d) to compare the results previously the calibration factors with the results after the calibration factors.

Third step of calibration

To ensure that the results given in the two oedometer tests are approximately the same, the tests were run for test V121, Figure 5-16 (a), (b), (c) and (d). By comparing the results in the tests from modified and normal oedometer, the property of the clay is approximately the same and the real testing can start.

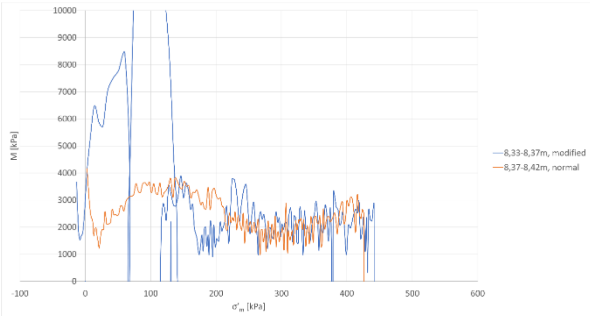
Figure 5-16: Calibration sample V121

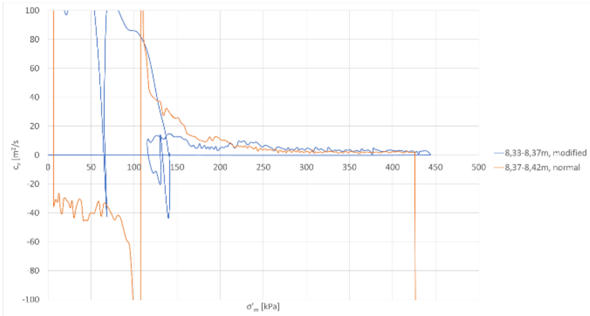




(a) Effective stress vs. strain

(b) Effective stress vs. pore pressure





(c) Mean effective stress vs. oedometer modulus

(d) Mean effective stress vs. coefficient of consolidation

5.4 Limitations

There are several limitations in this thesis. The most severe is the time limitations. First and foremost, the thesis only lasts for one semester. This demands that the experiments will not cover secondary consolidation settlements, nor will the experiments cover the effects of thermal cycles.

The modification of the GDS oedometer cell took longer time than expected. Due to other master's thesis and NTNU's ongoing projects, not to forget a longer delivery time on the copper coil than expected. Therefore, the workshop team that were to create the copper coil took longer time. When the copper coil was implemented in the cell, there were other obstacles. The obstacles in the calibration period were among others, the temperature to get an accurate temperature in the sample, so that the sample and the water in the cell could have the same temperature. This was not possible due to the heat transfer in the bottom of the cell and in the plastic frame, due to the low heat conductivity. It is not possible to measure the temperature inside the sample during the CRS loading because the load and the sharp edge on the ring might cause the thermocouple to split into two. This actually happened with Temp 0, and there were only two thermocouples left. Therefore, the temperature inside the plastic frame was only measured in the calibration before starting and during one of the last tests, shown in Table 5-5.

Due to the time limitation the research was more qualitative than quantitative. The tests were thoroughly tested with a focus that researchers in the future can use the parameters and information found in the thesis to test their own quick clay samples. The comparison between the parameters found from the normal oedometer and the modified oedometer, as well as the index parameters found in the thesis from NTNU's test location, Flotten, can be helpful to get approximate parameters. Then, the next researchers can use the approximated parameters to get more quantitative research.

Yet there are other limitations as well. There is no trimming system to adjust the diameter of $54\ mm$ from the extrusion to be the initial diameter of $50\ mm$ in the GDS test. Due to the lack of trimming devices to the ring of $50\ mm$ diameter and $22\ mm$ height, one needs to be precise when pushing down the ring over the sample. If pushing to fast and hard, the sample might get disturbed or there might be air inside the ring, see Figure 5-17. This might cause incorrect results. It is therefore important that ring is slowly pushed down over the sample with silicone around the ring, for smoother penetration.

Figure 5-17: GDS sample ring with clay

5.5 Summary

The aim of the study discussed in this thesis is to examine the impact of temperature on soil behaviour, using a modified GDS low pressure cell with added heat. The modification involved adding a copper coil with high thermal conductivity to transfer heat from silicone oil running through the coil placed in the cell. The copper coil was placed near the glass of the cell, and insulation was added to prevent heat transfer to the test room. A calibration test and a test run using the silicone oil were executed. The silicone oil had a high flash point and good thermal characteristics. The calibration indicate that the copper coil was effective in transferring heat, and the use of silicone oil was successful in maintaining a constant temperature in the system, even though there are temperature differences between the TX150, the water in the cell and in the clay sample.

The project in the thesis has several limitations, with the most severe being time limitations. The experiments could not cover secondary consolidation settlements or the effects of thermal cycles due to the thesis only lasting for one semester. The modification of the GDS oedometer cell took longer than expected. Due to time limitations, the research was more qualitative than quantitative. There is also no cutting system to adjust the diameter of $54\ mm$ from the extrusion to be trimmed into the initial diameter of the ring, $50\ mm$. The comparison between the parameters found from a normal oedometer and from the modified oedometer, as well as the index parameters found in the thesis from Flotten, can be helpful for future researchers to get approximate parameters and conduct more quantitative research.

Part Ⅲ

Results & Discussion

6 Oedometer test

This chapter present the results and discussion from the experimental work done in the laboratory. The results from the oedometer tests are presented in collaborated graphs.

6.1 Thermal Loading and Constant Mechanical Loading

During the heating phase, there is a presence of thermal loading while there is a constant mechanical load of $\pm 0\ kPa$. In this chapter the results and discussion from the heating phase are presented.

6.1.1 Strain vs. Effective Vertical Stress

During the thermal loading from 20°C up to 70°C the strains are increasing while the effective vertical stress is decreasing for most of the clays. Sample 508 has an increasing vertical effective stress, but it is important to mentioning that the stresses are relatively small. The reason why the effective vertical stress is negative is due to the fact that the pore pressure is higher than the applied load. This is further discussed in 6.1.2 Pore Pressure vs. Effective Vertical Stress.

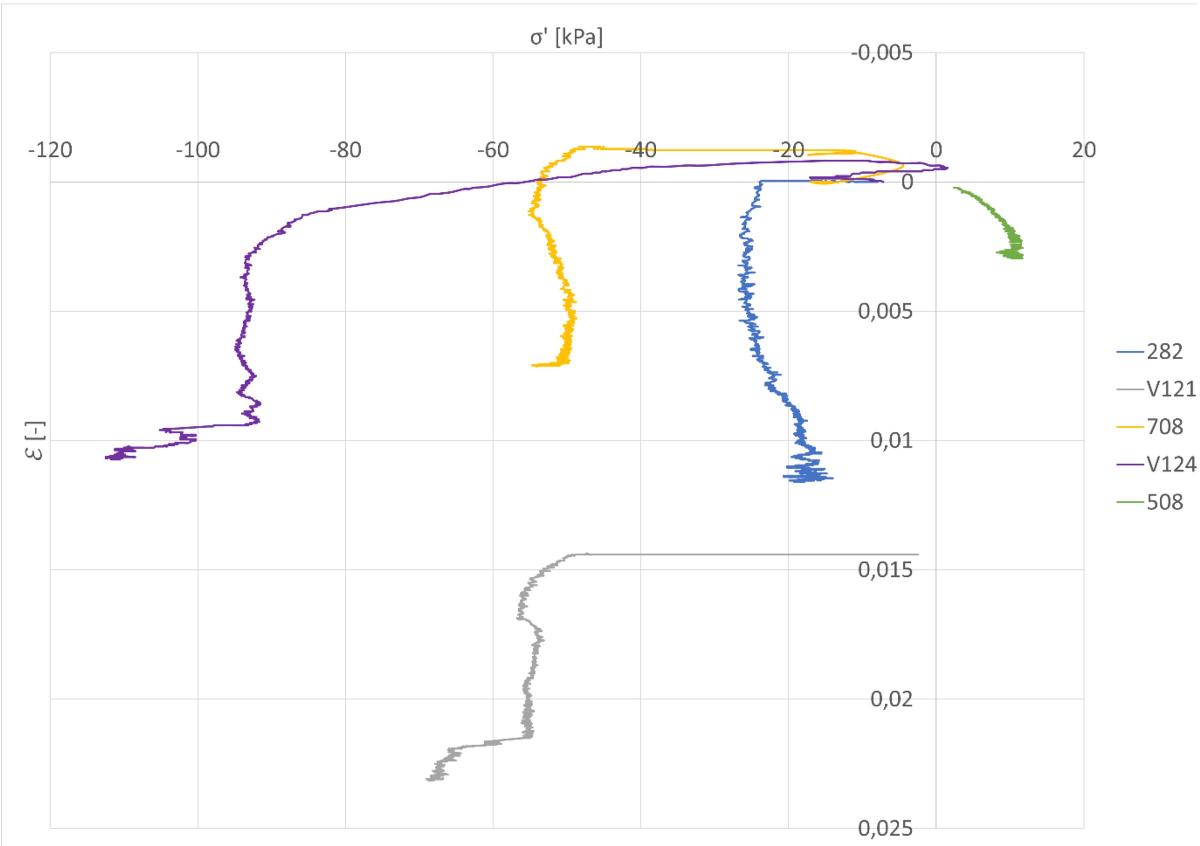


Figure 6-1: Effective vertical stress vs. strain during thermal loading

6.1.2 Pore Pressure vs. Effective Vertical Stress

During the heating phase the pore pressures first decreases and then in order to increases, this can indicate the presence of physico-chemical forces at play. In a natural clay, which consists of minerals, water, gas, and sometimes organic material, capillary forces play a significant role in soil-water interactions (Abuel-Naga et al., (2007)). In a fully saturated clay, the clay particles are surrounded by strongly bound water, and there is minimal particle-to-particle contact. However, in fine-grained soils like clays, as the temperature increases, the small pore space and strong adhesive forces between soil particles and water molecules cause the strongly bound film of water to decrease. Under drained thermal loading, where the stress level remains constant, the physico-chemical forces between the soil particles increase. Leading to a negative pore pressure shown in Figure 6-2 and later on in Figure 6-13. This increase in interparticle forces includes the repulsive forces. As a result, the clay particles and their structure find it more challenging to stick together, leading to potential changes in the soil's behaviour and properties. However, the temperature effects on the interparticle physico-chemical forces exhibit reversible behaviour and is a reversible component (Abuel-Naga et al., (2007)).

The decrease in the strongly bound film of water can lead to the water in the soil being placed under tension. Consequently, the clay experiences a decreasing pore pressure during the heating phase of the specimen. The change in pore pressure is bigger for the NC clay, V121, in this experiment, shown in Figure 6-2 . Otherwise, it does not seem like there is some correlation between the depth of the sample and the pore pressure. The change in pore pressure can have an impact on the geotechnical properties of the soil. With an increase in repulsive forces and decrease in attractive forces between the particles, the strength of the soil is weakened.

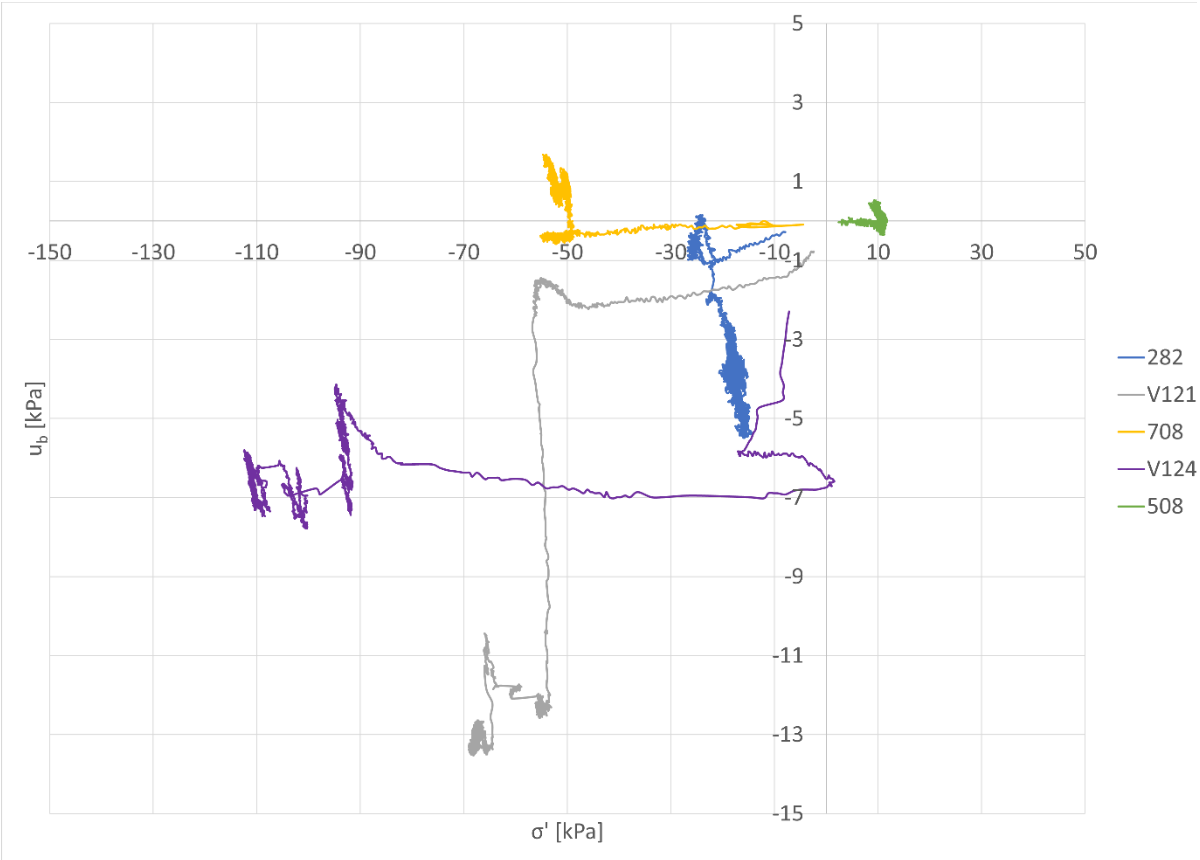


Figure 6-2: Effective vertical stress vs. pore pressure during thermal loading

6.2 Isothermal Loading and Mechanical Loading

Experimental studies shows that the behaviour of natural clays during one-dimensional compression if strongly influenced by strain rate and temperature. In this experiment the strain rate is kept constant, and the influence of temperature is only considered. The effect of temperature on the deformation characteristics, e.g., the preconsolidation pressure, can then be assumed to be dependent on magnitude of temperature increase, clay content, microstructure, and water content. The individual results from the oedometer tests are illustrated in Appendices 2-6. In Table 6-1 the oedometer properties from the tested clays are presented, in the same order as presented in Appendices 2-6. The following chapter consists of results and discussion of the results.

Table 6-1: Oedometer properties from the tested clays

282	20°C	70°C
p'_c [kPa]	210	140
σ'_{v0} [kPa]	143	98
M_{OC} [kPa]	6640	6454
m_{NC}	13.85	7.97
c_v	4.6	–

V121	20℃	70℃
m_{NC}	11.97	7.95
c_v	2.6	–

708	20℃	50℃	70℃
p'_c [kPa]	287	286	203
σ'_{v0} [kPa]	164	181	155
M_{OC} [kPa]	8881	8415	6906
m_{NC}	13.66	16.81	8.19
c_v	7	–	30

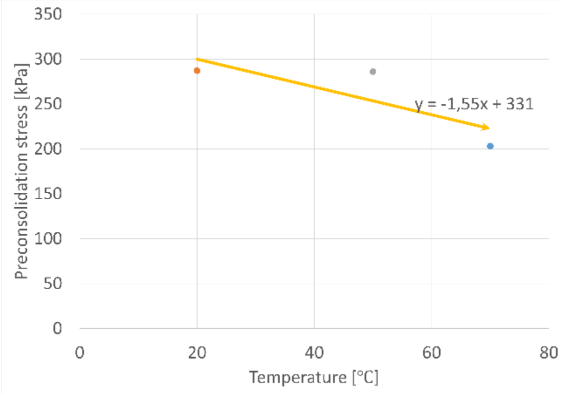


Figure 6-3: Sample 708, temperature vs. preconsolidation

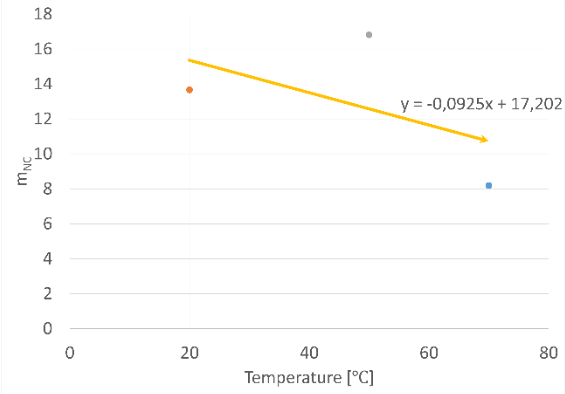


Figure 6-4: Sample 708, temperature vs. m_{NC}

V124	20℃	50℃	70℃
p'_c [kPa]	280	166	130
σ'_{v0} [kPa]	175	3,53	−16
M_{OC} [kPa]	1543	6530	6404
m_{NC}	12.30	17.38	15.32
c_v	−5	–	–

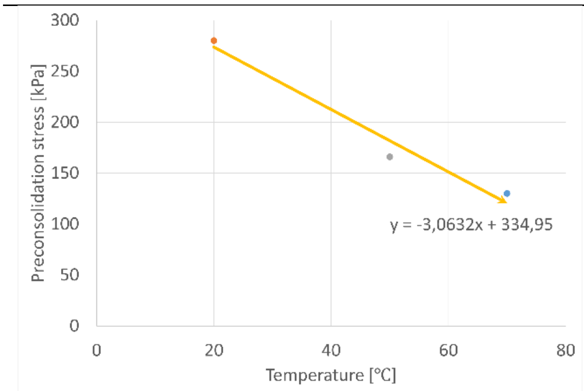


Figure 6-5: Sample V124, temperature vs. preconsolidation

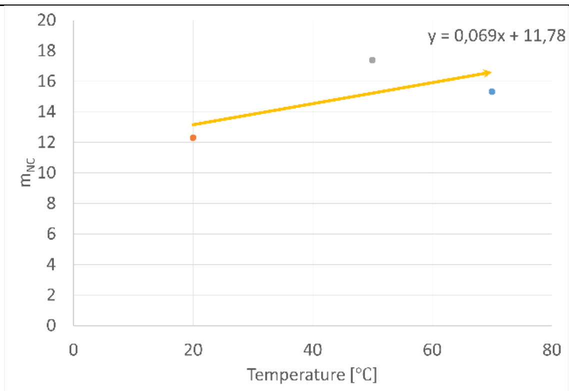


Figure 6-6: Sample V124, temperature vs. m_{NC}

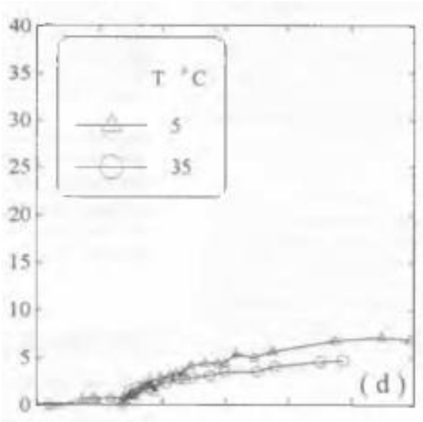
508	20℃	70℃
p_c' [kPa]	250	108
σ_{v0}' [kPa]	177	71,46
M_{OC} [kPa]	6393	12000
m_{NC}	13.85	6.70
c_v	3	−

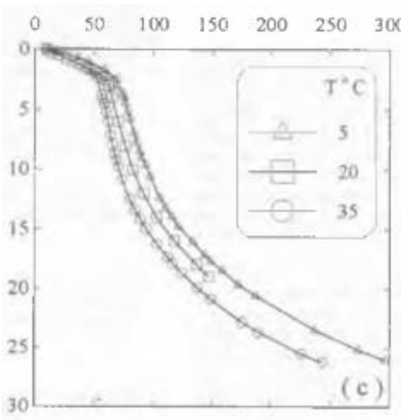
Canadian and Swedish clay

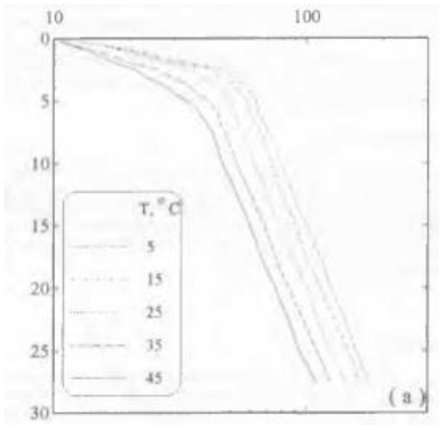
Before continuing to the gathered results given from the experiments, some results from Canadian and Swedish clay will be presented and further on compared with the results given from the Norwegian clay tested from Flotten.

The effects of temperature on the one-dimensional consolidation behaviour of Berthierville clay is presented in Figure 6-7. Boudali et al., (1994) concluded that at a given strain rate, the lower the temperature, the higher the effective stress is at a given strain. Boudali et al. conclusion was already presence in Figure 6-8 (a) from the Luleå clay from Eriksson (1989)

Figure 6-7 (c) shows that the preconsolidation pressure is higher for temperature at 5℃. The pore pressure at the given temperature for preconsolidation pressure starts to increase for effective stresses larger than the preconsolidation pressure, this is illustrated in Figure 6-7 (d).







$\dot{\varepsilon_v} = 1.5\ x\ 10^{-6}\ s^{-1}$

Figure 6-7: Effects of and temperature on the one-dimensional consolidation behaviour of Berthierville clay (Boudali et al., 1994)

$\dot{\varepsilon_v} = 1.5\ x\ 10^{-6}\ s^{-1}$

Figure 6-8: Oedometer tests perfomed on Luleå clay at various temperatures (Eriksson, 1989)

6.2.1 Effective Vertical Stress vs. Strain

The preconsolidation stress decreases with temperature for the tests executed on Flotten clay, this is shown in Figure 6-9. According to Tidfors and Sällfors (1989) the preconsolidation pressure decreases with increasing temperature.

Comparing the results given from Figure 6-9 with the results from Berthierville clay and Luleå clay, given in Figure 6-10 and Figure 6-11, the Norwegian clay from Flotten has a higher preconsolidation pressure. The preconsolidation pressure from Canada and Sweden is below

100 kPa, while the Norwegian clay's preconsolidation is above 100 kPa. The strain is bigger for both Berthierville clay and Luleå clay, leaving bigger deformation for smaller stresses, compared with the Flotten clay. It is, in fact, the tests performed at 20℃ that has the highest strains for Flotten clay sample 282 and V121. Nevertheless, there are certain uncertainties associated with the instruments employed in this study, as the quantitative findings do not align with those obtained from Canadian and Swedish clay samples. However, the qualitative findings indicate a positive correlation and alignment between the results obtained and the characteristics exhibited by the Swedish and Canadian clay. This implies that the curves illustrated in the study exhibit similarities with those observed in samples of clay from Berthierville and Luleå. Therefore, despite some uncertainties with the instruments, the qualitative results suggest a favorable agreement and consistency between the findings and the properties of the Swedish and Canadian clay.

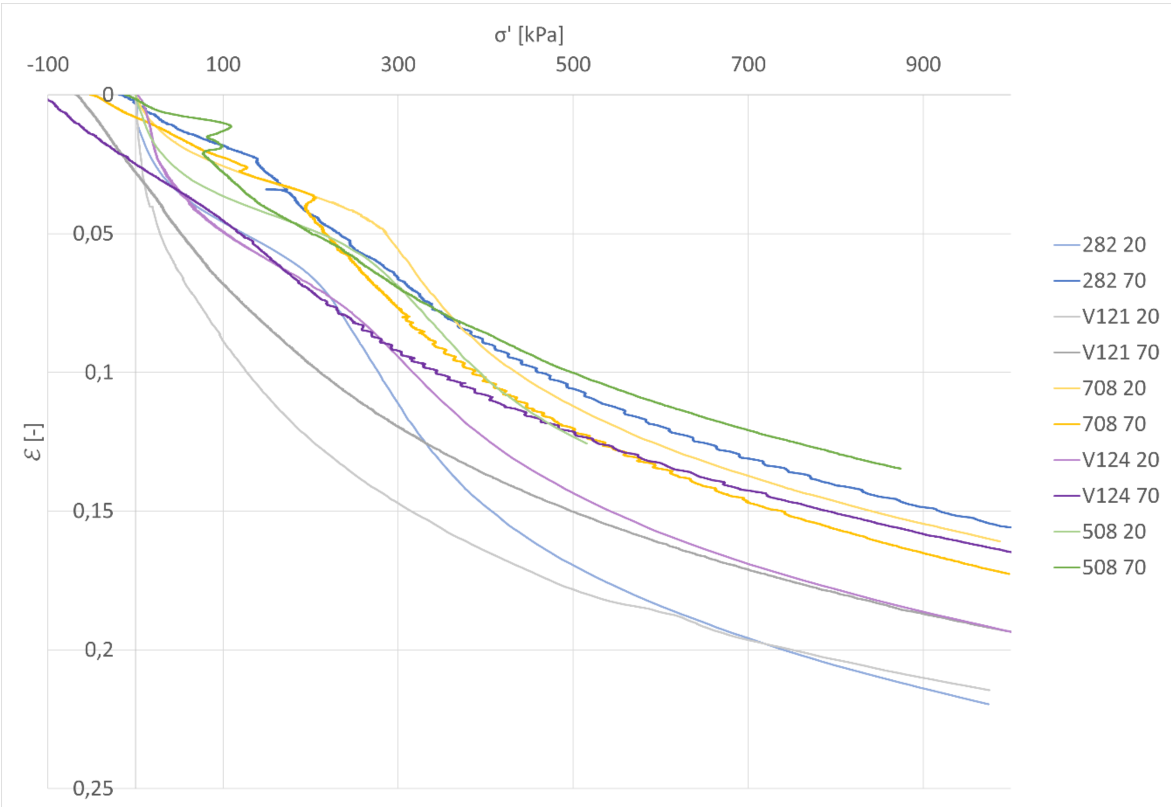


Figure 6-9: Effective vertical stress vs. strain

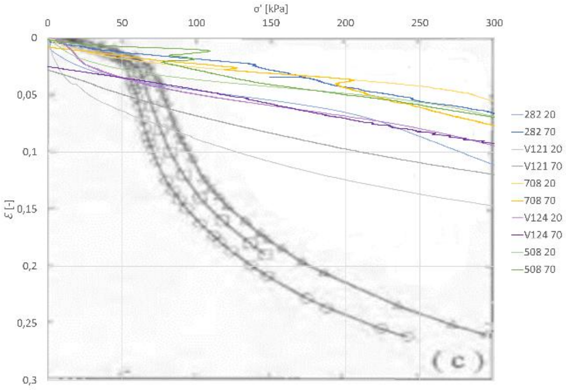


Figure 6-10: Comparison of stress vs. strain curve with Boudali et al.,(1994)

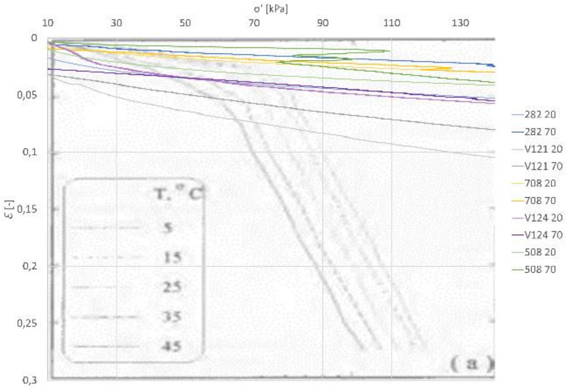


Figure 6-11: Comparison of stress vs. strain curve with Eriksson (1989)

Figure 6-12 shows the different preconsolidation pressures and liquid limits for the tests. Both temperatures are presented in the figure. The preconsolidation pressure after applying temperature is lower than the preconsolidation given at room temperature. The normalized values are given in Table 6-2, indicating that the new preconsolidation values are 29,3 – 56,8 % lower than the preconsolidation at 20℃. Which cooperates with the Boudali et al., (1994) idea of preconsolidation.

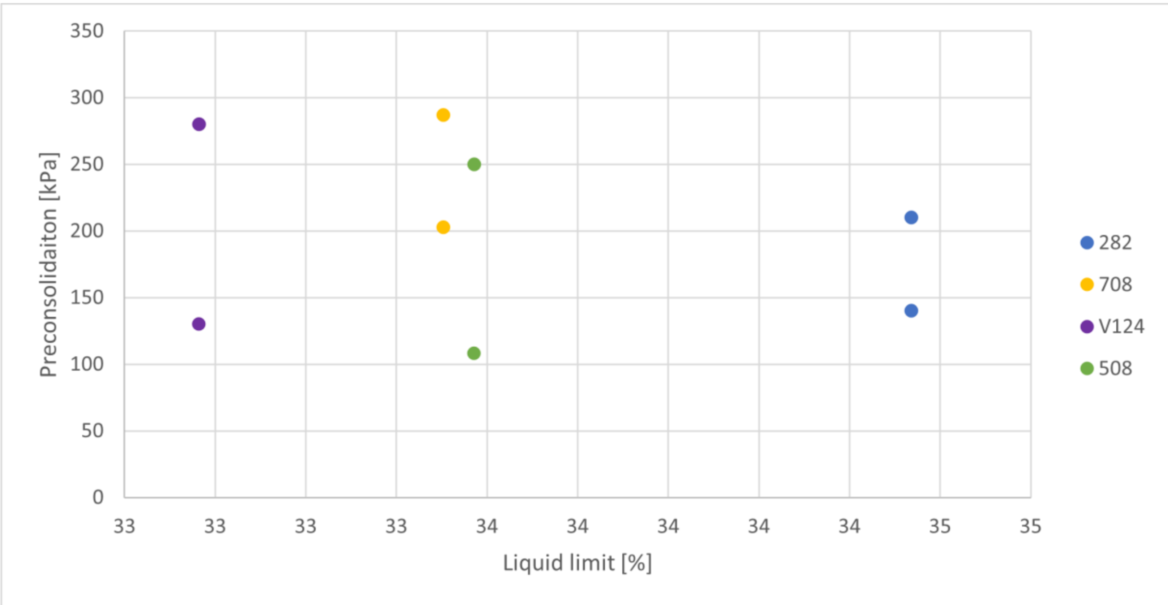


Figure 6-12: Preconsolidation pressure vs. liquid limit

Tidfors & Sällfors (1989) stated that "It is evident that the higher the liquid limit the greater effect the heat load has on the preconsolidation properties. For the most common clays in Sweden the effect is around 6 to 7 % per 10℃ temperature increase". This does not correspond with the illustration in Figure 6-12 nor the values from Table 6-2. The highest given liquid limit does not have the greatest effect on the preconsolidation properties given as the normalized value.

Table 6-2: Normalized preconsolidation pressure values and liquid limit

Test	Liquid limit	Normalized value $\frac{p_c(20°C) - p_c(70°C)}{p_c(20°C)}$
282	35	33.3 %
708	34	29.3 %
V124	33	53.6 %
508	34	56.8 %

6.2.2 Pore Pressure vs. Effective Vertical Stress

From the Canadian Berthierville clay presented by Boudali et al., (1994) the higher the temperature, the lower the pore pressure. Comparing Berthierville clay with Flotten clay, in Figure 6-14 the negative pore pressures from Figure 6-13 is visible. Even though, it is possible to observe the results from sample 708 and 508 both from the room temperature test and the 70°C test is observed closer to 0 kPa.

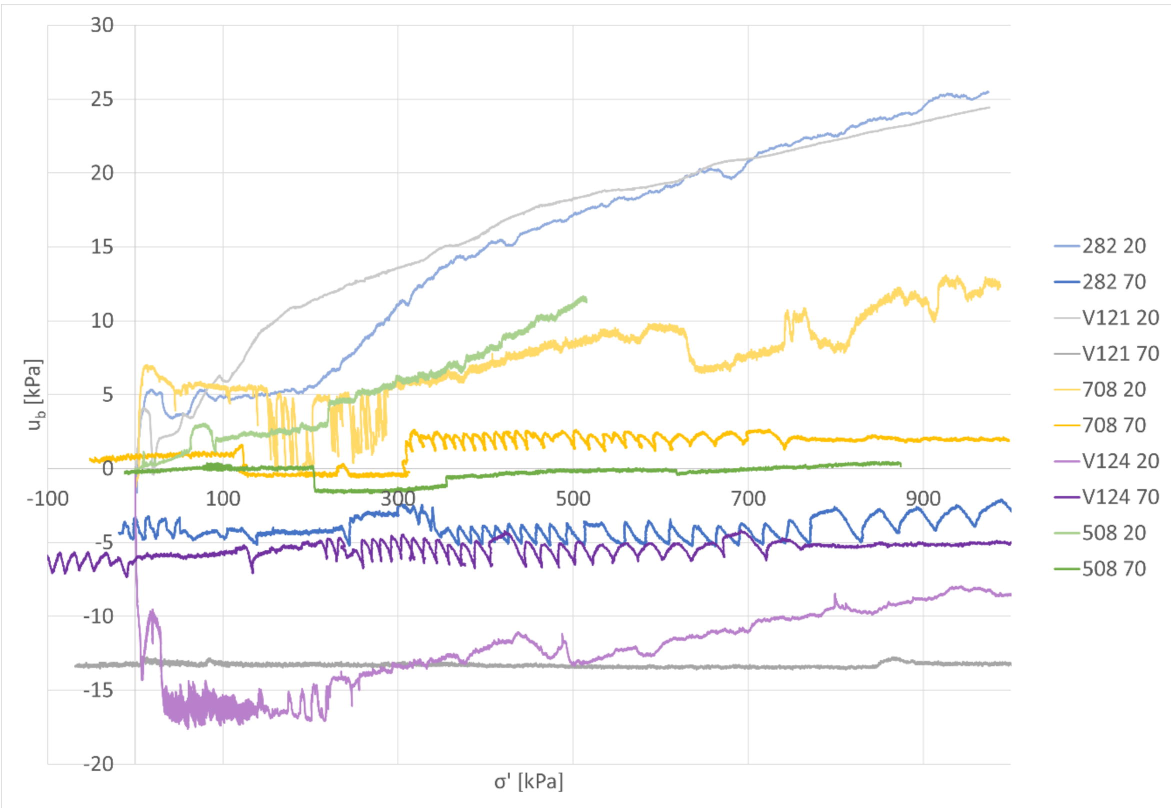


Figure 6-13: Effective vertical stress vs. pore pressure

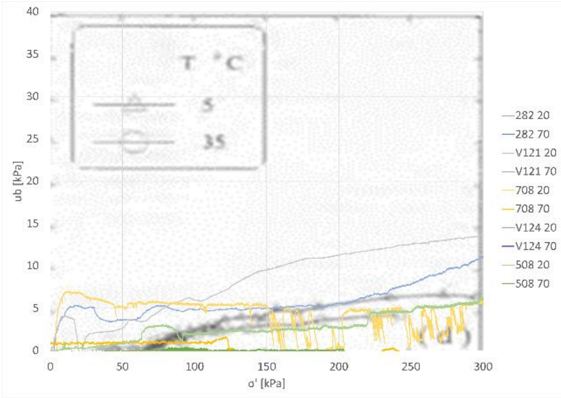


Figure 6-14: Pore pressure measurements at Flotten compared to Berthierville

Some of the pore pressures for the 70℃ tests are negative. It is important to notice that none of the pore pressures decreases significantly after the mechanical loading is applied. After the loading is applied the pore pressures decreases and then increases after reaching the preconsolidation pressure. When applying load there will be an increase in the attractive forces. It is worth noting that when detaching the sample after the testing, the specimen was not liquid, but had a firm texture.

6.2.3 Oedometer Modulus vs. Mean Effective Vertical Stress

The oedometer modulus for the overconsolidated area is lower for only half of the tests in the heated specimens, sample 282 and 708, shown in Figure 6-15. Most of the test, except the V124 sample, gives a lower modulus number for the normally consolidation area for the heated specimens, the soil gets softer when applying heat. The test heated to 70℃ for sample V124 has a higher M_{OC} and a higher m than the not heated sample at 20℃.

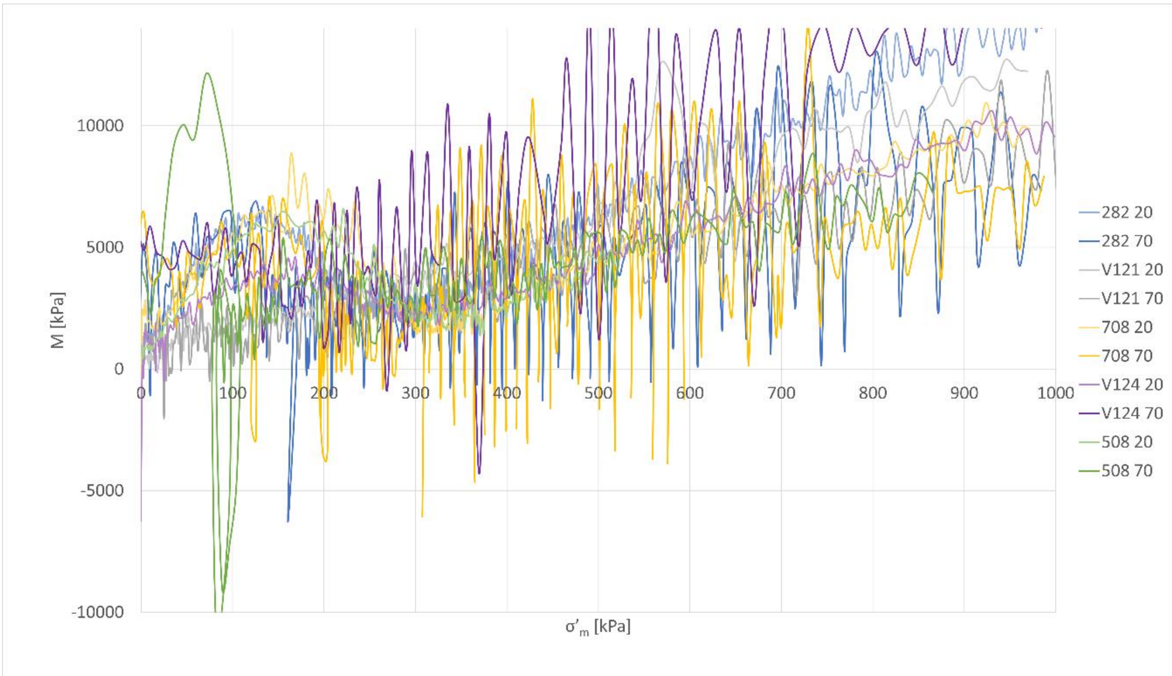


Figure 6-15: Mean effective vertical stress vs. oedometer modulus

Figure 6-16 shows the correlation between the modulus number and the depth. The modulus number for $70°C$ is only higher for the V124 sample compared to $20°C$ test. Figure 6-17 shows the correlation between the plasticity index and the normal compression index, λ, for the two different temperatures. Experimental studies with natural clays (Boudali et al. (1994), Tidfors & Sällfors (1989) and this study) shows that under a small effective stress, i.e., in OC range, the change in void ratio due to change in temperature is relatively small. Tidfors & Sällfors (1989) noticed that the effect of temperature seems to increase with the plasticity of the clay.

The change in void ratio in NC area is significant larger for the higher temperatures. With a larger λ there will be more compressibility in the soil and on the other hand a small λ gives a smaller change in specific volume, Δv, which indicates less settlements. Using equation 3.3 the modified λ can be calculated from the modulus number given in the NC area. The modulus number m in Table 6-1 indicates that the tests of $70°C$ as markable lower value of m than the tests of $20°C$, except for sample V124. In Figure 6-17 the largest values of λ are from the heated tests, which will give larger change in specific volume and void ratio.

However, looking at Figure 6-6, when applying heat, the modulus number increases for both $50°C$ and $70°C$ compared to the modulus number given at $20°C$. The depth of sample V124 is $9 - 10\ m$ and normally, the soil tends to become stiffer for deeper samples compared to shallow ones. This is because the overlying weight of the soil layers increases with depth, leading to an increased stress on the underlying soil layers. A high modulus number indicates that the soil is stiffer and less compressible. With a higher modulus number, the soil needs more load or stress to undergo the given amount of deformation. The modulus numbers for $70°C$ compared to the depth given in Figure 6-16 shows that sample V124 is distinguishes itself from the rest of the values. The grey line in Figure 6-16 classifies the other depths tested at $70°C$ to be soft soils. The shallower depths ideally, should have had a smaller modulus number. Looking at some explanations why sample V124 stands out could have been that the tests are executed on different depths. But the $20°C$ and $70°C$ tests are from the depths of $9.15 - 9.25\ m$, respectively. While the $50°C$ test shown in Figure 6-6 is from the depth of $9.35 - 9.4\ m$. Even thought, the depths do not differ much this might be the explanation why the modulus number from the $50°C$ test is higher than the $70°C$ test, as well as the temperature affection is $\Delta T = 20°C$. It seems like the $20°C$ test is somewhat correct looking at the other test in Figure 6-16, except for the shallower samples 282 and 708. However, it is difficult explaining why the high temperature tests do not follow the pattern of decreasing modulus number explained by Tidfors and Sällfors (1989).

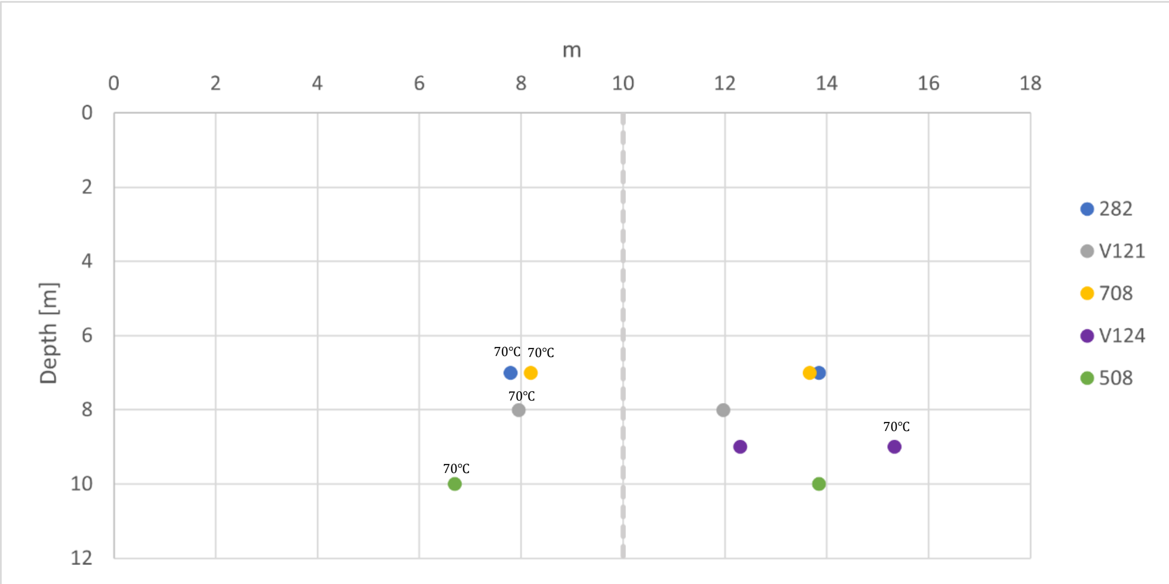


Figure 6-16: Modulus number vs. depth

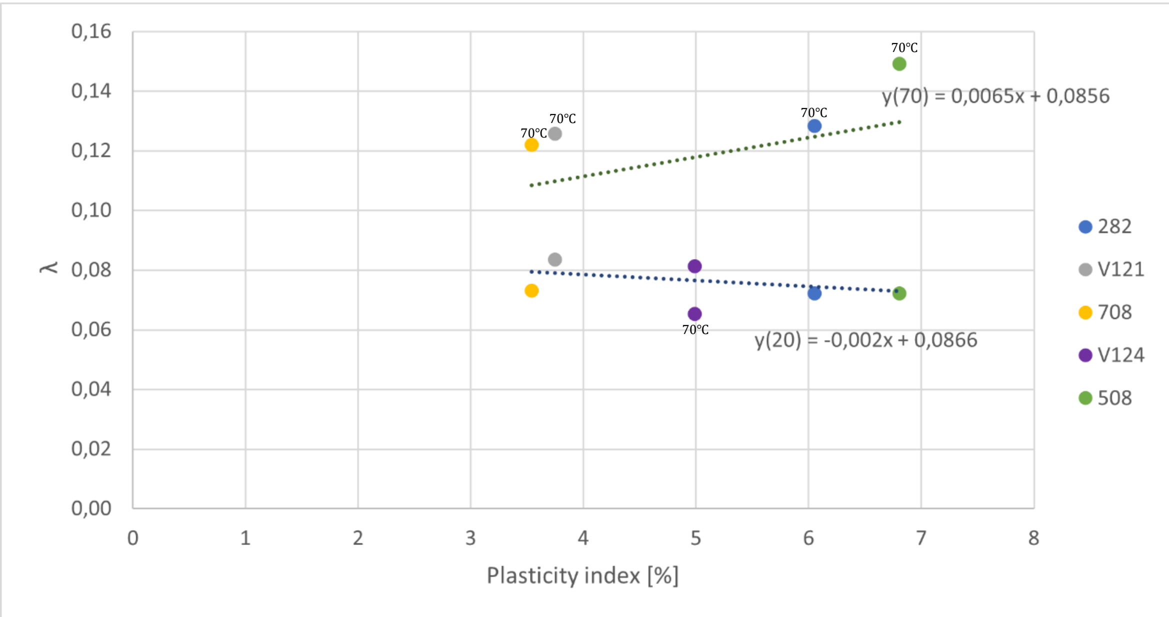


Figure 6-17: Plasticity index vs. λ

From Figure 6-4, the modulus number during 50°C for sample 708 have an increased value compared to 20°C which seems rather suspicious. It is predicted that the modulus number will decrease with applied temperature, hence the change void ratio and specific volume would increase. In the test ran for 50°C, for sample 708, it shows that the compression will decrease, making the soil less compactable. The high preconsolidation stress from the 50°C is high and pushed the trendline to be more conservative than the test from sample V124 provides. Even though, due to the lack of quantitative tests ran on 50°C the test was run one more time, and the results is shown in Appendix 7. The results and its outcome are further discussed later on in this chapter.

When temperature increases the effect of temperature seems to increase with the plasticity of the clay. The higher the temperature, the more different in void ratio. The inclination of $\lambda(I_p)$ for 70°C is 0.0065 while the inclination for 20°C is -0.002, given in Figure 6-17. Still, the test for sample V124 do not follow the same tendency. There can be several explanations for this, maybe some of the limitations such as there is no cutting device for fitting the clay in the oedometer ring.

The preconsolidation pressure is dependent on temperature. It needs to be further experimented on whether it is dependent on strain rate as well, in this experiment the clay is only exposed to the strain rate of $0.54\ \%/hr$. It is possible to ensure from the experimental tests that $\sigma'_p = f(T)$. From the theory given from Tidfors and Sällfors (1989) and Boudali et al. (1994) the preconsolidation pressure is supposed to decrease with increasing temperature. In Figure 6-5 from test V124 the preconsolidation decreases with temperature, with the given equation of $p_c(T) = -3.06\,T + 334.95$. But, for test 708 the preconsolidation pressure for 50°C increases significantly, Figure 6-3. Because of this there was a need of a new test to secure that the results were accurate. The redone test's results from depth $7.55 - 7.6\ m$ is shown in Appendix 7, where Figure 6-18 is extracted from the Appendix, and the redone test is marked in orange. The preconsolidation is approximately 160kPa which is not close to the preconsolidation given from the previous test. It is also worth mentioning that the strains are much higher for the $7.55 - 7.6\ m$ test than the $7.5 - 7.55\ m$ test. Looking in Appendix 7 the results from the depth of $7.55 - 7.6\ m$ is offset comparing them to the $7.5 - 7.55\ m$ test.

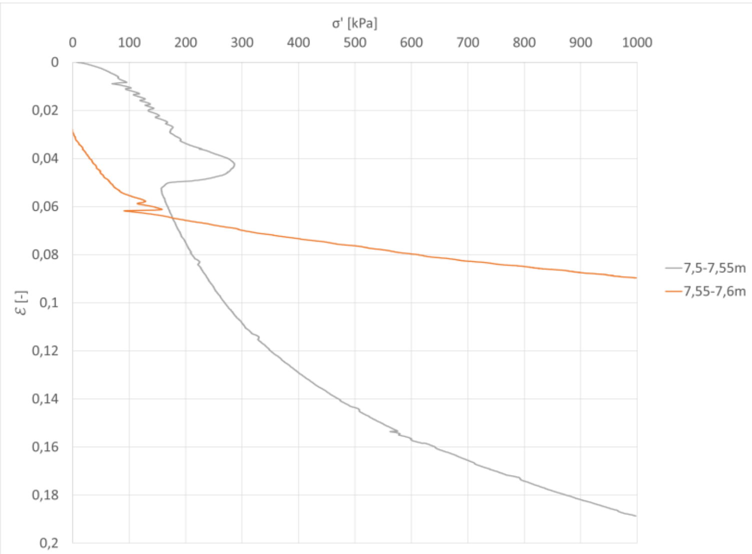


Figure 6-18: Sample 708, tests at 50°C

There are therefore huge uncertainties in the experiments. Even though Figure 5-16 (a), (b), (c) and (d) showed that the oedometers displayed similar graphs, it turns out that it is not the case when temperature load is applied. In the graphs in Appendix 7, significant differences can be observed among the various tests taken from depths 7.5-7.55 m and 7.55-7.6 m. To conclude, there is something wrong with the cell, maybe there is missing some type of calibration, change in mechanical or thermal applications etc. In previous chapters, calibration of temperature and pore pressure, as well as temperature and deformation, has been documented. However, as mentioned in chapter 5.4 SummaryLimitations there might be gaps between the ring and the specimen. This can cause a preferential flow path which can accelerate consolidation. The preferential flow is defines as water and its constituents moving by preferred pathways through a porous medium (Stumpp & Kammerer, 2022). This can make the soil stiffer because the soil becomes more compact. There is a possibility that the trimming of one of the specimens have left a gap between the ring and specimen that was unintentionally overlooked.

Researchers such as Yashima et al., (1998) stated that the preconsolidation pressure also depends on the strain rate and other viscoplastic parameters, m' and α. The viscoplastic parameter α is temperature dependent and is a defined in equation 4.9. α is determined as a function of a known preconsolidation pressure at a given temperature. The preconsolidation is presented in Table 6-3 and illustrated in a graph in Figure 6-19. With the actual values, from the experimental tests, given in the dotted lines. The idealized preconsolidation pressure given from Yashima et al., (1998) is not far from the trendline created from the experimental tests. The assumption that the modulus number can predict how the soil is behaving, the viscoplastic parameter α can also give an indication on the material's properties. By evaluating the viscoplastic parameters, the understanding on how the soil reacts to arise in temperature.

Table 6-3: Yashima et al., (1998) prediction of preconsoldation pressure by viscoplastic parameter α

	$\boldsymbol{p'_{cr}}$	$\boldsymbol{p'_c(70}$℃$)$	**α**	$\boldsymbol{p'_c(50}$℃$)$
282	210	140	0.32	157
708	287	203	0.28	222
V124	280	130	0.61	160
508	250	108	0.67	135

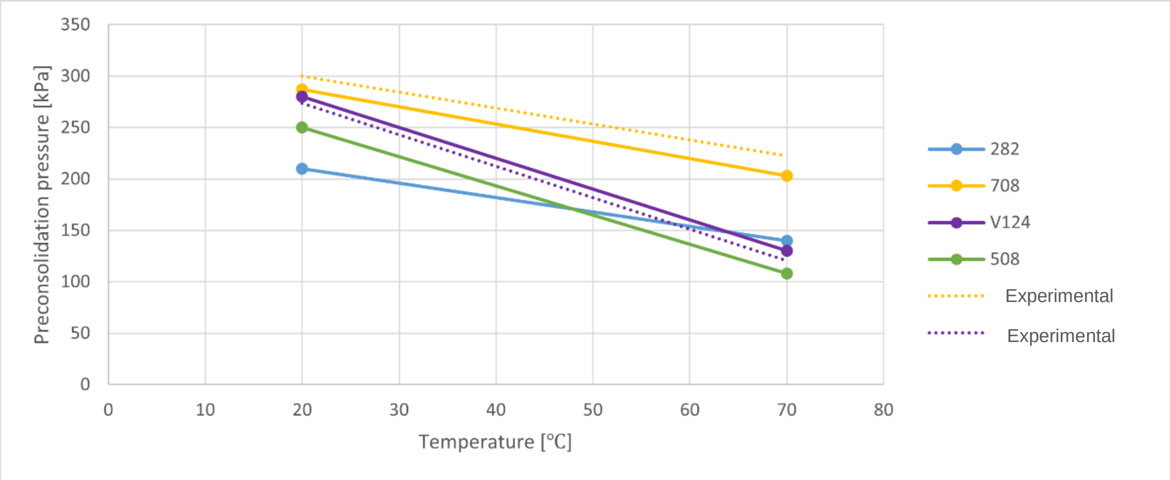


Figure 6-19: Temperature vs. preconsolidation pressure

6.2.4 Coefficient of Consolidation vs. mean Effective Vertical Stress

For the tests conducted at room temperature, a stable coefficient of consolidation is observed. However, for the tests conducted at 70℃, a constant value is not obtained.

The fact that a stable coefficient of consolidation is observed for the tests conducted at room temperature suggests that the soil under those conditions is exhibiting consistent consolidation behaviour.

On the other hand, the absence of a constant value for the tests conducted at 70℃ indicates that the soil's consolidation behaviour is affected by the elevated temperature. This could be due to various factors, such as changes in pore water pressure, soil structure, or the thermal properties of the soil.

Figure 6-20: Mean effective vertical stress vs. coefficient of consolidation

6.3 After Oedometer testing

To ensure that the heat from the TX150 has the same influence on the plastic frame as previous the testing another calibration was done. The result is given in Figure 6-21 where the temperature in the plastic frame is lower than the calibration test in Figure 5-9. The calibrated temperature of the plastic frame given in Figure 5-9 is 70°C given the clay a temperature of 74°C. While the calibrated temperature of the plastic frame during the testing of sample V124 is 62°C, which indicates that the temperature in the clay now is ± 67°C . This is clearly a source of error.

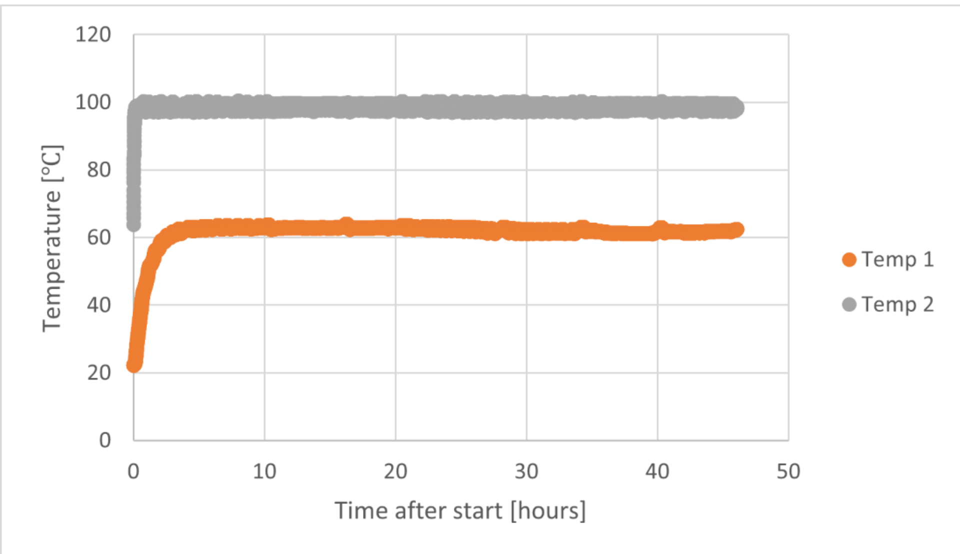


Figure 6-21: Calibration of temperature after testing

The results given in Appendices 2-6 might have a varying temperature in the clay of $\pm 7℃$. The tests using thermal heating from the TX150 up to $105℃$ were done in a given order shown in Table 5-5. GDS has an upgrade option to the GDS Constant Rate of Strain Cell with a temperature control. By using the temperature control upgrade feature, it may be that the temperature remains constant for all the tests conducted and that it provides approximately similar properties to the clay being tested.

Due to mixed results and many potential sources of error it is difficult to conclude whether it is possible to install GSHP in quick clay. Most of the clay samples tested in the experiment had a reduction in the preconsolidation pressure, and the clays became softer when arising the temperature. The negative pore pressure is a clear indication that the particles might not be in contact with each other. Because of the one-dimensional testing, before testing in-situ a triaxial testing phase is to recommend. In this way, the triaxial testing can provide more insight of how the soils three-dimensional behaviour.

7 Conclusions

In conclusion, the experimental results presented in this study provide insights into the behaviour of natural clays under thermal and mechanical loading conditions. During the heating phase, an increase in strain and a decrease in effective vertical stress were observed for most clays, indicating the influence of temperature on their deformation characteristics. Pore pressure exhibited fluctuations during the heating phase, suggesting the presence of physico-chemical forces at play.

The oedometer tests revealed that temperature affects the consolidation behaviour of clays, with a decrease in preconsolidation pressure observed with increasing temperature. A comparison with Canadian and Swedish clays showed variations in preconsolidation pressure and strain behaviour, indicating the influence of factors such as temperature, clay content, microstructure, and water content. The qualitative comparison reveals a high degree of concordance or agreement among the Canadian, Swedish, and Norwegian clays. The normalized preconsolidation values were found to be lower at elevated temperatures, supporting previous findings. The normalized preconsolidation pressure decreases for all experimental tests done in the study.

The changes in the strongly bound film of water and interparticle forces due to temperature variations can significantly impact the geotechnical properties of clays. The decrease in pore pressure during the heating phase and the observed changes in oedometer modulus and coefficient of consolidation suggest that temperature affects soil structure and water interactions, leading to alterations in soil behaviour.

It is worth noting that the behaviour of clays under temperature variations depends on factors such as clay plasticity. The effect of temperature on void ratio and specific volume changes was more pronounced for clays with higher plasticity. However, some deviations from expected trends were observed, highlighting the need for further experimentation and investigation.

Overall, this study contributes to the understanding of the thermal and mechanical behaviour of natural clays, emphasizing the importance of considering temperature effects in geotechnical engineering practice. The findings can aid in improving design and analysis approaches for infrastructure projects involving clayey soils subjected to thermal loading conditions. Further research is warranted to explore additional factors and conditions that may influence clay behaviour under temperature variations.

7.1 Recommondations for Future Work

- Further investigation into the discrepancies observed between the quantitative results and the characteristics of Canadian and Swedish clay samples. This requires a more in-depth analysis to understand the underlying factors causing these differences, such as an analysis of the apparatuses used.

- Employing advanced instrumentation and measurement techniques to enhance the accuracy and reliability of quantitative results, thus minimizing uncertainties associated with the instruments used in this study.
- Investigation into additional factors that may influence the relationship between temperature, liquid limit and preconsolidation properties in clays. This could include considering other geotechnical properties and environmental conditions to gain a more comprehensive understanding of clay behaviour.
- Analysing the influence of strain rate and thermal cycles on quick clay behaviour.
- Conducting comprehensive studies on diverse quick clays locations in Norway to expand the reach and generalizability of research outcomes.

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Appendices

Appendix 1: Index testing

Appendix 2: Results from 282

Appendix 3: Results from V121

Appendix 4: Results from 708

Appendix 5: Results from V124

Appendix 6: Results from 508

Appendix 7: Sample 708 at $50^{\circ}C$ from 7.5-7.55m and 7.55-7.6m

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