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Compression and swelling of cohesionless soils with organic content

Master's thesis in Geotechnics and Geohazards
Supervisor: Steinar Nordal
June 2023

Master's thesis

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Abstract

Almost 9% of Norwegian lands are covered with peat materials. The number of engineering projects within built-covered areas is increasing as the resource and infrastructure development continues to expand in Norway. Predicting settlement and deformations, especially in the long term, for structures built on sites where soils are mixed with peat poses significant challenges. Peat soils exhibit high compressibility, making settlement prediction complex. Moreover, removing peat from shallow surfaces is not considered environmentally friendly, and deeper peat layers may be impractical to excavate. This becomes further complicated as there are organic matter mixed with cohesionless soils which has more challenging laboratory investigations due to the difficult and costly intact soil sampling and also lack of proper well-fitting apparatus to granular samples to test compression or swelling in such materials. This study proposed a compressible ring oedometer to overcome shortage of conventional oedometers for cohesionless materials.

Nineteen samples were analyzed in this study, including samples constructed by the author and samples collected from the Klettelva road and bridge project, which is part from the new E39 road development in Betna-Hestnes, Norway. The examination primarily involved oedometer tests, utilizing both a conventional oedometer, and a newly proposed oedometer equipped with a compressible ring set. The aim was to enhance the understanding of how predominantly cohesionless soils with up to 10% organic matter behave. The oedometer testing yielded promising results in terms of stiffness and creep characteristics. Specifically, there was an observed increase in stiffness after preloading, indicating improved soil behavior. However, it is important to note that the assessment of creep during the study was based on a relatively short duration, leading to a rough approximation.

Preface

This report presents the culmination of research conducted as part of a Master's thesis in geotechnical engineering at the Department of Civil and Environmental Engineering at the Norwegian University of Science and Technology (NTNU) in Trondheim. The thesis was undertaken during the spring of 2023. Throughout this academic journey, invaluable guidance, support, and motivation were provided by Steinar Nordal, the thesis supervisor, and Reza Babadi, a Senior Geotechnical Engineer at Sweco, whose invaluable contribution served as the inspiration for this thesis. Gratitude is also extended to Geotechnic leader of Sweco, Andreas Roald, as well as the laboratory supervisors and the geotechnical workshop at NTNU, whose assistance was instrumental in the successful execution of the research and the development of the proposed oedometer apparatus.

Trondheim, June 30, 2023

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LIST OF ABBREVAIATIONS AND SYMBOLS

IL	Incremental loading
UIL	Incremental Unloading
RIL	Incremental Reloading
FROedo	Fixed Ring Oedometer
CROedo	Compressible Ring Oedometer
BP	Borehole Pint Number
OCR	Over Consolidation Ratio
LoI	Loss on ignition
E	Young's modulus
G	Shear modulus
M	Oedometer stiffness
m	Modulus number
r_s	Time resistance number
R	Resistance
K	Volumetric bulk modulus
OC	Over Consolidated
NC	Normally Consolidated
GSD	Grain size distribution
K_0'	Effective at rest earth pressure coefficient
a	Stress exponent
t_p	Duration of the primary consolidation phase
σ	Stress
σ_a	Stress equivalent to one atmosphere (in SI: 100kPa)

σ'	Effective stress
σ_{c0}	Preconsolidation stress
ε	Strain
ε_s	Secondary strain
ε_p	Primary strain
δ_i	Initial settlement
δ_p	Primary settlement
δ_s	Secondary settlement
ν	Poisson's ratio

Chapter 1 Introduction

The distinctive behavior exhibited by soil masses, as opposed to materials like steel, presents significant challenges in accurately characterizing these entities. Therefore, simplifications are often made to adequately describe and model soil materials. Engineers commonly rely on classical soil mechanics as a practical approach for designing and implementing constructions and infrastructures. This approach entails dividing design methods into two primary considerations: stability, which encompasses *shear strength* calculations, and settlement, which involves assessing *stiffness*.

Between them, settlement calculations require time-dependent parameters to accurately model the soil behavior. Here, a distinction can be immediately made between these two groups of calculations. Stability calculations typically deal with the final state and mobilized shear strength after large deformations, while settlement calculations deal with *modeling* the soil's actual behavior (or a try to ideally model the behavior) in the different stress situations. Moreover, while it is typically more convenient to find the failure strains, how the strain develops over time remains another problem for design purposes. In relation to this, defining an important parameter called *stiffness* is considered as the most important objective.

From classical mechanics, it is known that the stress and stiffness parameters directly govern the strains. For most geotechnical problems, constant stress over time is assumed for simplicity; however, this assumption may not necessarily reflect the real behavior of soils. Many different research have already understood that finding the stiffness values over time and with varying stress level poses a huge challenge. Therefore, many different relationships have been proposed to solve this issue, among which Janbu's method (Janbu, 1985) is considered as one of the most reliable methods.

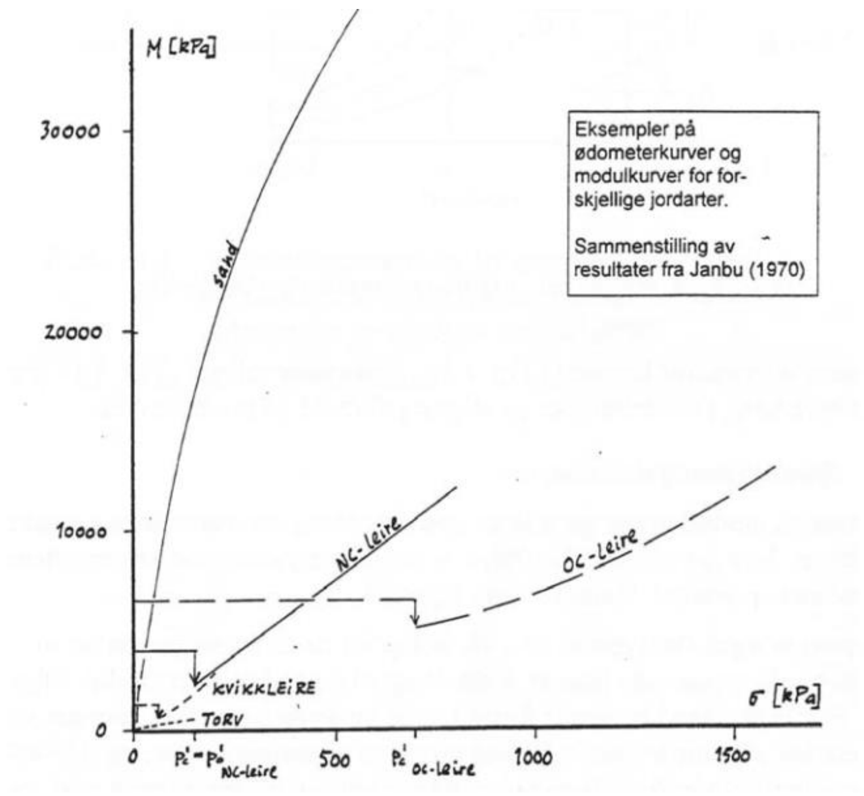


Figure 1-1 Graph showing the (oedometer) stiffness behaviors of different soil types. "M" (y-axis) shows the oedometer stiffness (defined in Chapter 2)

As seen in Figure 1-1, another factor that is significantly affecting the stiffness behavior is the *soil type*. However, one aspect common to almost all soil types, despite different stiffness pathways, is that the stiffness (M) changes in proportion to the stiffness (after loading a soil body). Ultimately, coarser grain materials tend to exhibit significantly stiffer behavior. Nevertheless, one notable conclusion from this simple loading result is that soils classified as *peat* (denoted as "*Torv*" in Figure 1-1) demonstrate considerably low stiffness values.

Due to its unique nature, Almost 9% of Norwegian lands are covered with peat materials (NIBIO, 2016). For infrastructure projects, it is usually the best practice to remove the top layer of peat. Nevertheless, the presence of thick peat layers at the outermost layer often indicates that the soil at deeper depths partially contains organic materials.

Problems with organic soils

Soils containing organic content are often less studied, and their engineering properties are hardly characterized, since the combination of soil type and organic type can create many possible outcomes. Nevertheless, these soils are primarily recognized for their problematic nature, primarily due to their high compressibility and lower shear strength under external loading. Assessing time-dependent parameters in such soils can pose additional challenges, often resulting in excessive settlement when establishing a foundation over them. The prevailing approach to mitigate these issues in many geotechnical projects involves removing the organic layer. Alternative measures may include employing techniques such as pile foundations or soil stabilization. However, it is important to note that most of these proposed methods often come with significant project costs. Furthermore, circumstances may arise where removing soil layers containing peat proves difficult due to factors such as depth, thickness, or other site-specific conditions.

Nonetheless, most of these methods arise from a lack of knowledge about the behavior of such soils. Existing guidelines address specific ranges of organic matter with high water content (500-1000%) (see Chapter 3); however, to date, there is a lack of information regarding lower organic content and lower water content ranges. Hence, there is a need for a better insight into understanding the behavior of peat and its contribution to soils, as well as achieving better documentation of the behavior of soils containing organic matter.

Interpreting soil behavior when mixed with organic matter is a challenging task, especially for cohesionless soils or those with low cohesion. This difficulty is mainly due to the challenges of obtaining undisturbed samples, which limits laboratory work to determine compressibility under appropriate stress history conditions. One commonly used laboratory test to analyze soil behavior is the oedometer testing.

Generally, the high compressibility and low bearing capacity of organic soils, the difficulty in removing peat layers in specific cases, the cost of projects, and the associated CO_2 emissions when removing peat layers are generally motivating factors for investigating the behavior of soils mixed with peat.

1.1 A real-case problem, construction of new E39 Betna-Hestnes

As mentioned, many Norwegian infrastructure projects are being affected by the presence of soils containing organic materials. One such project is the construction of the new E39 road between the cities of Betna and Hestnes in Norway (Figure 1-2). This project is currently being carried out by "The Norwegian Public Roads Administration" and is being consulted by the engineering firm, Sweco Norge AS. The project covers a wide range of construction types and encounters various ground conditions, which pose significant geotechnical challenges.

One specific aspect of the project involves building a bridge in the *Klettelva* area, near Henna in Heim municipality, Norway. Along with the bridge and the placement of foundations (designed with piles), a new road will be constructed approximately 6 meters above the existing terrain using embankments. In this area, the effective roadbed (the depth at which significant settlement occurs under the embankment or abutment) mainly consists of silty and sandy soil mixed with varying percentages of organic content, typically ranging from 2% to 10%. The depth of bedrock is relatively considerable (exceeding 15 meters). At lower depths, clayey materials with organic content have also been discovered. It is worth noting that although shallow layers of high-content peat have been removed, there are still areas where peat is mixed with the soil. The exact stratification for both the west and east sides of Klettelva can be found in Appendix A.

Figure 1-2 Location of the new E39 Betna-Hestnes (Statens-vegvesen)

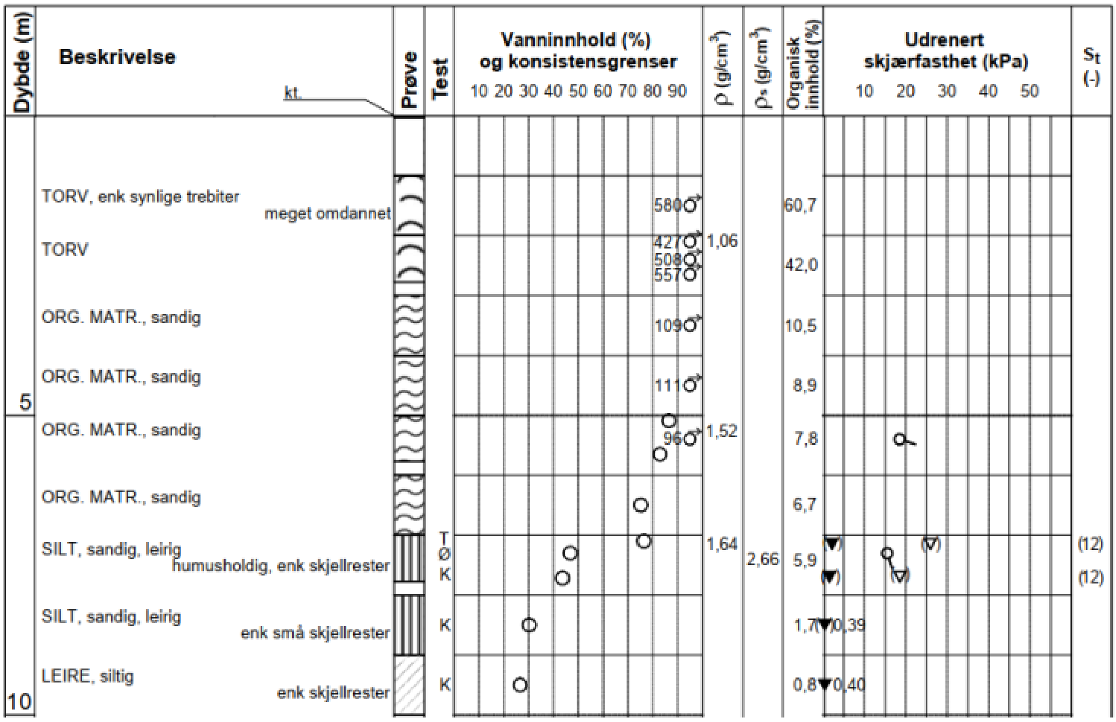


Figure 1-3 Example of laboratory results in the Klettelva area.

To address the challenges associated with the compressibility of the remaining material and the anticipated high settlement in the area, the primary approach taken was to construct the embankments stepwise and apply the preloading technique to remove excess deformations (refer to Table 1-1 for details). Due to the significant compressibility of organic-containing soils, implementing preloading in different phases has been deemed necessary to mitigate the expected compressibility and monitor the settlement at each phase of the project.

Table 1-1 Presumed stages of preloading for the current study

Phase no.	**Description**
1	Mass replacement 3 meters (from existing terrain)
2	Refilling 3m with blasted stone
3	Filling 3m with blasted stone (3m over existing terrain)
4	Filling extra 1.5m with blasted stone
5	Removing 3.5m of top
6	Refilling 2.5m light mass + 1m blasted rock

Additionally, pore pressure measurements using several piezometers have been conducted. Monitoring the pore pressure using gauges during the filling work will play a crucial role in regulating the progression of the project and tracking the settlement rate. By closely observing the changes in pore pressure, adjustments can be made to ensure that the construction proceeds in a controlled manner. Sampling and measurements conducted by Sweco during the construction phase revealed that the time for 90% consolidation is approximately 2-6 months, depending on the type of material, interpreted permeability, and the drainage pathways (either one-way or two-way).

However, it should be noted that these practices allow for continuous measurements to be taken during the construction process to monitor deformations and address any undesirable outcomes. Nevertheless, these measurements are limited to the construction phase, and the long-term deformations for the infrastructure remain unknown. The long-term deformations, referred to as *secondary settlement* or *creep* (as discussed in Chapter 2), introduce a degree of uncertainty in relation to long-term settlements in sandy masses containing an organic content of less than 10%.

1.2 Research objectives

This study aims to enhance the understanding of organic soil behavior, with a specific focus on sandy and silty materials mixed with organic matter, particularly peat, comprising less than 10% of the mixture. The primary objective is to investigate the properties of sand-peat samples and bag samples from Klettelva using an oedometer. A comprehensive geotechnical analysis will be conducted to evaluate the material and test results, with emphasis on both stiffness and time-dependent characteristics. Additionally, this research aims to evaluate the performance of a novel compressible ring oedometer by comparing its results with those obtained from the traditional oedometer.

Furthermore, one commonly employed technique to address settlement issues is preloading. In line with simulating the preloading stages of Klettelva, experimental tests will be conducted on cylinder samples obtained from the site to determine if creep occurs subsequent to preloading and assess the safety and reliability of the measures employed.

1.3 Limitations

One of the major challenges in understanding the behavior of cohesionless soils, especially during laboratory testing, is obtaining undisturbed samples. The subject material in this study can be divided into two categories. The first category consists of samples constructed in the laboratory at NTNU using available sand and peat, while the second category includes samples obtained from the Klettelva area in two forms of cylinder samples and plastic bag samples. It should be noted that both types of samples are disturbed, and obtaining undisturbed samples from such materials would require the use of alternative sampling methods, such as freezing the soil or using specialized sampling equipment like thin-walled tubes or pneumatic samplers, which can help minimize disturbance to the soil structure during the sampling process(Kim et al., 2018). However, the current project does not consider expensive specialized sampling equipment and methods or field tests such as standard penetration tests (SPT) or plate load tests, which are more precise methods for investigations, particularly for sand or cohesionless soils(Mahmoud, 2013). Therefore, the first limitation of this study is the use of samples built by the author.

The ignition method used to determine organic content is not claimed to be a reliable method for accurately determining peat content. While attempts have been made to use representative chunks of a sample for the test, other methods may be more suitable for checking organic content. In this research, only the loss on ignition test was utilized as a means of determining the peat content in the samples.

The laboratory tests in this study focus on understanding the behavior of sands/silts mixed with organic content under compression or swelling, and for this purpose, oedometer tests have been employed as a useful tool. However, it should be noted that the available oedometers are typically designed for conducting tests on clay, not sand. The primary issue is the sleeve friction that occurs when coarser particles come into contact with the oedometer ring. To address this issue, one proposed solution is to use a floating ring oedometer or modify an oedometer to make it better suited for testing sand samples. The latter idea has been implemented to propose a novel compressible ring oedometer (CROedo). It is worth mentioning that this study represents the first set of tests conducted using the proposed oedometer, and there is likely room for improvement in the future.

In the context of reviewing previous papers and guidelines, a common limitation is the lack of comprehensive guidelines for soils containing organic material, particularly when dealing with cohesionless soil masses with low levels of organic matter. While it has been suggested that soil with high organic content (over 6%) should be excavated in Norway, there is little guidance on how geotechnical engineers should proceed in other situations. This means that engineers must determine their own solutions or alternatives. Limited research has been conducted on soils with lower levels of organic content, with most of the available studies focusing on clays. Additionally, it is worth noting that existing research predominantly focuses on peat alone, while investigations into the behavior of sand mixed with peat have received comparatively less attention. In many cases, the prevailing approach involves the removal of these mixed layers as the primary solution. However, gaining a deeper understanding of soils containing low levels of organic matter can potentially lead to the development of alternative and environmentally sustainable solutions. By studying the characteristics of sand-peat mixtures, valuable insights can be gained, facilitating the exploration of more ecologically conscious alternatives.

1.4 Structure

The structure of this report is designed in a coherent manner to effectively achieve the objectives of the study. It commences with a background theory section that comprehensively covers the fundamental concepts related to consolidation, settlement, and oedometer testing. This theoretical foundation establishes the necessary framework for the subsequent analysis. Chapter 2 delves into background theory of oedometer testing within geotechnical engineering and will cover fundamental definitions and theories related to consolidation and generally oedometers. Additionally, it will provide an explanation of the structure of the new proposed oedometer with a set of compressed rings as well as methodology, limitations, and any related challenges associated with the proposed oedometer.

Following the background theory, a summary of the literature review is provided. Chapter 3 critically evaluates previous investigations conducted on soils containing organic matter, highlighting the existing knowledge gaps and areas that require further exploration. Additionally, the approach outlined in Norwegian guidelines pertaining to the subject matter is discussed, providing a context-specific perspective.

The report culminates in the presentation of experimental studies, which include detailed descriptions of the conducted tests, material, and methodology in Chapter 4. The obtained results are subsequently presented in Chapter 5. Through the comprehensive presentation of materials, methods, and subsequent test results in Chapter 4 and Chapter 5, the report ensures a systematic and clear depiction of the experimental study, facilitating the achievement of the research objectives.

Chapter 6 entails the comprehensive discussion of the results obtained from the oedometer testing across all experimental studies encompassing built samples by author, bag samples and cylinder samples collected from the Klettelva site. The focus of this Chapter is to analyze and interpret the findings derived from the oedometer tests, elucidating the implications and significance of the observed outcomes. This final section allows for the synthesis of the research findings and their alignment with the stated objectives, leading to meaningful conclusions.

Finally, Chapter 7 serves as the culmination of the research, where all the collected data and results are synthesized to establish a comprehensive understanding of the subject matter. This Chapter also provides potential avenues for future research and investigation.

Chapter 2 Background theory

2.1 Introduction

Soil displacements leading to settlement of the buildings can be considered as one of the most important areas of geotechnical engineering. To determine the settlement of a structure, the process involves calculating the *strains* caused by an increase in stress in various sediment layers, and then computing the compression of the sediment. Traditionally, this is achieved through *"oedometer testing"* on soil samples. Nonetheless, the complex behavior of soil is often a matter of debate for engineers and causes difficulty for expressing a proper relationship with respect to the total deformations.

2.2 Complex behavior of soils

The behavior of soil is significantly influenced by the magnitude and direction of stresses applied to it. Unlike conventional construction materials such as steel or concrete, soil exhibits a more complex and non-humogen behavior. Generally, as the stress level increases, the soil body becomes stiffer, which contrasts with the behavior of many other materials. It is important to note that this complexity is not limited to the existing stress level but also includes the soil's memory of its loading history, which can have a direct impact on its current behavior over millions of years.

Furthermore, the grain size distribution of the soil, whether it is fine-grained or coarse-grained, can affect its deformation properties. Fine-grained soils, such as clay, undergo a process called consolidation, where water is slowly squeezed out from the soil under load. As water is expelled, the soil particles come closer together, resulting in settlement.

On the other hand, cohesionless materials, like sand, exhibit a different settlement behavior compared to clay since they do not experience *significant* consolidation. Settlement in sand is influenced by various factors, including the type of sand, grain size, compaction, and initial density. Denser sands tend to have lower settlement.

In general, well-graded soils, which consist of a wide range of particle sizes, tend to exhibit lower settlement because the finer particles effectively fill the voids between the coarser particles. In contrast, poorly graded soils, where most particles have similar sizes, have a poor interlocking mechanism, and voids between the particles may exist. This characteristic can lead to higher settlement potential.

Understanding these aspects of soil behavior is crucial for geotechnical engineers in designing structures and predicting settlement to ensure the stability and durability of construction projects.

2.2.1 Soils with organic content

Apart from clay and sand, there are intermediate soils that exhibit more intricate behaviors, which are different from ideal clays and sands. These types of soils often have mechanical properties that are difficult to predict using existing theories developed for cohesive and frictional soils. Interpreting in-situ tests can also be challenging due to the partially drained behavior of intermediate soils. Therefore, geotechnical engineers must exercise caution and have the necessary training and guidance to properly understand and analyze these types of soils.

In addition, the composition of the soil and the presence of organic content can affect the friction and cohesion of the soil, resulting in unexpected behaviors. When the organic content is high, it can increase the water-holding capacity of the soil, leading to a reduction in its strength and stiffness. This is because organic material tends to absorb water, creating a lubricating effect that makes it easier for soil particles to slide over each other. Nevertheless, soils with high organic content are expected to experience greater settlement and deformation over time. This behavior arises from the fact that organic materials decompose over time, causing a loss of soil volume and a reduction in its load-bearing capacity. The decomposition of organic material can also release gases that cause soil particles to shift and settle, leading to further deformation. Therefore, when dealing with soils that have a high organic content, it is important to consider the potential effects on their geotechnical properties and deformation behavior, and to design appropriate measures to mitigate any potential issues.

2.3 Fundamentals of engineering soil settlement

When analyzing settlement, it is common to divide it into three main contributions: *initial*, *primary*, and *secondary* settlement.

The initial settlement (δ_i) refers to the settlement that occurs during the construction phase and is typically measured at the end of this period for practical purposes. Primary settlement (δ_p) refers to the settlement that takes place during the primary consolidation period, which is when the excess pore pressure is mostly dissipated. Lastly, the time-dependent settlement (δ_s) that occurs under consistent effective stresses is called secondary settlement, with creep being the dominant factor contributing to this type of settlement. Determining the boundaries between different parts of the soil in terms of size and time can be challenging and there may be some overlap between different settlement contributions, but it is still reasonable to separate them for practical purposes.

As mentioned, the deformation of a soil sample is dependent on both stress and time. The primary objective of experimental tests related to deformation is to determine the time and stress dependence of the soil sample. This is typically achieved through the oedometer test (which is defined more specifically in section 2.5). The results of the oedometer test are usually interpreted to determine primary and secondary settlement parameters, such as the modulus number (m), stress-dependent oedometer stiffness modulus (M), time resistance number (r_s), coefficient of consolidation (c_v), etc. for different soil types during both the primary and secondary settlement stages.

2.4 Oedometer testing

2.4.1 Oedometer equipment

Standard fixed ring oedometer

As mentioned, to understand the deformation characteristics of soils, an oedometer test is performed on a soil specimen to understand its deformation and settlement properties. The oedometer typically provides a one-dimensional state of deformation, shown in Figure 2-1, which is a simplification of reality, but at the same time is well adapted to the most common calculation models for settlement. These are mainly based on one-dimensional consolidation theory.

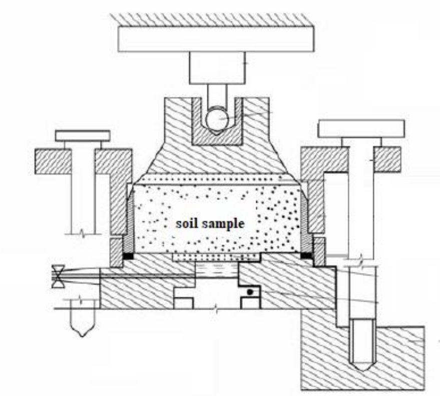


Figure 2-1 Principal sketch of oedometer-From (vegvesen Vegdirektoratet, 2022)

To conduct testing on soil samples, it is necessary to cut them into steel oedometer rings. The sample is then assigned a built-in area of 20 cm^2, while the standard height of clay and silt samples is 20 mm . Prior to shearing the sample, a thin layer of silicon oil is applied to the inner surface of the oedometer ring to minimize friction between the ring and specimen. To enable two-way drainage during testing, porous filters are typically positioned at both ends of the sample. Drainage channels are incorporated into both the base and top plates of the oedometer cell to ensure adequate drainage out of the filters. In certain types of oedometer equipment, it is also possible to measure pore pressure development at the specimen base. In such cases, only a small filter is placed in the base plate to facilitate communication between the sample and the pore pressure transducer.

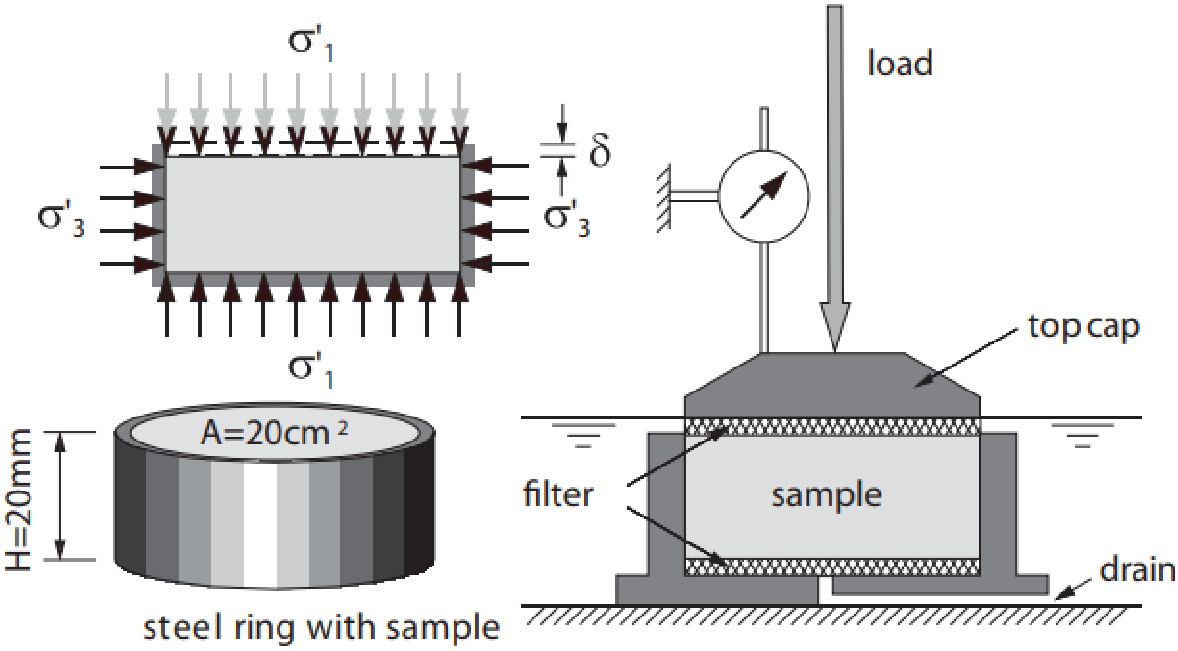


Figure 2-2 Cross section of a standard oedometer

Standard floating ring oedometer

This equipment is typically used for testing larger sand and coarse silt samples and is larger than the oedometer used for clay samples, with an area of at least 50 cm^2 (φ = 80 mm). The oedometer ring is reinforced to support the total load on the larger sample. One distinctive feature of this equipment, aside from the sample size, is that the oedometer ring is allowed to slide down the base on which it is mounted. During setup, the oedometer ring is supported by an outer installation ring to prevent the sample from moving. Once the soil sample is prepared and the top cap is mounted, the oedometer ring is released by rotating the installation ring, allowing it to suspend by friction between the ring and specimen. This decreases the distribution of friction between the sample and an ordinary steel ring, resulting in a more uniform distribution of total deformation over the sample height. This type of oedometer is often referred to as a "floating-ring" oedometer, based on the mechanical principle it uses. To prevent evaporation during testing, the oedometer cell is filled with water. If testing swelling materials, dry filters are recommended, but can be moistened once the current overburden stress, σ_{vo}, is reached. Prior to testing, the oedometer cell and dial gauge for recording deformation are mounted in the oedometer rig, and the cantilever beam is balanced. Contact between the loading piston and the sample is established, and all measuring devices are zeroed out. The load is then applied in a stepwise manner by adding deadweights to the loading plate, according to the chosen loading scheme. The exchange ratio on the beam may vary but is typically 1:10, meaning that 1 kg on the loading plate results in 10 kg effectively applied to the sample. Alternatively, hydraulic actuators can be used for loading, which can be programmed to apply the load, making it possible to operate during impractical hours without the need for additional staff.

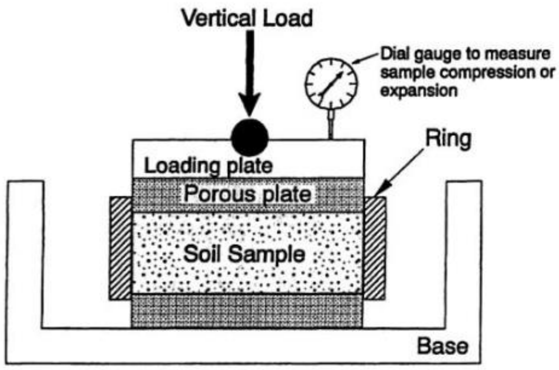


Figure 2-3 Sketch of **floating** *ring oedometer (Mazhar, 2009)*

The floating ring oedometer is a widely used equipment for testing the compressibility and consolidation properties of sand and coarse silt samples. However, like any testing equipment, it has some limitations and specifications that can be improved.

One of the limitations of the floating ring oedometer is the friction between sample and oedometer ring. The sliding oedometer ring is designed to reduce friction between the sample and the ring, resulting in a more uniform distribution of total deformation over the sample height. However, there may still be some friction between the sample and the oedometer ring, which can affect the accuracy of the test results. Furthermore, the load distribution over the sample is not uniform due to the presence of the oedometer ring. This can result in some areas of the sample experiencing a higher load than others, leading to non-uniform deformation and affecting the accuracy of the test results.

To improve the equipment and oedometer, some modifications can be made such as reducing friction between the sample and oedometer ring, increasing the measurement range of the equipment, and improving the load distribution over the sample. These modifications can help to increase the accuracy and range of the test results and make the equipment more suitable for testing a wider range of soils.

Compressible Ring Oedometer (Modified Oedometer)

There can be different ideas to modify the conventional oedometer to overcome some of its limitations and improve its performance. The biggest limitation when doing the oedometer test on granular and cohesionless soils is the friction between the sample and the ring which can be improved by reducing the surface of the ring that is in contact with the soil. for example, a type of coating like polytetrafluoroethylene (PTFE) coating on the oedometer ring or using multiple ring sets with compressible foam in between the ring set. Using a rubber oedometer ring to improve the load distribution over the sample can also be helpful.

The current study has proposed a new apparatus for oedometer testing which is more appropriate for cohesionless granular soils than conventional/standard oedometer. The presented compressible ring oedometer represents an attractive alternative when oedometer testing on cohesionless soils is in consideration. The principle and setup of the oedometer are as follows.

Setup:

The compressible ring oedometer is an innovative apparatus that was designed and developed by the author and supervisors at NTNU and constructed in collaboration with the NTNU workshop. This equipment is engineered to facilitate the compression of both the ring set and soil with minimal friction. The equipment incorporates several salient features, which are described below and illustrated in Figure 2-4 and Figure 2-5.

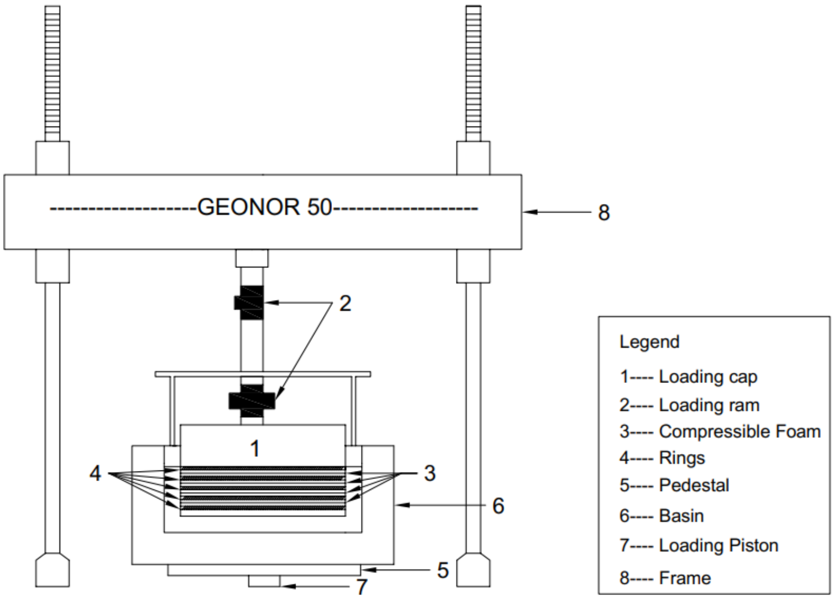


Figure 2-4 Sketch of the proposed compressible ring oedometer (CROedo)

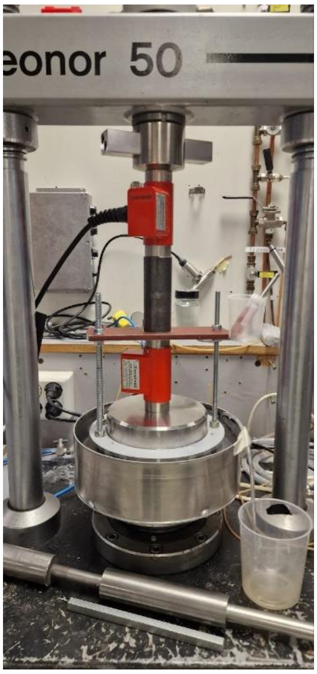


Figure 2-5 Setup of the proposed oedometer with a set of compressible rings

The specimen mold is devised to produce samples with a diameter of 150 cm and an initial height of 5 cm. The compressible ring set comprises five rings, and foams are affixed to both sides of each ring to enhance its compressibility. To ensure the alignment of the ring set and foam, three pins are incorporated to assist the operator. The ring set is positioned in a basin with a larger diameter, and the sample is kept saturated by pouring water into the basin. Drainage shears are implemented at the bottom of the basin, along with a space for the porous stone to act as a bottom filter. Additionally, two paper filters are positioned at the top and bottom of the specimen to expedite drainage.

To apply the load, two load cells are employed to exert force on the loading cap. The equipment features two loading caps: the outer loading cap applies pressure on the set of confining rings, while the inner loading cap contacts only the material. The load on the soil is indirectly determined by subtracting the force log of the total force from the force exerted on the ring set. The load cells and load caps are attached to a sturdy load frame that remains stationary throughout the testing process.

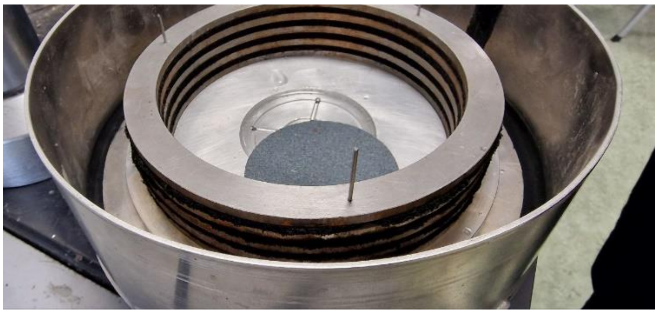


Figure 2-6 Basin, Ring set, Filter stone, and pins of the proposed Oedometer

To execute a test, the ring set, and foams are aligned and placed in the basin or water bath. The sample is subsequently built inside the ring set, and the setup is placed on a pedestal to commence the testing process. The pedestal propels the entire setup upwards, while the load cells register the force. The axial force is measured by a load cell, and the whole load frame ascends frictionlessly. The applied load can be varied by a program on the computer.

While the friction between the specimen and oedometer ring set may slightly affect the measurement, the equipment's accuracy provides valuable results. The vertical strain is measured by a displacement transducer with an accuracy of 0.5 micrometer, which is situated between the base plate and the top cap of the soil specimen to eliminate the impact of load frame deformation.

Limitations

Over time and with repeated use, the smooth surface of the cut foam between the ring set may experience degradation. It was found that the primary requirement for the foam material used between the ring set was to possess sufficient compressibility during loading, while also exhibiting good memory to return to its original state upon unloading. Two types of foam were tested for compressibility (see Figure 2-7 and Figure 2-8) The first foam (Type A) experienced quicker degradation and was thus replaced by a second type (Type B) that exhibited better durability and compressibility, while also having the ability to return to its initial state. Further research can be undertaken to identify a more suitable foam material for the ring set, such as other durable memory foams. For example, polyurethane foam can be a good alternative since it is engineered to be viscoelastic, meaning it has both viscous and elastic properties. It may be of interest to evaluate the impact of foam type on the results of the compressible ring oedometer, as any significant effect could warrant further investigation. However, it should be noted that the present study does not address the potential impact of foam type on the results.

Figure 2-7 Foam TypeA

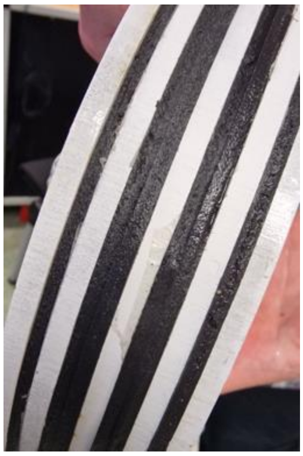


Figure 2-8 Foam Type B and rings

In the initial stages, the foam thickness was lower(3mm). In order to enhance the durability and longevity of the foam, a decision was made to augment the thickness by doubling the number of foam layers between each ring. Furthermore, it is crucial to acknowledge the necessity of accurately positioning and aligning the loading cap on the samples, specifically at the center. To achieve this, indicator pins were utilized as a means to ensure proper alignment among the rings and accurate placement of the load cap.

2.4.2 Test procedure

Load sequence

In an oedometer test, different load levels are applied to a sample over time to measure different deformation-related properties. The two most common approaches to oedometer testing are *incremental loading* (IL) and continuously increasing loading tests (CRS). The former will apply load to the specimen in incremental steps, starting from lower load levels to higher ultimate load, while the latter consists of applying a continuous load depending on the chosen constant parameter, which could be either the strain, pore pressure ratio or pore pressure gradient. Nevertheless, the constant rate strain (CRS) is more typically applied in Norwegian practice especially for clays. In the current study, incremental stepwise loadings have been performed.

Load duration

The outcomes of the oedometer test can be impacted by the selection of load steps and their duration. Historically, the conventional approach has been to employ modest load increments (ranging from 5 to 10 kPa) and then double the load for every subsequent step. This leads to elevated stress levels in a small number of load steps. The deadweights that accompany the oedometer equipment are typically adjusted accordingly. A stress sequence of 12.5 - 25 - 50 - 100 - 200 - 400 - 700 - 1200 kPa may be used as a standard. However, there may be situations that necessitate deviating from this pattern. For instance, to obtain a reliable determination of the preconsolidation stress σ_{c0}, supplementary load steps can be introduced near the expected location of σ_{c0}. Alternatively, for testing delicate, strain-softening clays (such as quick clays), it may be necessary to employ intermediate load steps to prevent structural collapse and sample squeezing between neighboring load steps (at high stress levels). In such scenarios, the issue can be mitigated by using filter paper between the sample and top cap.

To ensure an accurate determination of deformation parameters in the virgin stress range for a specific practical problem, it is recommended that the maximum load level during testing covers the stress range adequately. This is especially crucial when the results will be used for structures subjected to high or alternating loads on the ground, such as grain silos. Based on the results of the preliminary study using incremental loading, it was determined that the ultimate load of 1200kPa may be considered as relatively high. To improve the assessment and comparison of outcomes in real-life scenarios, this study has prioritized load increments of up to 600kPa during sample construction. This decision stems from the preliminary investigation, which revealed instances of structural collapse and sample compression between adjacent load stages under high stress levels.

To achieve each incremental step in loading within the compressible ring oedometer, CRS approach was utilized in between the load increments to minimize sudden changes between load steps and increase the precision of load application. In simpler terms, the tests began with zero stress and gradually increased the load in a consistent ratio until reaching the first load increment. The load was then maintained for a specified duration before increasing again to reach the next load increment. This process of gradually increasing the load and holding it at each increment was repeated throughout the tests.

It is also noted that different laboratories use alternative loading procedures. For instance, at the Norwegian Geotechnical Institute (NGI), a load procedure involving multiple unloading and reloading loops is utilized, where the sample is loaded to around $9_{\sigma c0}$ with a load step duration of 2.5 hours. Deformation development during unloading is often recorded, typically applying double load steps and shorter load duration. Moreover, for built-in samples of sand and silt, shorter load duration and equal load steps are common, with the load sequence tailored to the physical problem (e.g., 50 kPa load steps, 5 min. duration). The precise load increments and their duration for the current experimental study will be presented in detail in Appendix A.

2.5 Basic theory of geotechnical parameters in oedometer testing

The fundamental principles of the basic settlement theory will be briefly discussed below, as a means of providing a framework for conducting experiments (oedometer testing) aimed for determining the time and stress dependent deformation. Typical oedometer results from different soils are shown in Figure 2-10 and Figure 2-10.

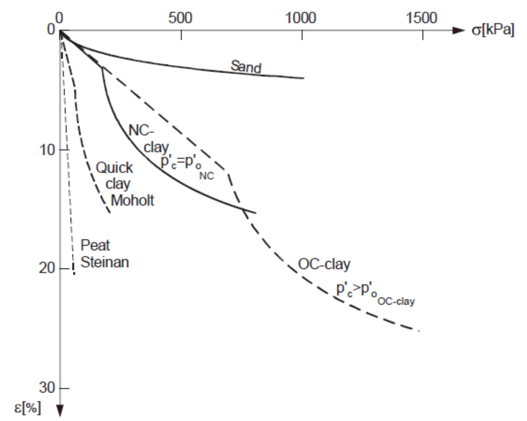


Figure 2-9 Expected results or stress-strain curve from oedometer test for different material.

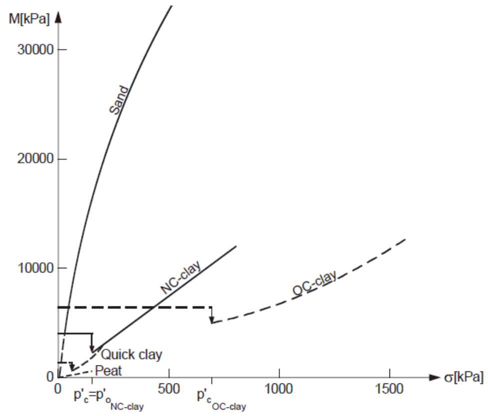


Figure 2-10 Expected results for oedometer stiffness from Oedometer test for different material.

2.5.1 Fundamentals of stress-dependent properties of soil (Stiffness)

This section provides an overview of the stress dependent properties of soil, with a focus on stiffness. Stiffness is a fundamental property of soils that describes their ability to resist deformation under an applied load. It is an important parameter in geotechnical engineering, as it governs the behavior of soil structures subjected to external loads.

Stiffness parameters are determined through laboratory and field tests and measurements. In the laboratory, the parameters can be obtained from both oedometer and triaxial tests. The most relevant parameter determination is expected when representative stresses and stress changes for representative points in the ground are selected.

The one-dimensional oedometer modulus (M) can be often used to express the stiffness for a test sample in oedometer conditions. This measurement has practical implications, as when the load on a sediment is significantly greater than the depth of the layer, and there will be no lateral strains. The oedometer modulus (M) can be determined using conventional isotropic elasticity parameters, such as Young's modulus (E) and Poisson's ratio(ν), or alternative parameters, including volumetric bulk modulus (K) and shear modulus (G) in the following manner:

Equation 2-1

$$\varepsilon_1 = \frac{1}{E}(\Delta\sigma_1' - \nu\Delta\sigma_2' - \nu\Delta\sigma_3') \;\&\; \varepsilon_3 = 0 \rightarrow \Delta\sigma_3' \frac{\nu\Delta\sigma_1'}{1-\nu} = K_0'\Delta\sigma_1'$$

Where:

K_0' = effective at rest earth pressure coefficient= $\frac{\nu}{1-\nu}$ for drained condition.

By using the definition on the deformation modulus and simplifying the expression, one might simply obtain the oedometer modulus by taking the tangent of the effective stress-strain graph, which the equation is given as Equation 2-2:

Equation 2-2

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

The oedometer modulus(M) is influenced by the stress level and has a similar stress level dependency as the parameters E, ν, K, and G, since they are related to the oedometer modulus (Equation 2-3).

Equation 2-3

$$M = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} = K + \frac{4G}{3}$$

By presenting a broader interpretation of M that applies when $\varepsilon_3 \neq 0$, one can utilize the primary stress correlation to derive the subsequent formula:

Equation 2-4

$$M = \frac{E}{1-2\nu K'}$$

The stiffness of a material known as M, rises as K increases, leading to a firmer reaction of the soil in one-dimensional behavior, which is subject to how firmly the sample is fixed laterally. It is presumed that the soil is perfectly elastic, resulting in reversible and linear deformations. Nonetheless, real soils often do not conform to ideal elasticity theory, and exhibit permanent or plastic deformation, causing material behavior to deviate from linear behavior.

M can also be called the compression modulus, and the swelling modulus, in loading and unloading, respectively. Typical development of the tangent modulus M with effective stress together with stress-strain curves from oedometer tests are plotted in Figure 2-11 in different types of soil:

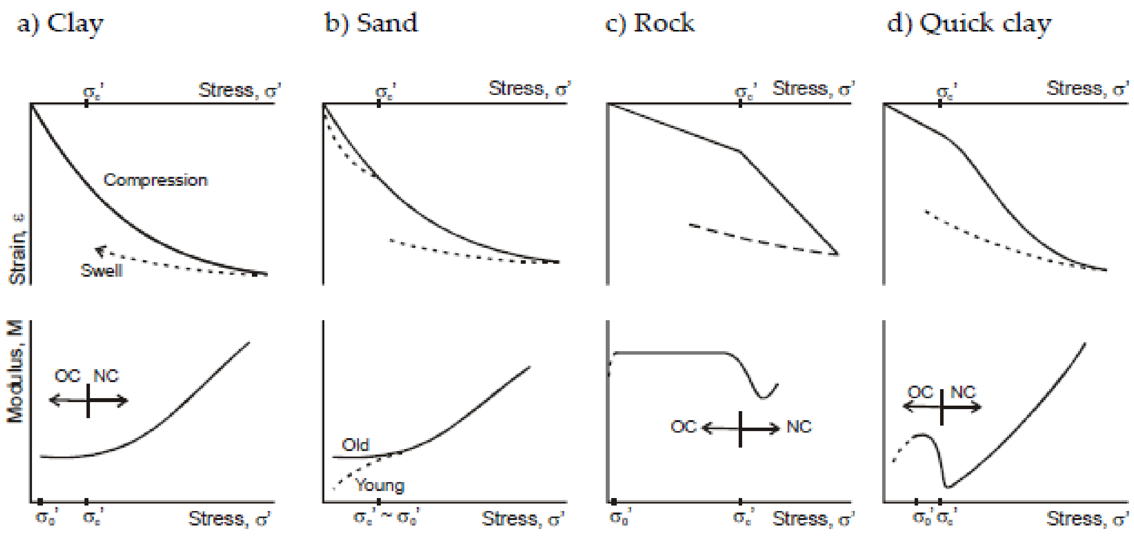


Figure 2-11 Typical stress strain and modulus curves obtained from Oedometer tests (Havel, 2004)

Four sets of diagrams represent: a) an over-consolidated clay, b) a silty sand at in-situ porosity, c) an intact sample of a cemented moraine, a shale or sedimentary rock and d) a sample of the intact, fairly undisturbed quick clay, or an extra-sensitive soil in general, with loose, porous structure, easily collapsible for increasing stress around preconsolidation pressure (Janbu, 1998). All diagrams are plotted in arithmetic scale.

Generally, the stiffness of a soil is a factor of its memory and its current stress level. If the current stress level of the soil does not exceed its past maximum stress level, the soil will remain in an elastic range and will behave stiffer, and if the current state is the situation where maximum stress is occurring, the soil will behave elastically until the point it reaches its past maximum and then a strain softening behavior is expected for samples.

The stress-dependent relationship between the deformation modulus and various types of soil was expounded by Janbu in 1963. He expressed the generalized behavior by the following mathematical expression (Equation 2-5). The concept of the constraint modulus (M) by Janbu, may serve as a unified basis for practical estimates, in which the tangent modulus (M) is as a function of the effective stress. The concept goes as follows:

Equation 2-5

$$M = m\sigma_a \left(\frac{\sigma'}{\sigma_a}\right)^{1-a}$$

Equation 2-6

$$\varepsilon = \frac{1}{a.m}.(\frac{\sigma'}{\sigma_a})^{a}$$

Where m is the modulus number dependent on soil type and stress level, σ' is the actual effective stress level, σ_a is the stress equivalent to one atmosphere (in SI: 100kPa), and *a* is called the stress exponent. This equation states the stiffness parameter (M) is a factor of stress level (σ') and the soil type. For Normally consolidated materials (NC) a = 0, and for over consolidated material (OC) materials a = 1.

For granular soils, tests on sand have shown a stress exponent close to 0.5 will represent the stress-strain curve quite well in most cases. Thus, for simplicity a stress exponent of 0.5 has been used as a standard value for sands and silts. Current study has relied on the recommended stress exponent, 0.5 for both sand and mixture of sands and peat.

Equation 2-7

$$M = m\sigma_a \left(\frac{\sigma'}{\sigma_a}\right)^{1-0.5} = m\sqrt{\sigma_a\sigma'}$$

Equation 2-8

$$\varepsilon = \frac{2}{m}\sqrt{\frac{\sigma'}{\sigma_a}}$$

As mentioned, the typical range of soil stiffness varies widely depending on soil type, stress level, and loading duration. For example, stiff clays can exhibit stiffness values on the order of 10-100, while loose sands may have stiffness values on the lower orders. In general, soil stiffness decreases with increasing stress level and loading duration, which is known as stress softening or strain softening behavior. Typical ranges of modulus numbers for different soil types are as follows:

Table 2-1 recommended values for modulus number in different soils (Bowles, 1996; Budhu, 2010; Coduto, 1999; Das, 2021)

Material type	**Modulus number (m)**
Clayey soils	10-100
Sandy soils	20-200
Silty soils	20-150
Gravelly soils	50-500

Since the study focuses more on cohesionless soils, the typical ranges of modulus numbers for specifically sand presented in Table 2-2.

Table 2-2 recommended values for modulus number in sand.

Material type	**Modulus number (m)**
Loose, fine sand	<150
Medium dense sand	150<m<250
Dense, coarse sand	>250

**Oedometer Modulus for sand, when unloading, is typically 3-5 times higher than loading.*

It is important to note that these values are general ranges and may vary depending on the specific conditions of the soil and the project requirements. Site-specific testing and analysis are usually necessary to determine the actual modulus values for a particular soil type.

2.5.2 Fundamentals of time-dependent properties of soils

The time-dependent behavior of soil can be quantified by the time resistance or the resistance number, which are key parameters in geotechnical engineering. The time resistance is defined as the ratio of the change in effective stress to the logarithm of time during the secondary consolidation stage. The resistance number, on the other hand, is defined as the ratio of the change in effective stress to the square root of time during the primary consolidation stage.

The time resistance of a soil refers to its ability to resist deformation over time under a constant load. This property is often quantified using the concept of creep, which is defined as the time-dependent deformation of a soil under a sustained load. Creep behavior is typically characterized by the relationship between stress and strain over time and is commonly modeled using empirical or semi-empirical models.

The resistance number is another important parameter that characterizes the strain time-dependent properties of soil. It is defined as the ratio of the final stress to the initial stress after a specified time of loading. The resistance number can be used to predict the long-term behavior of soils under different loading conditions and is often used in the design of geotechnical structures.

In this section, underlying principles that govern the time resistance and resistance number of soils, as well as the various factors influencing the behavior, will be explored.

When a soil body is subjected to surcharge load, excess pore pressure will be generated within the soil. The dissipation rate is related to the type of soil and in granular soils is much faster than clay where particles have more coherent structure. As the excess pore pressure starts to dissipate, deformations will start to build up and cause settlement in the soil. This type of settlement is often referred to as *primary consolidation* and is more important in the behavior of fine-grained soils (clay) compared to sand.

Secondary compression, also known as creep, continues after the conclusion of primary consolidation where excess porewater pressures have been entirely dissipated. The driving mechanism behind the long-term settlement is completely different to that for primary condition. The long-term deformations are mainly due to creep, developing at an approximately constant effective stress level. It is thought to be caused by the rearranging and reorientation of mineral particles of the soil. Peat is characterized by significant creep, which may be a result of a continued breakdown of fibers over time.

This study aims to examine the primary and secondary consolidation behavior of sand and silt. It is important to note that the primary consolidation period for the material under investigation is relatively short, and the behavior of the material can be described as drained. As a result, the pore pressure generated during primary consolidation dissipates rapidly, and any associated changes in pore pressure can be disregarded when measured using an oedometer.

To study the secondary deformation process and the long-term creep settlements, the concept of time resistance can be formulized as in Equation 2-9, in which the soil strain-time behavior and the basic definition of the time resistance (R) has been defined.

Equation 2-9

$$R = \frac{dt}{d\varepsilon}$$

This can be regarded as the reciprocal of the rate of strain.

In one-dimensional compression, the time resistance of most granular materials typically increases over time. However, for a certain period after the start of consolidation, denoted as t_0, the relationship between the time resistance (R) and time (t) becomes linear. Empirical observations have shown that t_0 is generally much smaller than the duration of the primary consolidation phase (t_p). This suggests that intergranular shear stress undergoes creep and induces shear strains only during the period from t_0 to t_p. Consequently, this behavior has significant implications for the design and analysis of geotechnical structures subjected to time-dependent loading, such as long-term settlement of foundations or creep behavior of retaining walls. Accurate prediction of the time resistance and its evolution over time is essential for ensuring the stability and safety of such structures.

The linear relationship between time resistance R and time t, which is observed after the crossing of time t_0, can be characterized by the creep resistance number r_s (also known as the creep resistance or creep number). A considerable number of oedometer tests have demonstrated that the creep resistance number is dependent on the level of effective stress. This parameter, along with the time resistance, is an important factor in the quantification of secondary compression, or creep.

Figure 2-12 provides a visual representation of the methodology employed in obtaining these parameters, and their corresponding equations are presented. It should be noted that ε_s denotes the secondary settlement. To find the total strain of a soil element, one shall notice that $d\varepsilon = \varepsilon_s + \varepsilon_p$, where ε_p is the primary strain at $t=t_p$. The term "primary settlement" refers to the primary strains (ε_p) that occurs in a soil mass when a load is applied to its surface. This type of settlement is mainly caused by the compression of air voids and the rearrangement of soil particles under load. It should be noted that primary settlement can result in the initial compaction and consolidation of the soil, while secondary settlement can lead to further densification and strengthening of the soil.

Equation 2-10 $R = \frac{dt}{d\varepsilon} = r_s t$

Equation 2-11 $\varepsilon_S = \frac{1}{r_s} ln \frac{t_2}{t_1}$

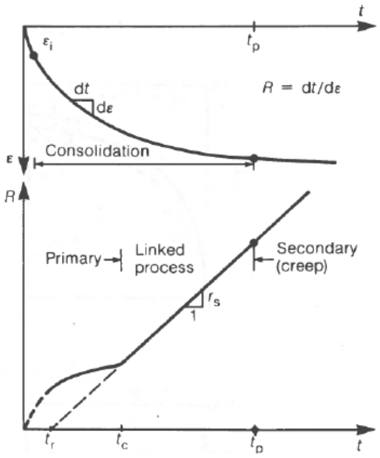


Figure 2-12 Derivation of time resistance and time resistance number (NTNU, 2015)

Table 2-3 provides a recommended range for the time resistance number, which is defined as the slope of the linear segment of the stress-strain curve that occurs after the completion of primary consolidation under load.

Table 2-3 Anticipated range of r_s for different soil types (NTNU, 2015)

Material	NC-Clay	OC-Clay	NC-sand
r_s	100-500	1000-5000	1000-10000

In unloading, when the time resistance (R) of soil becomes negative, it indicates that the soil is swelling. The magnitude of the negative time resistance value can be used as an index to quantify the degree of swelling. This negative time resistance value is sometimes referred to as the swelling index. However, it is important to note that the swelling index based on the negative R value in unloading is a different concept from the swelling index based on the percentage increase in volume after saturation, which is determined using the ASTM D4546-17 standard test.

Chapter 3 Literature review

3.1 Previous investigations

Numerous endeavors have been made to conduct examinations and investigations aimed at anticipating both the primary and secondary settlement that occurs when soil is combined with peat or organic matter. The analysis and comprehension of soil behavior, as well as the determination of approaches to address geotechnical challenges, have consistently posed significant difficulties. Although certain studies have exclusively focused on cohesive soils, such as clay, or solely on peat itself, there has been a dearth of emphasis on advancing the comprehension of predicting settlement in cohesionless soils combined with low levels of peat content. Regrettably, these studies have demonstrated inadequacies in accurately predicting long-term creep settlements, particularly in cohesionless soils with low organic content. Existing efforts in this area have predominantly focused on fine-grained soils, namely clay (Long, Paniagua, et al., 2022).

This Chapter will commence by introducing the fundamental principles and definitions pertaining to peat and organic contents within soils. Subsequently, an exploration of pertinent research, guidelines, and standards will be undertaken. Furthermore, the existing knowledge gap pertaining to specific categories of organic soils will be examined prior to any endeavors made to address this gap.

3.1.1 Basics and definitions of organic soil

Organic soils and particularly peat is predominantly distributed in the high latitudes of the Northern Hemisphere. The countries with the most extensive peatland coverage are Canada and Russia, encompassing vast areas (Long, Paniagua, et al., 2022). Additionally, significant peatland areas can be found in northern European countries, notably Finland, Sweden, Norway, Ireland, and the Netherlands. Peats are formed naturally through the decomposition of plant and animal matter under anaerobic conditions over a long period. There are characteristics in organic matter in different climate and type of plant materials constituting peat. The characterization of peat is normally defined by its inherent locality and are often described differently from both a qualitative and quantitative perspective (Zainorabidin & Wijeyesekera, 2008).

The classification of peat types was initially introduced by von Post in 1922(von Post, 1922). Alongside determining the peat type, von Post proposed evaluating its color, degree of decomposition, moisture class, fiber content (including types such as Carex and Phragmites), presence of rootlets, and woody particles.

The von Post humification classes are determined based on a scale ranging from H1 to H10, where H1 represents peat completely devoid of muck, while H10 signifies fully humified peat. In the case of the latter, when squeezed, all peat substances pass through the fingers. In the von Post classification system, moisture classes span from B1, indicating very dry conditions, to B5, indicating very wet conditions. Fiber content classes range from no fiber (F0) to mainly fibers (F3). Additionally, the classification considers the content of root threads (R) and wood (V), which can vary from 0 (indicating low content) to 3 (indicating high content).

In addition to the von Post classification, alternative methods exist for classifying peat. One such method involves the determination of pyrophosphate-soluble organic matter, utilizing an index derived from the Munsell color chart(Group et al., 1998) or determining unrubbed or rubbed fiber content in percent of total (McKeague et al., 1984).

It is worth noting that the von Post method does not necessitate the use of specialized instrumentation, making it highly suitable for field applications. Furthermore, it is considered the least time-consuming and cost-effective method. Consequently, numerous guidelines and recommendations rely on the von Post classification as a practical and accessible approach for peat classification (Stanek & Silc, 1977).

3.1.2 Geotechnical properties of peat and peat behavior in consolidation

The properties of peat are influenced by its specific locality, and variations in soil behavior can be observed when different types of peat are present. In Norway, the Norwegian Geotechnical Institute (NGI) conducted data collection on peat properties, including index data, strength, and deformation characteristics, from a total of 19 sites located in Trøndelag, Norway. The purpose of this data collection was to establish a comprehensive database for Norwegian peat. This report provides a summary of the typical geotechnical values obtained for Norwegian peat, which predominantly consists of organic content ranging from 90% to 99%. Consolidation tests employing the constant rate of strain (CRS) method and the peat oedometer apparatus were conducted to derive representative geotechnical values for Norwegian peat. These typical geotechnical values are summarized in Table 3-1.

Table 3-1 Typical geotechnical values for the Norwegian peat (Paniagua et al., 2021)

Parameter	**Value**
Total unit weight	10-12 kN/m^2
Organic content	90-99%
Water content	500-1000%
Degree of humufication	H3
Peat thickness	3-4 m
Preconsolidation stress	10-12 kPa
Modulus number	3-10
Coefficient of consolidation	3-30 m3/year
Undrained shear strength	2-8 kPa at 0.6m

In a study conducted by Long in 2013, peat sites located in Trondheim, Norway, and Ireland were investigated. Field loading tests were performed at 5 of these sites and then compared the results from laboratory tests with these field tests. The results of CRS oedometer tests and peat oedometer for peat with water content in the range of 300-1600% were included. One of the results was constrained modulus at in situ stress, which was of average of about 0.2 MPa (Long & Boylan, 2013a).

In a study by Carlsten in 1988, oedometer tests were conducted on 60 samples obtained from various Swedish soils. As a result of these tests, a deformation-water content chart was derived, which is illustrated in Figure 3-1. This chart has since become widely utilized as an initial estimation tool for predicting deformation under different load conditions. It is important to note, however, that the validity of the chart is limited to water content values ranging from 700% to 1500%. Therefore, any design or engineering practices should be based on a thorough local site investigation and testing of undisturbed samples specifically obtained from the site in question (Carlsten, 1988).

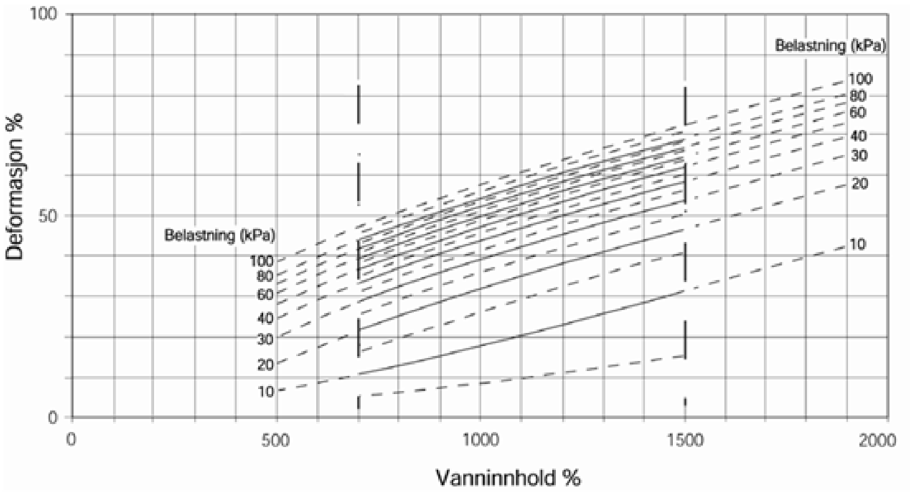


Figure 3-1 Deformation for different loading and water content. Y-axis denotes the relative compression, and x-axis denotes the water content. (Carlsten, 1988)

In another study by Berbar in 2020, an attempt was made to replicate the deformation-water content chart by collecting data from various sources. The findings of this study proposed a 20% lower estimation of strain for peat within the water content range of 500% to 1500%, in comparison to the chart originally developed by Carlsten. Berbar suggested that the significant amount of creep observed in peat may be attributed to its fiber content and the gradual degradation of fibers over time, resulting in increased compression. Based on the complex structure of peat, the study recommended the removal of peat prior to construction whenever feasible, emphasizing the challenges associated with its behavior and the potential implications for construction projects (Berbar, 2020).

3.1.3 Alternatives for dealing with peat

Peat and organic soils are typically avoided at construction sites due to their challenging geotechnical behavior. However, there are instances where avoiding or bypassing them is not possible or cost-effective, necessitating specific measures to address these soils. In cases where the organic accumulation is shallow, excavation and replacement techniques can be implemented successfully. However, for deeper peat deposits, alternative approaches such as preloading techniques or other methods such as soil stabilizing should be taken into consideration. These alternative methods aim to mitigate the adverse effects of peat and organic soils during construction and ensure the stability and long-term performance of the infrastructure.

Preloading is recognized as one of the oldest techniques used to improve the strength and stability of peat soils, enabling them to safely support the intended loads and achieve long-term compression within a shorter timeframe. Accurate prediction of the settlement of peat under both service loads and preloads is crucial in preloading design. Field testing can provide valuable data to derive rheological parameters, which are essential for employing settlement prediction methods and effectively controlling the duration of preloading. Numerous case studies have been conducted to investigate settlement prediction during preloading (Gruen & Lovell, 1984).

For example, Long in 2015 did a Full-scale loading at five sites and compared the behavior of peat with laboratory tests and found good correlations between vertical yield stress and compression index and concluded that conventional staged construction with surcharge loading may be successfully applied to peat soils as long as adequate drainage exists to permit consolidation over reasonable time intervals (Long & Boylan, 2013b).

Indeed, there have been notable case studies focusing on the long-term deformation of peat beneath embankments. For instance, in a specific case study conducted in Canada, an analysis was performed utilizing three years of direct measurements of deformation and pore pressure. The findings revealed that long-term deformation in peat is influenced by the generation of gases and the seasonal temperature-driven expansion and expulsion of these gases. These factors play a significant role in the ongoing deformation processes observed in peat over extended periods. The case study highlights the importance of considering these dynamic mechanisms when assessing the long-term behavior of peat under embankments, emphasizing the need for a comprehensive understanding of the underlying processes and their impact on peat deformation.(Acharya et al., 2015)

Indeed, there have been studies that specifically focus on modeling the behavior of peat and organic soils. One such recent study by Long in 2022 utilized the Soft Soil Creep model to predict embankment settlement on Swedish peat. The Soft Soil Creep model offers an advantage in providing an approximate estimation of settlement that considers creep deformation in both the primary and secondary consolidation phases. This modeling approach is considered a preferred method for calculating settlements in peat, especially when a realistic estimate is needed. By incorporating creep deformation into the analysis, the Soft Soil Creep model enhances the accuracy and reliability of settlement predictions for peat and organic soils, improving the understanding of long-term behavior under embankment loading conditions. (Long, Grimstad, et al., 2022).

In conclusion, significant efforts have been made to enhance our understanding of the behavior of organic soils. However, it is crucial to continue these endeavors as there are still gaps in knowledge that can be addressed through further research and study. Conducting investigations on various soil samples and engaging in comprehensive case studies will contribute to filling these gaps and expanding our understanding of the complex behavior of organic soils.

3.2 Handbooks and guidelines

Understanding the terminology used for different types of organic matter is crucial when referring to handbooks and guidelines. The Norwegian Geotechnical Society (NGF) guideline provides specific terminology for various organic materials. These include:

- Fibrous peat (known as "torv" in Norwegian): This type of peat exhibits a recognizable plant structure and is characterized by its fibrous consistency.
- Partially fibrous peat: It refers to peat that has some plant structure but may also contain other organic components.
- Amorphous peat: This type of peat is black in color and lacks visible plant structure. It has a spongy consistency.
- Gytje and dy: These are two Norwegian terms used to describe organic matter consisting of water-deposited plant and animal remains. Gytje typically exhibits an organic structure and has a grey-brown or grey-green color that lightens when dried. Grovgytje has a clear structure, while fine gytje has a less distinct structure. Dy, on the other hand, is a structureless mass rich in precipitated humic colloids. It has a brown-black color that does not lighten upon drying.

It is important to note that transitional forms can exist between these different states of organic matter, further emphasizing the complex nature of organic soils. Familiarity with these terminologies is essential for accurately describing and categorizing organic materials in geotechnical assessments and studies (NGF., 2011).

The significance of marsh areas, which are wetlands characterized by the presence of soft-stemmed vegetation adapted to saturated soil conditions, lies in their ability to store substantial amounts of carbon dioxide. Therefore, it is crucial to prioritize the preservation of these areas to mitigate carbon emissions. It is important to recognize that any interventions or alterations in these lands can have an impact on their sustainability. When interventions are necessary, it is essential to choose methods that minimize the disturbance to peat resources as much as possible. The ecosystem of marsh areas heavily relies on maintaining a high level of groundwater. Preserving the groundwater levels in these lands is therefore vital to ensure the continued functioning of their ecosystems. Consequently, it is crucial to safeguard these wetlands and their groundwater resources.

In the context of road construction, it is recommended by the National Road Administration to limit or avoid interventions that directly impact marsh areas. This recommendation takes into consideration the environmental value of these lands and emphasizes the importance of preserving them in the interest of sustainability and environmental conservation (Aker & Dalen Johansen, 2015).

The Norwegian Road authority, Statens Vegvesen, employs a method for estimating peat settlements developed by Peter Carlsten. This particular method is advantageous as it does not require any deformation laboratory testing. Instead, it relies on values obtained from a Swedish study conducted in 1988. The method enables an estimation of peat deformation based on water contents and in-situ effective stress. By utilizing these parameters, engineers and practitioners can initially have a rough approximation for peat settlements without the need for extensive laboratory testing, facilitating the assessment and design of infrastructure projects in peat areas. (vegvesen Vegdirektoratet, 2022). Using this correlation, peat deformation can be estimated, requiring only water content testing, which is relatively straight forward and inexpensive, however, as mentioned in 3.1.2, the chart should be used with cautious and just as a tool for initial estimation of deformation. This model is presented in Figure 3-1. Statens Vegvesen also employs an alternative but similar model, which is presented in Figure 3-2.

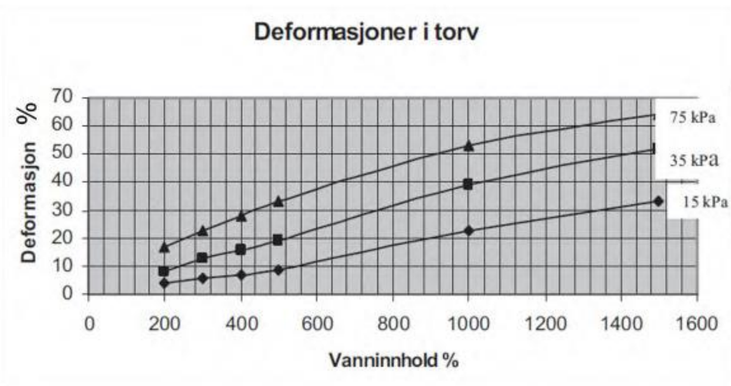


Figure 3-2 deformation for different loading and water content-The y-axis denotes the relative compression, while x-axis denotes the water content (vegvesen Vegdirektoratet, 2022)

As it has been implied, one can have a rough estimation for settlement by the help of these two graphs in Figure 3-2 and Figure 3-3. For a given water content, peat thickness and load, time (in 24 hours) corresponding to different levels of consolidation (two-sided drainage) is read.

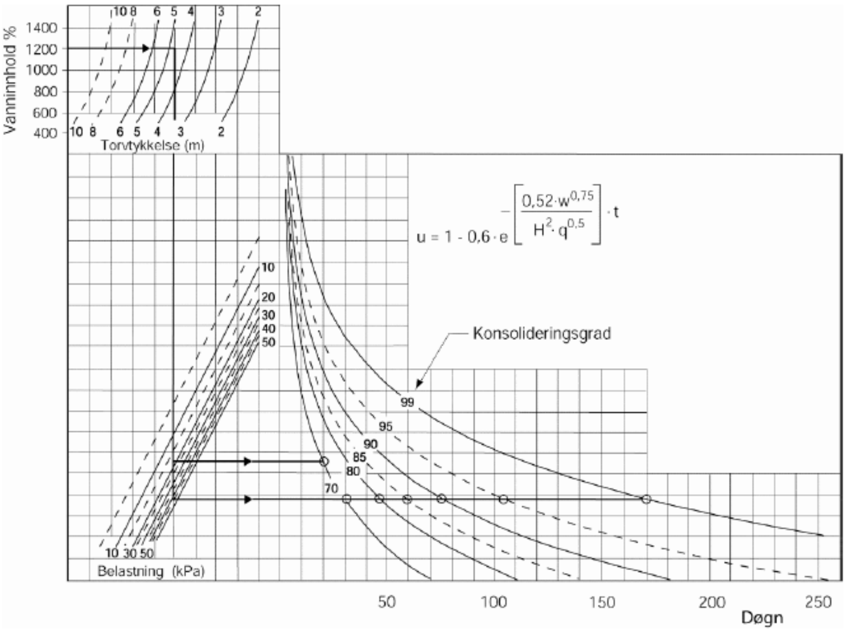


Figure 3-3 relationship between load and deformation (Carlsten, 1988)

Even low organic content will be able to provide one significant increase in a soil ability to absorb and bind water and thus lead to large changes in the mechanical properties of the soil. The high water content in organic soils leads to great compressibility when loaded or drained.

3.3 Gap in previous research

As discussed earlier, even the results that have been recommended by guidelines as an initial estimation of settlement, have a high range of organic content with high water content. There are few guidelines on how soils with lower organic contents can be handled. In Norway, Eurocode7 (EC7, 1997-2:2007+NA:2008) is being used as a standard for geotechnical practice. Regarding organic matter, emphasis is made on its negative influence on bearing capacity and compressibility, and its possible effect on laboratory test results. Nevertheless, it is evident that the standard guidelines on soils containing organic content are limited and too general. This leaves the practicing engineer to decide what is required in the face of different geotechnical problems. Nearly all studies have been conducted on high organic soils and most of them are related to clays.

Furthermore, there are projects that have faced challenges and created uncertainties for geotechnical engineers, prompting the need for further investigation into the behavior of granular materials mixed with organic content. One notable example is the project located in Klettelva, which has been discussed in Chapter 1 as a real case problem.

Chapter 4 Material and methods

The experimental investigation conducted in this thesis encompasses two primary types of materials, which were tested using both a fixed-ring oedometer (referred to as a standard or conventional oedometer/FROedo) and a compressible-ring oedometer (newly proposed oedometer/CROedo).

- Samples prepared at the NTNU laboratory referred to as *built samples.*
- The collected samples from the Klettelva site in two forms: *bag samples* and *cylinder samples* (refer to Chapter 1 for project description).

The focus of testing the built samples and bag samples from Klettelva is to establish a fundamental and comprehensive understanding of the behavior exhibited by cohesionless soils containing organic constituents. Moreover, there will be a greater emphasis on studying the case in Klettelva by testing cylinder samples. Oedometer testing on cylinder samples aims to accurately simulate the preloading method proposed by Sweco for the E39 project, effectively mimicking the in-situ process, and incorporating all stages of preloading, unloading, and reloading on a less disturbed sample. This approach has been adopted to attain a more comprehensive understanding in the behavior of soil, investigate the presence of long-term creep, and assess the efficacy of preloading as a measure to mitigate excess settlement in materials mixed with organic content (≤10%).

4.1 Material

4.1.1 Built samples

To develop a comprehensive understanding of the behavior of sand mixed with peat, built samples were constructed using sand from the NTNU laboratory and purchased peat, which will be referred to as "built samples" in this study. The grain size distribution of the sand is determined using standard sieves and a mechanical shaker and hydrometer for finer particles, following the standard guidelines (ISO, 2004a). The results of grain size distribution will be presented in Chapter 5.It can be concluded that the sand primarily falls within the Medium sand category, with little finer sand particles, based on the soil classification guidelines provided by NTNU (NTNU, 2015).

The peat used in the experiments was potting soil purchased from a store in Trondheim, Norway. The peat composition primarily consisted of weakly decomposed peat (H2-4), highly decomposed peat (H6-8), bark, sand, clay, lime, chicken manure, and mineral fertilizers.

Detailed specifications of the peat constituents can be found in Table 4-1. For more detailed information and the complete specifications and ingredients of the peat, refer to Appendix B.

Table 4-1 Peat Properties obtained from the peat's catalogue.

Electrical conductivity (+/-25%)	40 mS/m
pH (H_2O)	5.5-6.5
Organic content	>60% of ts
Density	425 kg/m^3
Particle sizes	0-35 mm

Sample preparation

To prepare "built samples", the peat underwent an initial drying process before being mixed with dry sand (Figure 4-1). Drying the peat was carried out through spreading the peat material in layers on a concrete surface and periodically turning it over. The material was left out to gradually dry over 15 days at room temperature of 23°.

After the drying process, different combinations of sand and peat with varying water contents (specified in Table 4-2) were mixed and placed in Moisture barrier bags to maintain consistent moisture levels across all samples. It is important to note that the proportions of the sand and peat mixture are expressed in weight percentages. The built samples in this study were constructed by changing the weight percentage of peat from 0 to 10%. The specific mixture proportions are presented in Table 4-2. The method and details of which will be discussed in subsequent sections.

The decision to investigate specimens within the range of peat content in this study is based on preliminary findings obtained from a specialization project (Fakhari, 2023). The preliminary results indicated that soils containing more than 10% organic content exhibited excess deformation and were recommended to be removed from the site. Furthermore, considering the Klettelva project as a significant motivator for this study, the choice to focus on the range of up to 10% organic content aligned with the study's objectives and areas of interest.

Figure 4-1 Dried peat (Left) and dried sand (Right)

Testing of the built samples, which involves the simple mixture of dried sand and dried peat, neglects the potential influence of stress history, which could impact the results. However, it is expected that the effect of stress history on granular soils may be relatively low. It is important to acknowledge that investigating the impact of specimen disturbance on oedometer tests could be an additional scope of work that has not been addressed in this study.

Sample labeling

To facilitate the distinction between the tested samples, each sample is labelled according to the type of material. For built samples, the naming system consists of letters “S” and “P”, indicating sand and peat, respectively, followed by a number that suggests the percentage of the peat in the respective sample.

SP(x) ← x is peat percentage

Table 4-2 specimen mixtures and their characteristics

Sample notation	**Sand percentage**	**Peat percentage**	**Water content**	**Bulk Density**	**Fine portion (%)**	**Coarse portion (%)**
	(%)	(%)	(%)	(gr/cm^3)	63	37
SP0	100	0	16	1.81	63	37
SP2	98	2	20	1.73	62	38
SP4	96	4	20	1.62	61	39
SP6	94	6	20	1.54	64	36
SP8	92	8	25	1.51	63	37
SP10	90	10	30	1.42	63	37

*The distinction between fine and coarse grain is according to the guidelines in handbook R210 of SVV. With this regard particles with diameter under 500 µm are deemed fine, and particles’ diameter over 500 µm is considered as coarse.

4.1.2 Klettelva samples

For this research, two new boreholes, BP3088 and BP3089, were drilled at the Klettelva project site. Bag samples and cylinder samples for this study were obtained from these boreholes during the construction of the new E39 project at the specified date. The location of these boreholes is indicated in Figure 4-2.

Additionally, it is worth noting that these boreholes were excavated after the preloading process had already been carried out, and measurements had been conducted to address compression and settlement issues caused by organic content. The primary area of interest, which experienced the most severe conditions in Klettelva, was located around borehole no. 3041 (as shown in Figure 4-2). However, it was not possible to obtain samples from this area as the project was nearing completion, and the road had already been constructed over it.

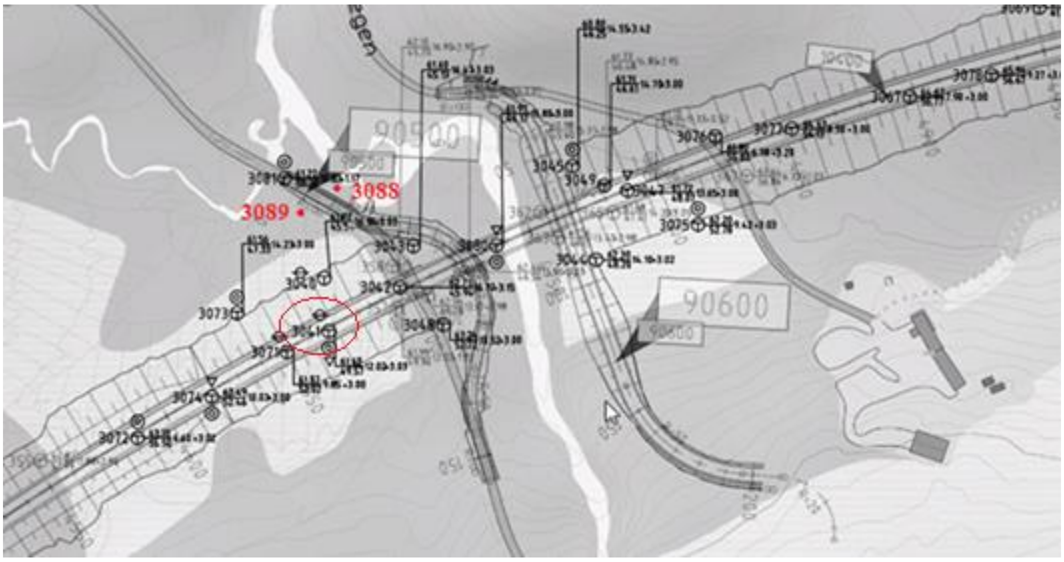


Figure 4-2 Location of two new boreholes at Klettelva site for this study

Bag samples from Klettelva

A total of 10 bag samples were collected from different depths within these boreholes. Table 4-3 presents the specifications for each bag sample. It is important to note that bag sample no. 5 could not be tested due to insufficient material available for reconstructing the sample in the proposed oedometer. Initial characterization tests including grain analysis, peat percentage content (using loss on ignition), density, and water content were determined prior to the oedometer tests.

Table 4-3 Bag samples specifications

Bag sample no.	Borehole no.	Depth (m)	Fine portion (%)	Coarse portion (%)	Water content (%)	Density (gr/cm^3)
1		2-3	92	6	79	1.62
2		3-4	93	7	58	1.63
3	BP 3088	4-5	84	16	96	1.6
4		5-6	98	2	56	1.59
5		6-7	-	-	-	-
6		7-8	99	1	60	1.65
7		3-4	98	2	94	1.52
8	BP 3089	4-5	99	1	58	1.54
9		5-6	97	3	61	1.55
10		6-7	100	0	49	1.61

*The distinction between fine and coarse grain is according to the guidelines in handbook R210 of SVV. With this regard particles with diameter under 500 µm are deemed fine, and particles' diameter over 500 µm is considered as coarse.

It is crucial to note that the bag samples also underwent significant disturbance during collection, which led to the elimination of depth-related influences. The initial round of tests was conducted promptly upon receiving the samples to minimize water content loss. The results obtained from these tests served as a basis for developing a fundamental understanding of the behavior of soils containing organic content. Subsequently, the behavior of these soils was analyzed under various load increments to assess their compressibility and long-term settlement characteristics.

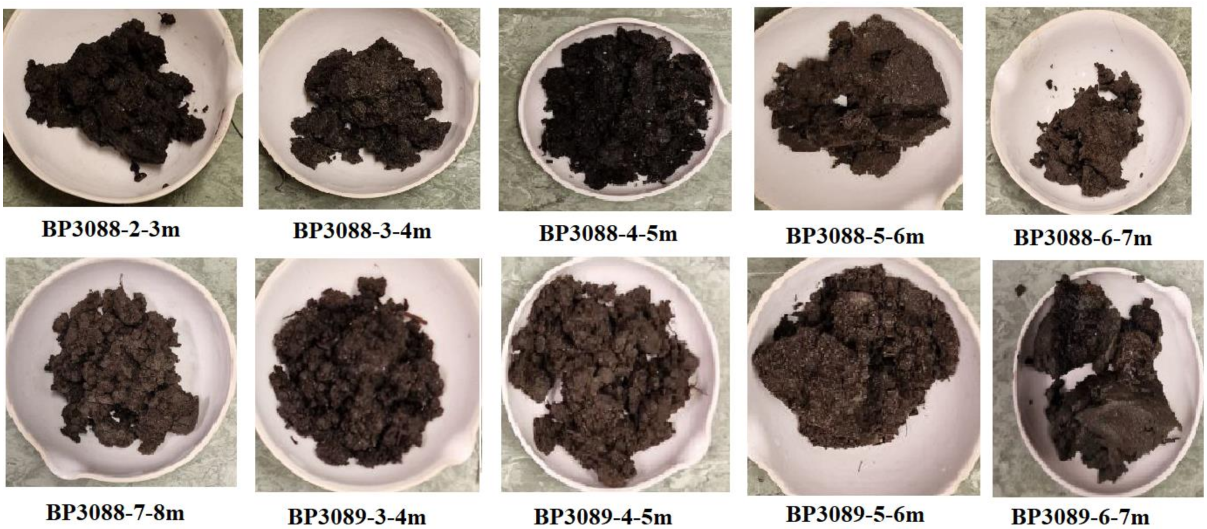


Figure 4-3 Appearance of Klettelva bag samples

Cylinder samples

A total of four ø54mm cylinders samples were collected from the two additional drillings boreholes of BP3088 and BP3089 (Figure 4-2). Detailed information about the cylinders and the comprehensive laboratory results can be found in Appendix D. A summary of the relevant information pertaining to these cylinders can be presented in Table 4-4. As with the bag samples, the same characteristics tests including grain analysis, peat percentage content (using loss on ignition), density, and water content were performed.

Table 4-4 Cylinder samples' initial specifications

Cylinder No.	**Borehole No.**	**Depth (m)**	**Water content (%)**	**Density (gr/cm^3)**
1	BP3088	4-5	90	1.6
2	BP3088	6-7	80	1.8
3	BP3089	4-5	48	1.7
4	BP3089	6-7	40	1.9

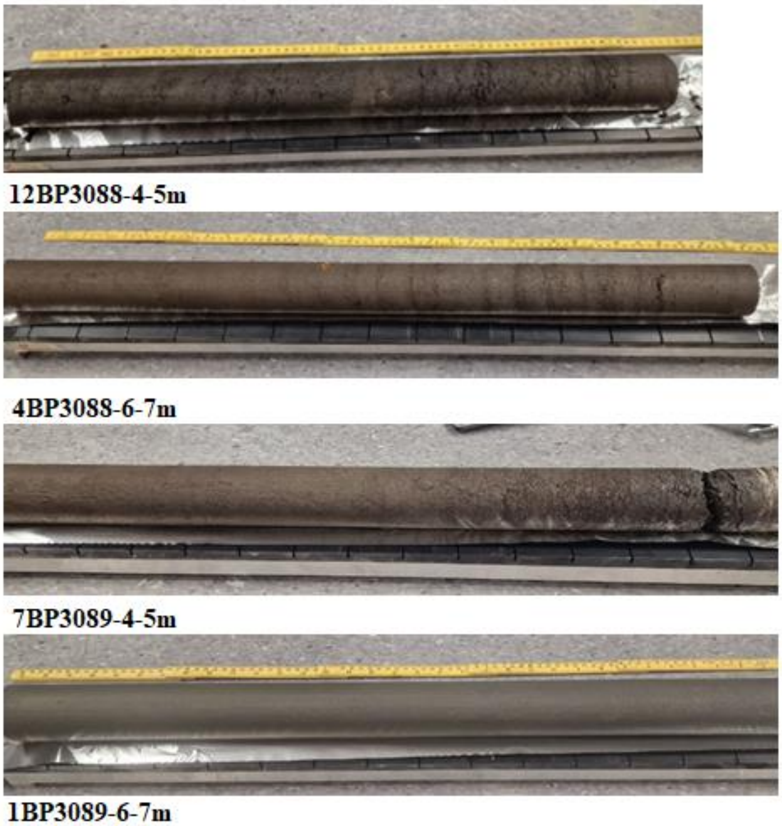


Figure 4-4 Extracted Klettelva cylinder samples.

Sample labeling

For the bag and cylinder samples from Klettelva, the following name conventions are used throughout this research. Peat percentage, the type of the oedometer test and borehole number is indicated in the labels as the most relevant information for later interpretations.

(p)BP(3088 or 3089)(CROedo or FROedo)		
Peat percentage	Borehole number	Oedometer type

(p)BP(3088 or 3089)(x-ym)(CROedo or FROedo)			
Peat Percentage	Borehole number	Cylinder depth (top and bottom)	Oedometer type

For example, 7BP3089-4-5m-CROedo indicates a sample with 7% peat, which was taken from borehole 3089, from the depth of 4 to 5 meter and tested using the compressible ring oedometer.

4.2 Oedometer tests

4.2.1 Fixed ring Oedometer (FROedo)

The Oedometer test with a “fixed ring” and “incremental loading (IL)” was performed on all bag samples and cylinder samples collected from Klettelva (see Table 4-8). The fixed ring oedometer for this research follows the basic principles mentioned in section 2.4.

The fixed ring oedometer was utilized for both bag samples and cylinder samples. Of the two groups, the bag samples had a disturbed nature from the collection process, while the standard diameter of the cylinder samples allowed for better preparation of the sample.

Nevertheless, the standard oedometer is typically designed and applied to clayey specimens, while the samples for this study are largely dominated by organic cohesionless soils. This would result in several issues including the effect of side frictions or that the prevention of swelling is less effective when peat is combined with sand, compared to clay devoid of any organic constituents.

Figure 4-5 Fixed Ring Oedometer used from NTNU geotechnical laboratory.

Sample preparation

For the FROedo tests, the cylinder samples were prepared according to the standard practice in ISO 17892-5:2004. However, compared to the standard test for clayey samples, there remains a degree of uncertainty whether the test quality is acceptable, since the looseness of the sample is less negligible compared to the situation where finer particles' cohesion holds the specimen. This would still make the extraction of samples challenging, however, the samples in this study may be deemed acceptable with as minimal disturbance as possible.

For the bag samples, the main objective is typically to build a sample that inherits the *porosity* of the in-situ material. Hence, it is always crucial to employ appropriate techniques that yield samples with the desired range of porosity while ensuring homogeneity. Typically, to prepare loose samples with high porosity, a method involving gentle filling of sand and leveling with a glass rod is employed. This technique allows for the preparation of both dry and moist samples. Denser samples are normally obtained through various methods such as tamping, vibration, and sedimentation, with the specific approach depending on the soil type and the in-site specifications. It may be necessary to explore different preparation techniques to determine the optimal procedures for granular materials.

For this study, to build comparable specimens, efforts were made to rebuild samples according to their in-situ “density”. It is assumed that with the preserved water content from the field, density could be a representative parameter for the porosity. To achieve this, consistent tamping procedures were followed, including using the same hammer, applying the same force of 50 N per hit, and employing an equal number of 20 hits per layer. However, it should be noted that tamping and the sample preparation process remain important factors that may impact the results and should be considered when interpreting the findings. Finally, the surface of the sample was then fully flattened using a spatula to allow for uniform stress over the upper face of the specimen.

4.2.2 Compressible ring oedometer (CROedo)

The novel proposed oedometer (see Chapter 2, Section2.4) with a set of compressible rings was conducted with incremental loading (IL) on both the collected samples from Klettelva and the built samples. The primary configuration comprised a cylindrical soil specimen with a diameter of 15 cm and a varying height. Testing different foams, different number of layers, and different foam thicknesses were necessary to decide on the more durable ring set with compressible foam based on the tests. Furthermore, foam was degrading to a degree after each test and losing the memory, compressibility, and its thickness which led to a lower height of the ring set. Thus, the height of the ring set should be measured before running each test, in order to use it for further strain calculations.

Additionally, the larger diameter of the ring meant more material was needed for building the samples in the ring set. Also, the extent of homogeneousness of material was of more importance in this oedometer compared with the fixed-ring odometer. Thus, Building the sample needed more accuracy and the sample was more sensitive to the preparation process including tamping. It also took more time to build the samples, which could lead to loss of water content while building the sample.

For oedometer testing with compressible ring oedometer, the same principles outlined in section 4.2.1 were necessarily followed for sample preparation. Consistent tamping procedures were employed, using the same hammer, applying the same force, and delivering an equal number of hits per layer. If there is a desire to further enhance the proposed oedometer, considering the complexities surrounding porosity calculation would be a crucial aspect to consider.

4.2.3 Loading increments

While the author awaited the collection of Klettelva samples, experimental studies commenced using the compressible ring oedometer on constructed samples. After conducting initial tests on the built samples (specifically, the test on SP10), a minor adjustment in loading increments was made to gather additional data. Reloading steps were added to enhance the study of stress-strain behavior in particular. Consequently, there exists a slight variation in SP10, whereas the remaining samples adhere to the same loading steps to ensure comparability.

The test procedure involved incremental loading, unloading, and reloading. Unloading primarily aimed to better understand the soil behavior, especially with regards to swelling, while reloading reassessed the stiffness and compressibility of the soil.

For bag samples of Klettelva, the selection of load increments in the loading stages was aiming to gain a general insight regarding the behavior of materials. Loading steps are summarized in Table 4-5.

Table 4-5 Load increments for bag samples of Klettelva (for both FROedo and CROedo)

		Loading increments							
Load	**kPa**	**12.5**	**25**	**50**	**100**	**200**	**300**	**400**	**600**
Duration	**min**	30	30	30	30	30	30	30	30
		Unloading increments							
Load	**kPa**	**400**	**300**	**200**	**100**	**50**			
Duration	**min**	60	60	60	60	300			
		Reloading increments							
Load	**kPa**	**100**	**200**	**300**	**400**				
Duration	**min**	60	60	60	60				

The duration of load increments during loading was set at half an hour, while increments in unloading and reloading were extended to one hour to better monitor deformation changes, particularly over longer periods. To enhance the accuracy in monitoring creep, the final stage of unloading was held for 8 hours. The decision regarding the duration of load increments was based on the findings of the pre-study, which indicated the importance of longer duration increments during unloading to accurately assess the extent of swelling. It is important to acknowledge that due to the limited timeframe of the study, the durations of load increments were determined accordingly. While efforts were made to ensure an appropriate duration for each increment, it is recognized that a more extensive and detailed study with longer duration increments, where necessary and relevant, would provide improved insights and a deeper understanding of the soil behavior under investigation.

It is important to note that different load increments were applied to the cylinder samples of Klettelva, aiming to closely replicate the loads utilized in the Klettelva project. A summary of the load increments employed for the oedometer testing is presented in Table 4-6.

Table 4-6 load increments employed for the oedometer testing in cylinder samples of Klettelva.

Phase No.	**Description**	**Load increment**		**Phase duration in test (min)**
		Depth of 4-5m	**Depth of 6-7m**	
0	Simulating the in-situ condition	32	50	60
1	Mass replacement 3 meters.	11	28	60
2	Filling 3m with blasted stone	68	85	30
3	Filling 3m with blasted stone	125	142	30
4	Filling 1.5m with blasted stone	154	171	30
5	Filling 1.5m with blasted stone	183	200	30
6	Removing 3.5m of top	117	134	60
7	Filling 2.5m light mass+ 1m blasted rock	149	166	480
8	Simulating final condition with added traffic load	179	196	180
9	Adding extra load to check the stress history and behavior of soil	239	256	60

Following the completion of preloading stages and the removal of a portion of the filling, the ultimate condition of the embankment in the long term is represented in Phase 7. As this phase signifies the concluding state of the embankment, it has been determined that an 8-hour duration is sufficient for load application; however, it is acknowledged that this timeframe remains relatively short. One shall consider that the typical design life of a road ranges from 20 to 50 years or more, contingent upon specific circumstances and requirements. Additionally, the embankment will remain for the duration of the road's operational lifespan. Therefore, attempting to simulate a 20-year timeframe within an 8-hour period raises uncertainties, emphasizing the necessity for lengthier load increments when investigating the time-dependent properties of soil.

During Phase 8, an additional traffic load of 30 kPa has been considered in the study. It is noteworthy that in the actual project, the load due to traffic was considered to be 20 kPa based on the standards and specifications (NPRA, 2018). However, in the present study, a slightly higher load was considered. It should be noted that the duration of mass replacement phases took approximately 1-6 months.

4.3 Loss on ignition test

Loss on ignition tests is a method for the quantitative determination of organic content in loose materials and is based on the thermal decomposition of organic matter. The method can be used for silt and clay soils, as well as for gravel materials used in reinforcement and base layers (Davies, 1974).

It is of importance to note that there does not necessarily exist a unique definition for the LOI parameter. The definition could vary accordingly for each purpose in different sciences. However, for geotechnical engineering it may feasible to refer to the handbook R210 of "Norwegian Public Roads Administration"(vegvesen Vegdirektoratet, 2005). According to this guideline, several criteria is specified for the soil masses which includes:

- The organic content is defined only for the portion of the soil material with particle size lower than 500 μm.
- The sample must be fully dried out using the water content tests defined in ISO 17892-1:2004.
- Organic content is defined as the mass lose at 480 °C

The quantity measurement relates to material that passes through a sieve with a mesh size of 500 μm and therefore does not necessarily represent the same quantity measurement in relation to the total sample. However, in most cases, the method is considered accurate enough for typical geotechnical investigations. Table 4-7 shows the recommendation categories for the soil type with grain size less than 2 mm with organic content.

Table 4-7 Classification of soil type with grain size ≤2 mm with organic content ((vegvesen Vegdirektoratet, 2022))

condition	Organic content (weight percent)
Low organic	2-6
Medium organic	6-20
High organic	>20

Loss on ignition test according to handbook SVV R210

According to handbook R210 (vegvesen Vegdirektoratet, 2005) the organic content test starts by placing approx. 150 grams of the sample in a drying oven and dried to a constant weight at a temperature of 110 ± 5 °C. The sample is then pulverized and sieved through a 500μm sieve. About 20 grams of the material that passes through the sieve is placed in a porcelain dish and put back in the drying oven for about 2 hours at the same temperature as before to ensure that the material is completely dry. After about 2 hours, the sample is transferred to a desiccator and cooled to room temperature. After cooling, 10 grams of the prepared material is weighed in a numbered refractory dish, which is placed in an annealing oven that has a constant temperature of 480 ± 25 °C. After 24 hours, the sample is removed and placed in a desiccator to cool. After cooling, the sample is weighed, and the mass loss during annealing is determined.

The weight loss of the soil sample due to the oxidation of the organic matter, termed the loss on ignition (LOI (%)) can be formulated from the following:

Equation 4-1

$$LOI(\%) = \frac{W_{105} - W_{480}}{W_{480}} * 100$$

Where W_{105} is the weight of the soil after oven-drying to 105° for 24 hours and W_{480} is the weight of the soil after ignition at 480° after 24 hours. The temperature of 480° used in this equation is based on R210 Laboratory investigations (vegvesen Vegdirektoratet, 2005).

For this specific research, due to the significant variation in the collected samples from different depths at Klettelva, ignition loss test was three times to accurately determine the organic content. The sample was thoroughly mixed to ensure a representative portion was used for each test, thereby reducing potential bias. To provide a comprehensive analysis of the data, the average and standard deviation of the LOI values is reported.

It should be noted that one can use other methods, like Hydrogen Peroxide method, to have more comprehensive assessment of the organic content and see which alternative fits best depending on the type of material (Huang et al., 2009). This study has merely done the ignition loss test to report the organic content in the samples collected from Klettelva. Other investigations on different methods to report a representative value for organic content is an important factor that can be considered in further studies.

4.4 Pariometer

In this research study, the Pariometer, an automated hydrometer, was employed to gain a comprehensive understanding of the grain size distribution of the collected samples. Pariometer follows the same principle as hydrometer which involves dispersing a soil sample in water, allowing the particles to settle, and measuring the settling velocities and the tests shall follow ISO 17892-3:2004. The hydrometer readings are then used to calculate the particle size distribution. The Pariometer is a cutting-edge instrument that offers significant advantages over traditional manual methods, providing more precise and efficient measurements. By utilizing this advanced apparatus, the author was able to obtain accurate data on the grain size distribution. The results of the grain size distribution analysis using the Pariometer are presented in Chapter 5.

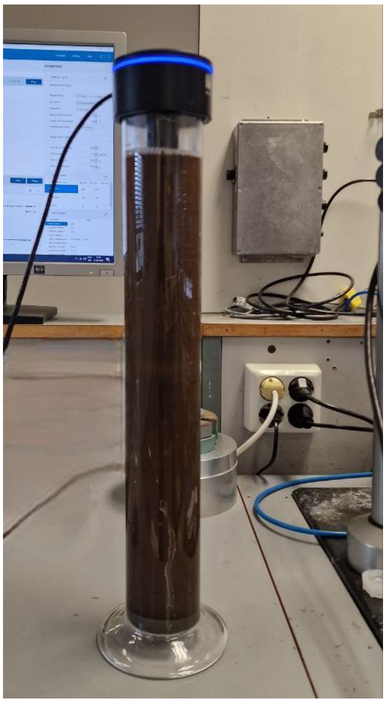


Figure 4-6 Pariometer used at NTNU laboratory.

4.5 Summary

In this study, a total of 19 samples, including built samples and Klettelva samples, were subjected to two different types of tests: Characteristic tests and oedometer tests. The characteristic tests provided valuable insights into the composition of the samples based on two key parameters: the percentage of organic content and the grain size distributions. The second type of tests involved oedometer testing using two different types of oedometers, the compressible ring oedometer (CROedo) and fixed ring oedometer

(FROedo), to study the consolidation behavior and compressibility characteristics of samples.

To facilitate a clear understanding of the implemented tests for each sample, a summary has been presented in Table 4-8. This table provides a comprehensive overview of which tests were conducted on each sample, enabling easy reference and analysis of the obtained results. Note that the samples are labeled in accordance with the explanation provided in Section 4.1.

Table 4-8 Summary of performed tests.

Type of sample	**Sample Name**	**Characteristic tests**		**Oedometer tests**	
		GSD*	LOI**	FROedo ***	CROedo ****
Built samples	SP0	✓	✓	×	✓
	SP2	×	✓	×	✓
	SP4	×	✓	×	✓
	SP6	×	✓	×	✓
	SP8	×	✓	×	✓
	SP10	×	✓	×	✓
Bag samples of Klettelva	5BP3088	×	✓	✓	✓
	2BP3088	×	✓	✓	✓
	7BP3088	×	✓	✓	✓
	3BP3088	×	✓	✓	✓
	1BP3088	×	✓	✓	✓
	9BP3089	×	✓	✓	✓
	7BP3089	×	✓	✓	✓
	6BP3089	×	✓	✓	✓
	2BP3089	×	✓	✓	✓
Cylinder samples of Klettelva	12BP3088-4-5m	✓	✓	✓	✓
	4BP3088-6-7m	✓	✓	✓	✓
	7BP3089-4-5m	✓	✓	✓	✓
	1BP3089-6-7m	✓	✓	✓	✓

*Grain size distribution
**Loss on ignition
***Fixed Ring Oedometer
****Compressible Ring Oedometer

Chapter 5 RESULTS

5.1 Characteristic tests

5.1.1 Grain size distribution tests

In order to obtain a comprehensive understanding of the material composition, both sieving analysis and digital hydrometer (Pariometer) tests were conducted on samples. The grain size distribution results of these tests are visually depicted in Figure 5-1 and Figure 5-2 for Klettelva samples and sand in built samples respectively. Moreover, Table 5-1 provides a summary of the sand, silt, and clay content in all the samples, offering a concise overview of their respective compositions. For further thorough information see Appendix C.

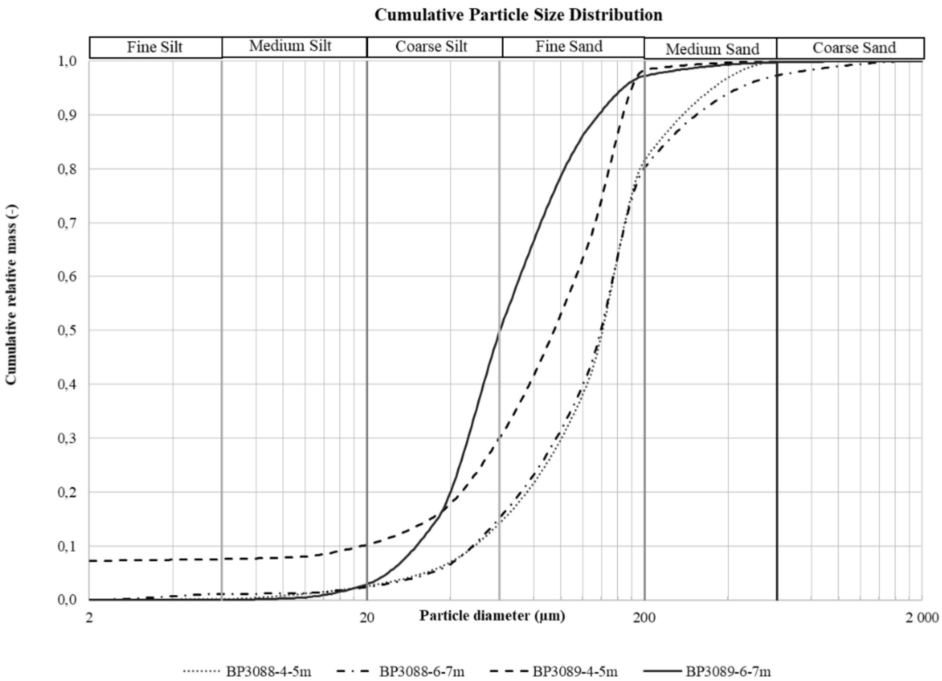


Figure 5-1 Size distribution of Klettelva samples.

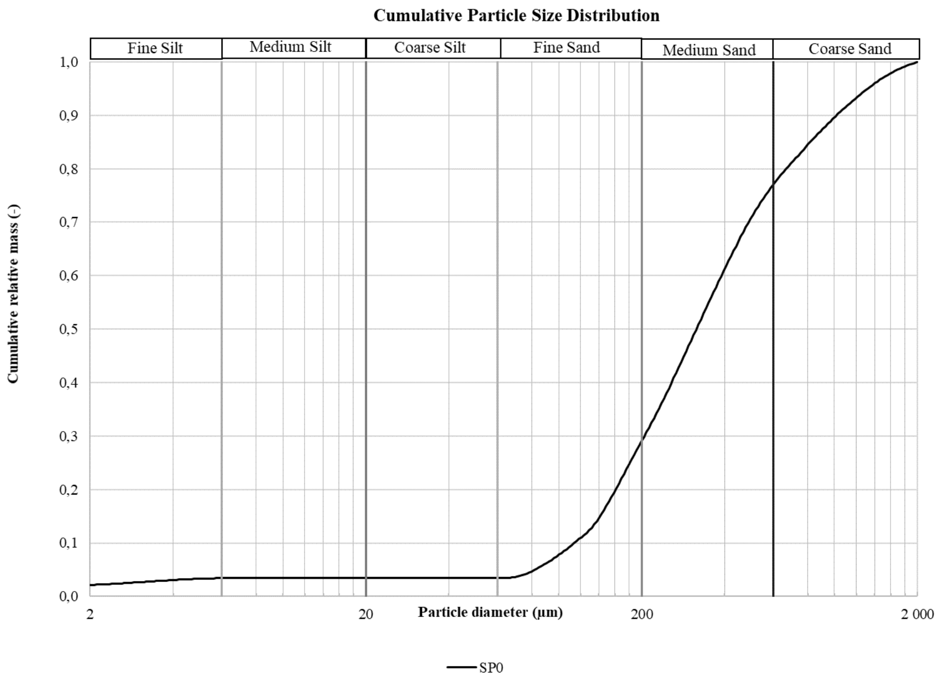


Figure 5-2 Particle size distribution of sand used in built samples.

Table 5-1 Summary of composition in cylinder samples and SP0 in built samples

	Sample	**Sand content (%)**	**Silt content (%)**	**Clay content (%)**
Klettelva Samples	BP3088-4-5m	90	10	1
	BP3088-6-7m	89	11	0
	BP3089-4-5m	76	17	7
	BP3089-6-7m	64	36	0
Built samples	SP0	97	1	2

5.1.2 Loss on ignition tests

To calculate the percentage of organic content, the author used the loss on ignition test according to described method in the previous Chapter. In the current study, the ignition loss test was repeated 3 times for all samples. The samples were thoroughly mixed to ensure a representative portion was used for each test, thereby reducing potential bias. The obtained values of loss on ignition tests exhibited variations among the samples. The variation in the measured percentages of peat content underscores the inherent uncertainties associated with the LOI method for estimating peat content, which should be considered when interpreting the results.

The average and standard deviation of the LOI values in Klettelva samples were calculated and presented in Figure 5-3 and Figure 5-4.

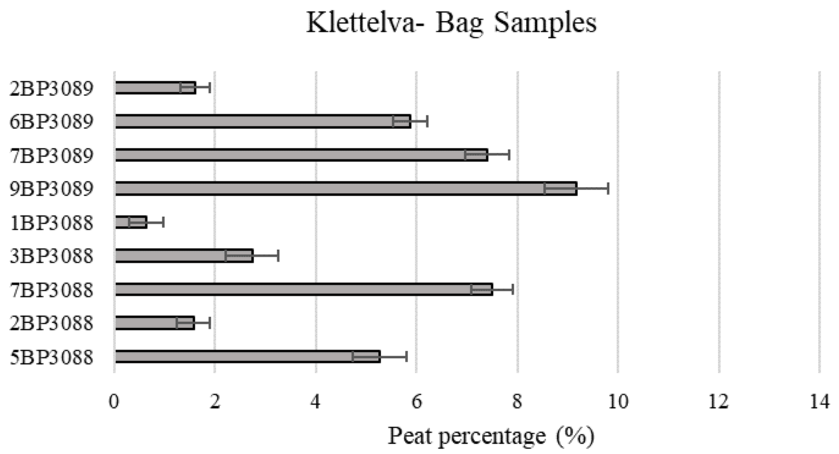


Figure 5-3 peat content in Klettelva bag samples

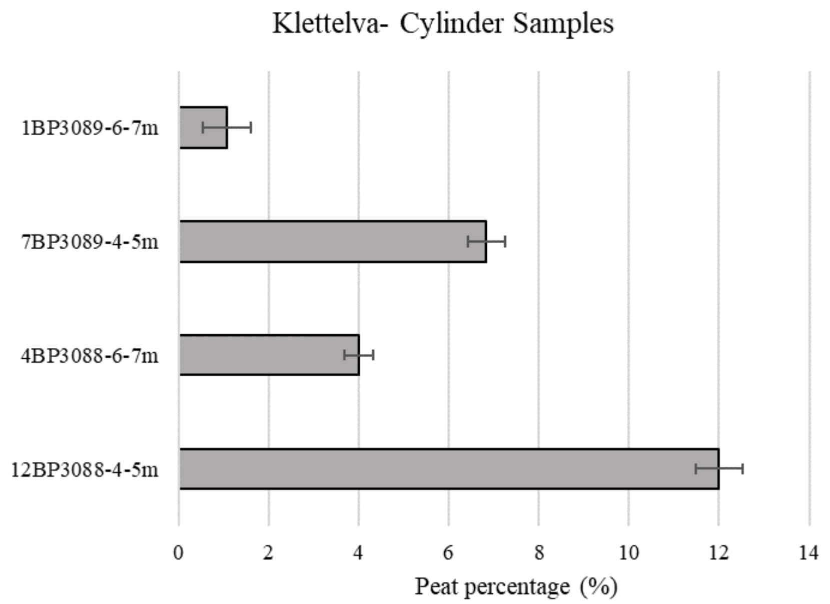


Figure 5-4 peat content in Klettelva cylinder samples

In addition, it is pertinent to discuss the nature of the built samples and the motivation behind conducting the LOI tests on built samples. As the samples were constructed in the laboratory, the content of peat was already known beforehand. Despite this foreknowledge, a deliberate decision was made to perform the LOI tests to assess the potential differences in peat percentage determination. The results on built samples confirm that the LOI test can yield slight variations in reporting the peat percentages. (See Figure 5-5)

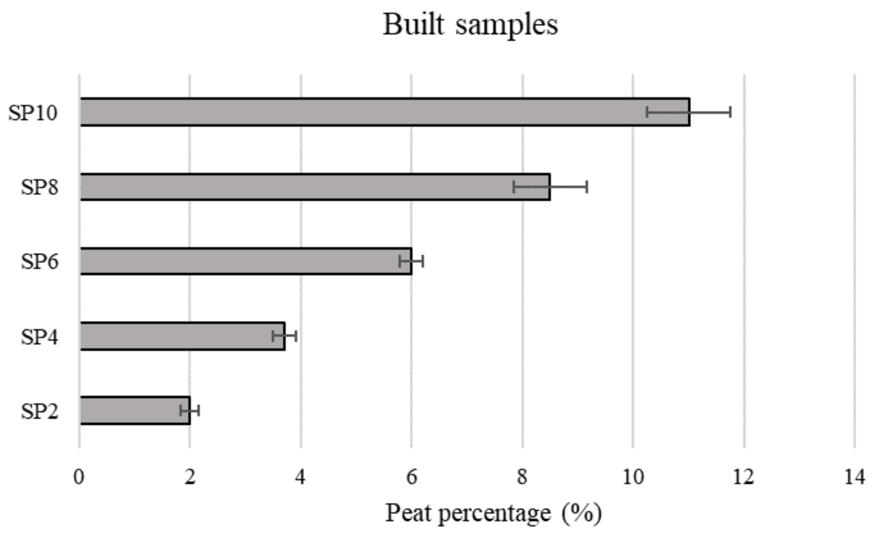


Figure 5-5 peat content in built samples

The observed variations in the peat percentages obtained through the LOI test can be attributed to several factors. One significant factor is the potential lack of sample homogeneity, which is mostly observed in Klettelva samples. During the LOI test, only a small chunk is selected to represent the entire sample. If the sample exhibits heterogeneity in terms of peat distribution, the selected chunk may not accurately represent the overall composition. The specific part of the sample from which the LOI sample is selected within the larger sample can influence the reported peat percentage. In cases where the sample is not uniformly mixed, variations in the peat content can arise depending on which part of the sample is chosen for the LOI analysis. This non-uniformity within the sample contributes to the observed variations in the LOI test results, further highlighting the importance of considering the representativeness of the selected chunk when interpreting the reported peat percentages.

Additionally, experimental factors, such as temperature fluctuations during the ignition process and potential measurement errors, can contribute to variations in the results. Based on the findings, it can be concluded that the built samples constructed by the author were comparatively more homogenous than the samples collected from Klettelva. These findings underscore the inherent challenges in accurately determining peat content using the LOI method and emphasize the importance of considering potential sources of variability when interpreting the results.

One can use other methods, like Hydrogen Peroxide method, to have more comprehensive assessment of the organic content and see which alternative fits best depending on the type of material. This study has merely done the ignition loss test to report the organic content in the samples collected from Klettelva. Other investigations on different methods to report a representative value for organic content is an important factor that can be considered in further studies.

5.2 Oedometer tests

The recorded data from oedometer testing with incremental loading can provide an initial insight into the behavior of soil samples. One of the primary outcomes that can be obtained is the time deformation curve which is represented in the following figures for all built samples and Klettelva samples.

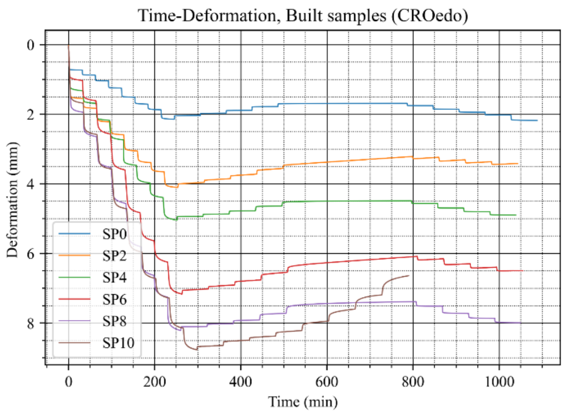


*Figure 5-6 Time deformation curve for built samples by author - tests are done by **CROedo** (*FROedo tests has not been done on built samples)*

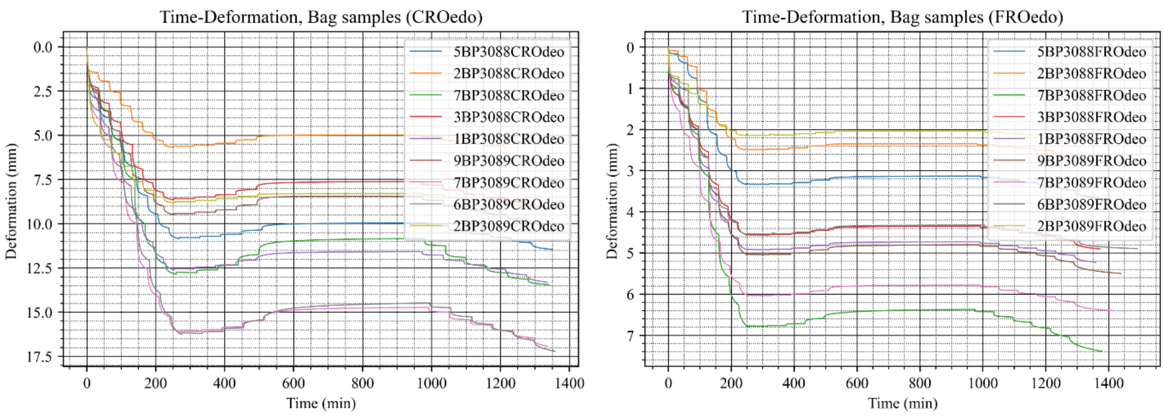


Figure 5-7 Time deformation curve of Klettelva bag samples

*left figure: **CROedo**-right figure: **FROedo.***

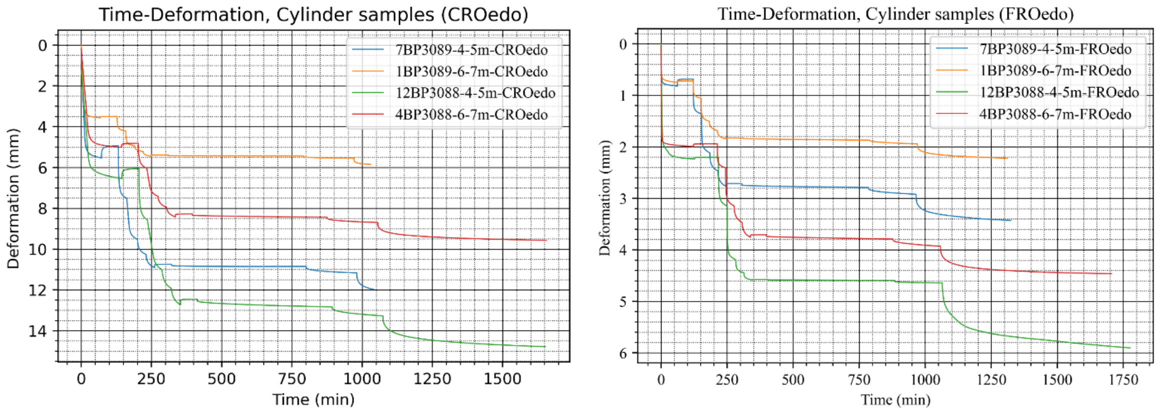


Figure 5-8 Time deformation curve of Klettelva cylinder samples
*left figure: **CROedo**-right figure: **FROedo.***

5.2.1 Stress strain behavior

The response of samples to different applied stress, including their deformation characteristics has been illustrated as stress-strain curves. These curves are used as a fundamental tool to analyze the mechanical behavior of soils under loading, unloading and reloading conditions in the next Chapter. The simplified stress-strain curves for all samples are constructed by selecting the final data point of each increment and presented in Figure 5-9 for built samples, as well as Figure 5-10 and Figure 5-11 for bag and cylinder samples of Klettelva respectively. It is important to mention that all stress-strain curves and further stiffness calculations are based on this simplified assumption.

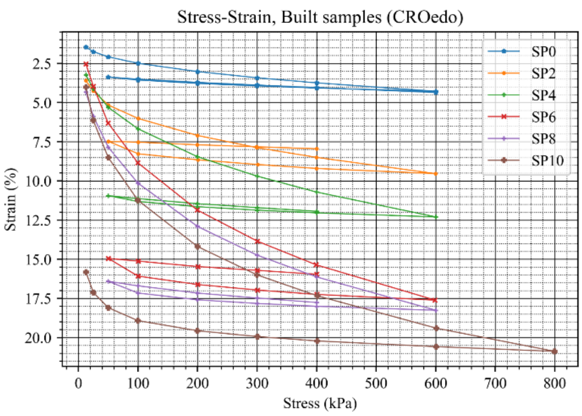


Figure 5-9 Simplified stress strain curve in all built samples tested with ***CROedo****.*

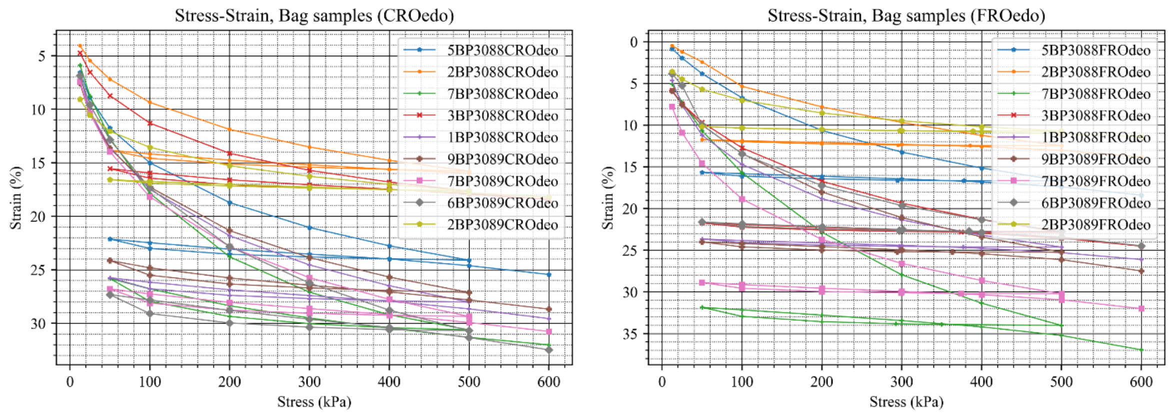


Figure 5-10 Stress strain curve for Klettelva bag samples-left figure: ***CROedo****-right figure:* ***FROedo***

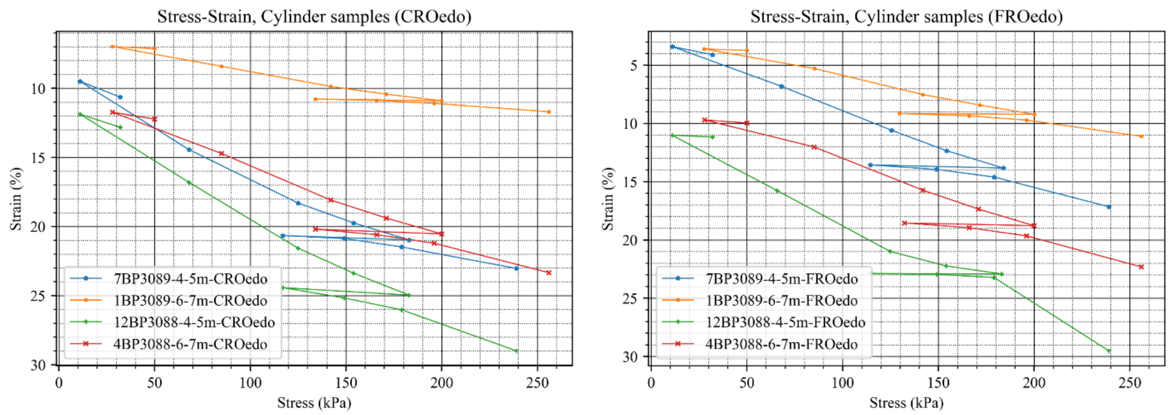


Figure 5-11 Stress strain curve for Klettelva cylinder samples-left figure: ***CROedo****-right figure:* ***FROedo***

5.2.2 Stiffness (oedometer modulus)

By calculating the derivative of the stress-strain curve and utilizing Equation 2-2 to compute the Oedometer modulus, modulus graphs are generated and depicted in this section. The graphs illustrate the relationship between the oedometer modulus and average stress. The resulting oedometer modulus curves are depicted in the following figures, separately for the loading, unloading, and reloading increments of all the constructed samples and collected samples from Klettelva.

<u>Built samples</u>

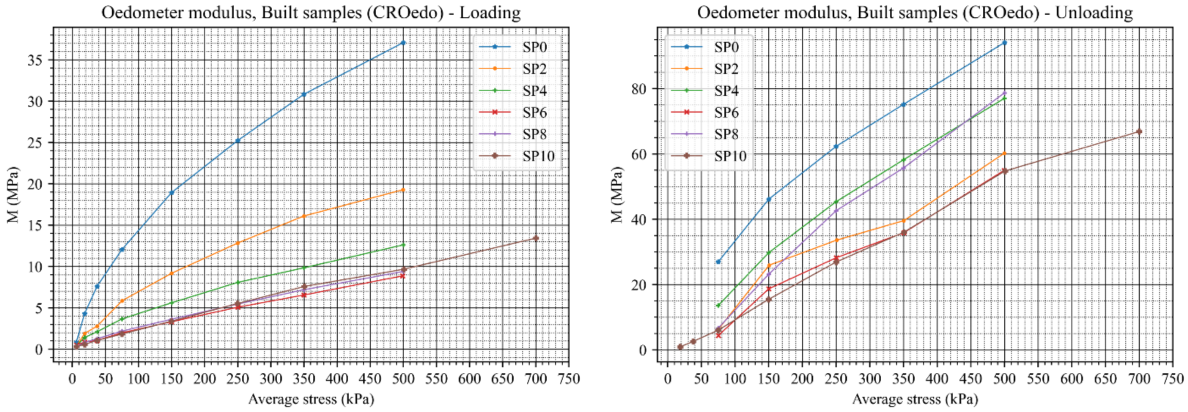


Figure 5-12 Oedometer modulus (M) versus average stress in built samples-left figure ***<u>Loading</u>****-right figure:* ***<u>Unloading</u>***

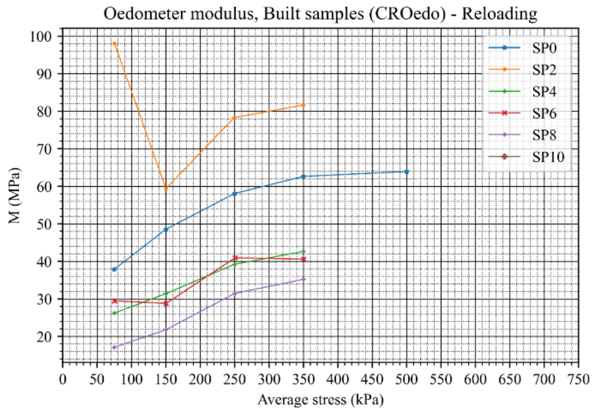


Figure 5-13 Oedometer modulus (M) versus average stress in built samples- ***<u>Reloading</u>***

Klettelva bag samples

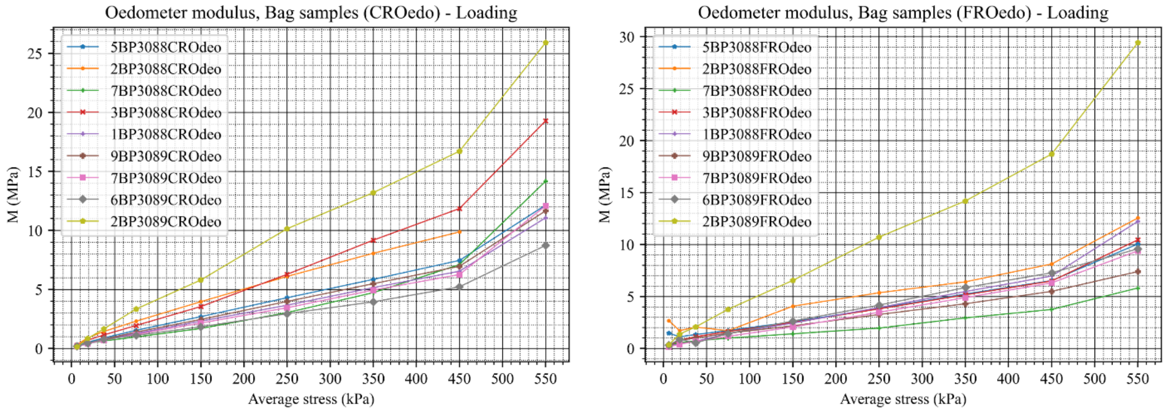


*Figure 5-14 Oedometer modulus (M) versus average stress in Klettelva bag samples-**Loading**- left figure: **CROedo**-right figure: **FROedo***

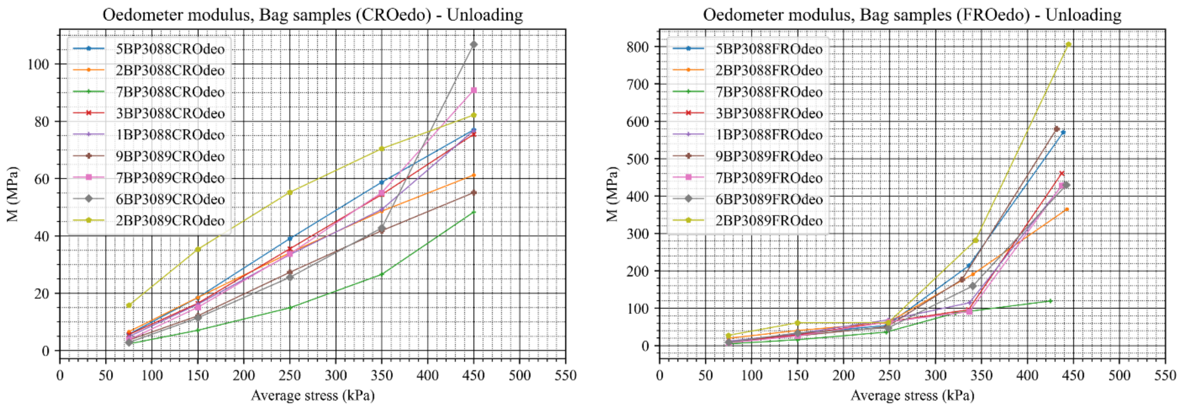


*Figure 5-15 Oedometer modulus (M) versus average stress in Klettelva bag samples-**Unloading**- left figure: **CROedo**-right figure: **FROedo***

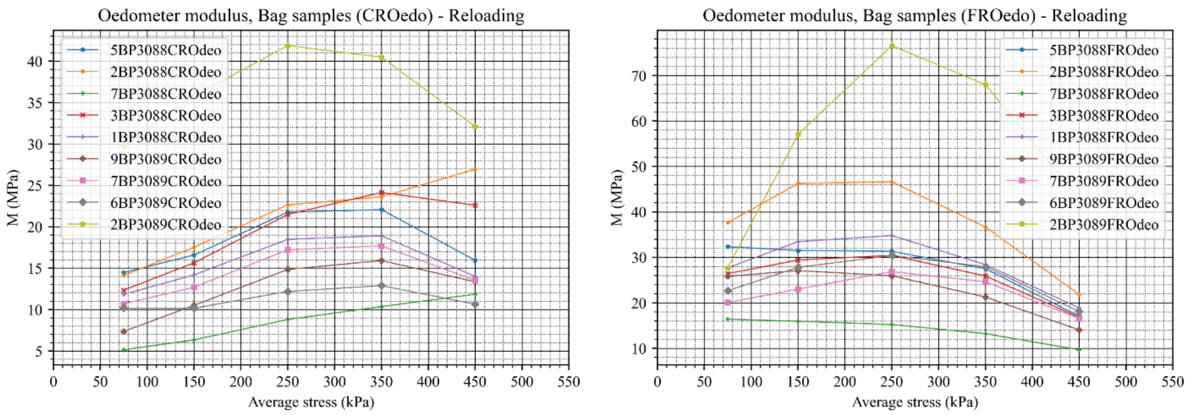


*Figure 5-16 Oedometer modulus (M) versus average stress in Klettelva bag samples-**Reloading**- left figure: **CROedo**-right figure: **FROedo***

Klettelva cylinder samples

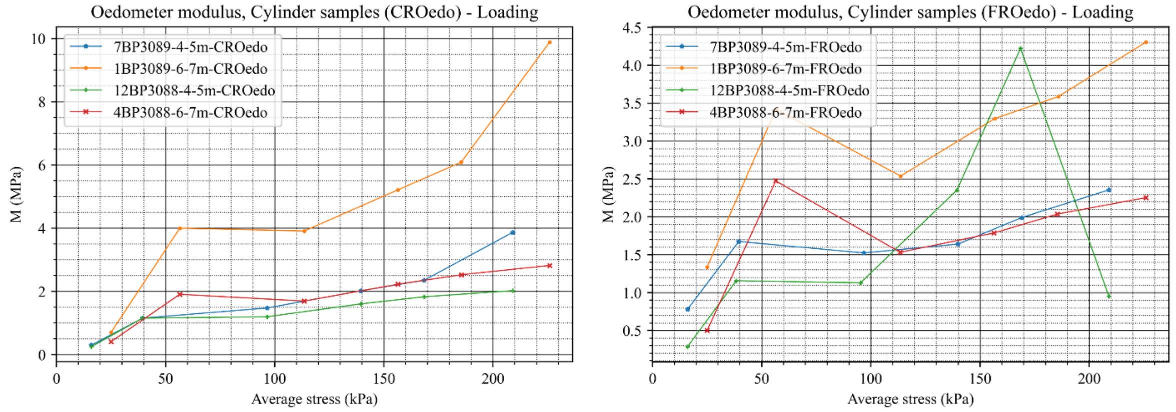


*Figure 5-17 Oedometer modulus (M) versus average stress in Klettelva cylinder samples-**Loading***

*left figure: **CROedo**-right figure: **FROedo***

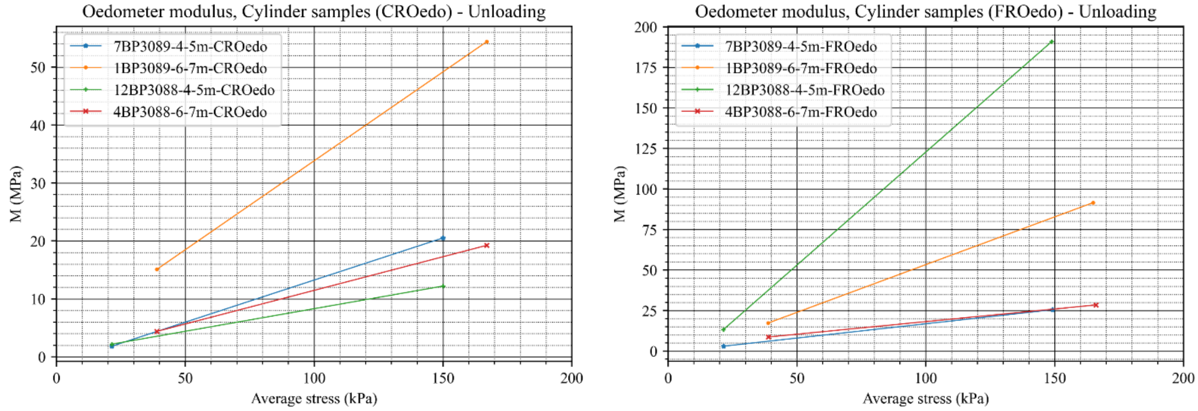


*Figure 5-18 Oedometer modulus (M) versus average stress in Klettelva cylinder samples-**Unloading***

*left figure: **CROedo**-right figure: **FROedo***

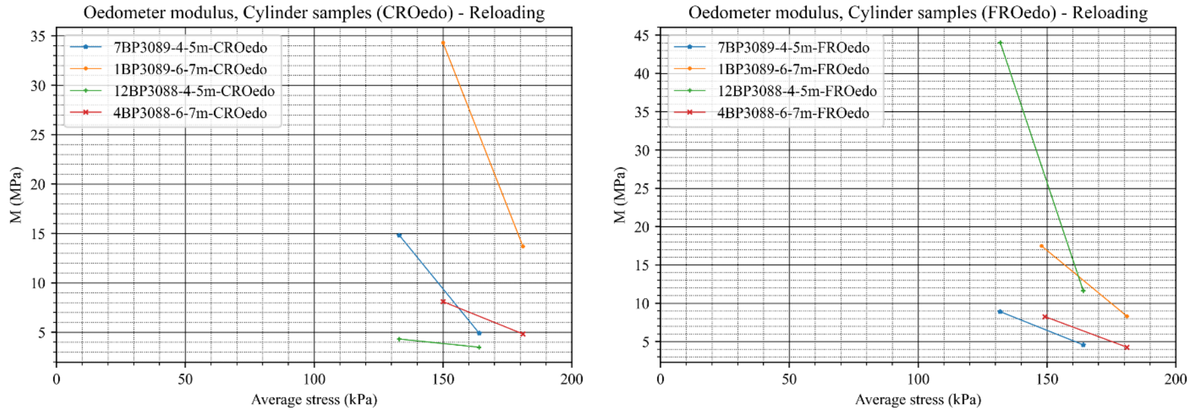


*Figure 5-19 Oedometer modulus (M) versus average stress in Klettelva bag samples-**Reloading***

*left figure: **CROedo**-right figure: **FROedo***

The representative modulus number (m) has been determined by using Equation 2-7. Modulus numbers are summarized for all samples in the following figures.

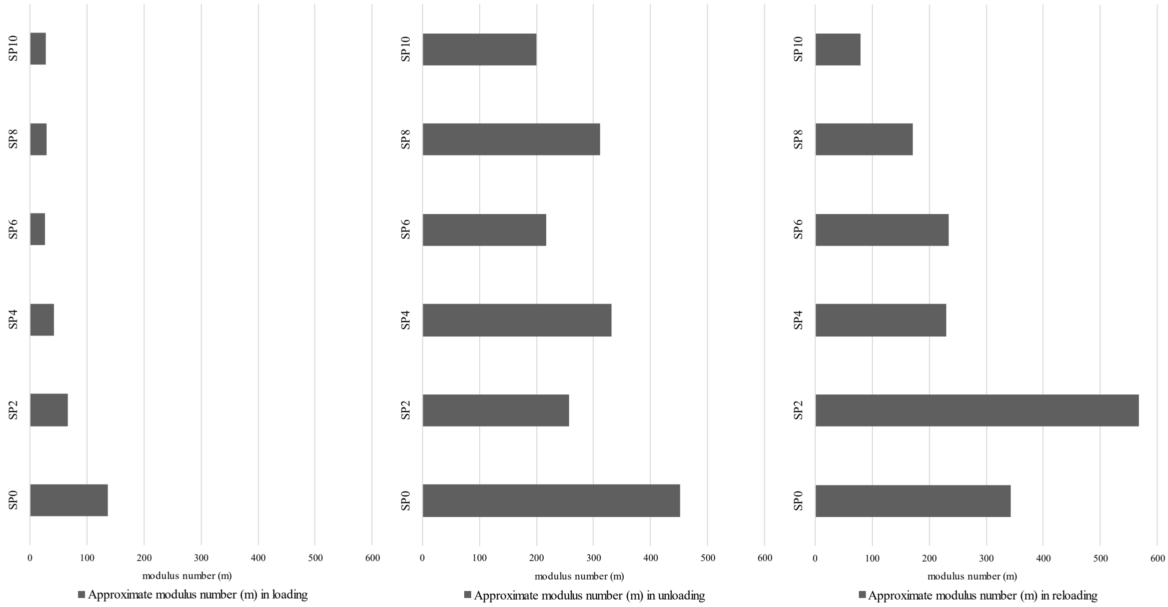


*Figure 5-20 Comparison of modulus number in loading, unloading, and reloading increments on built samples-**CROedo***

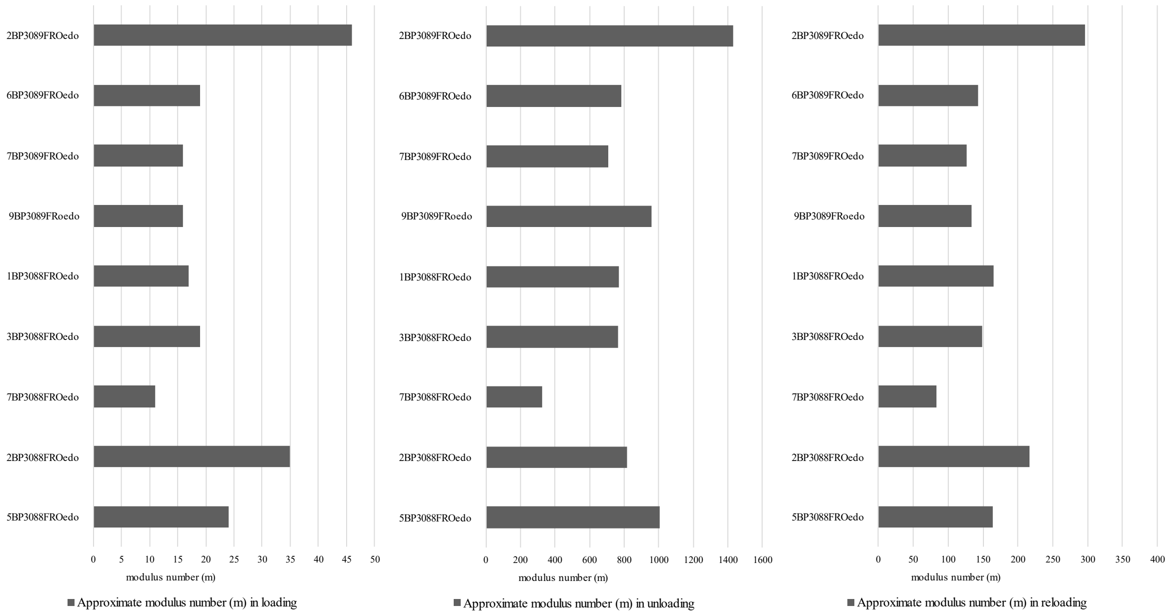


*Figure 5-21 Comparison of modulus number in loading, unloading, and reloading increments on Klettelva bag samples- **FROedo***

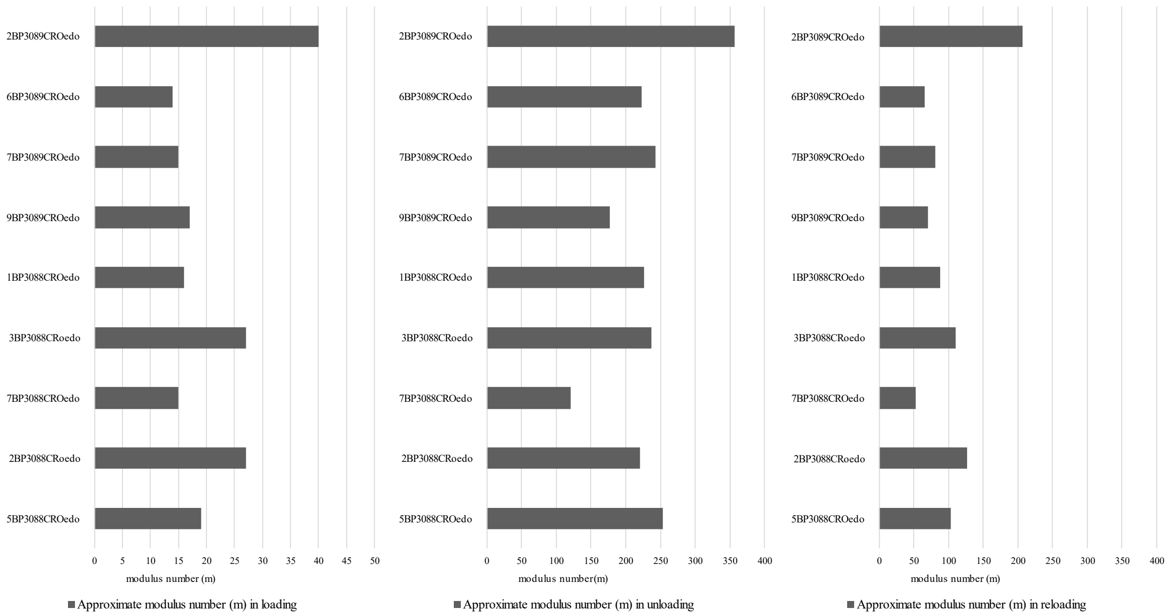


Figure 5-22 Comparison of modulus number in loading, unloading, and reloading increments on Klettelva bag samples- ***CROedo***

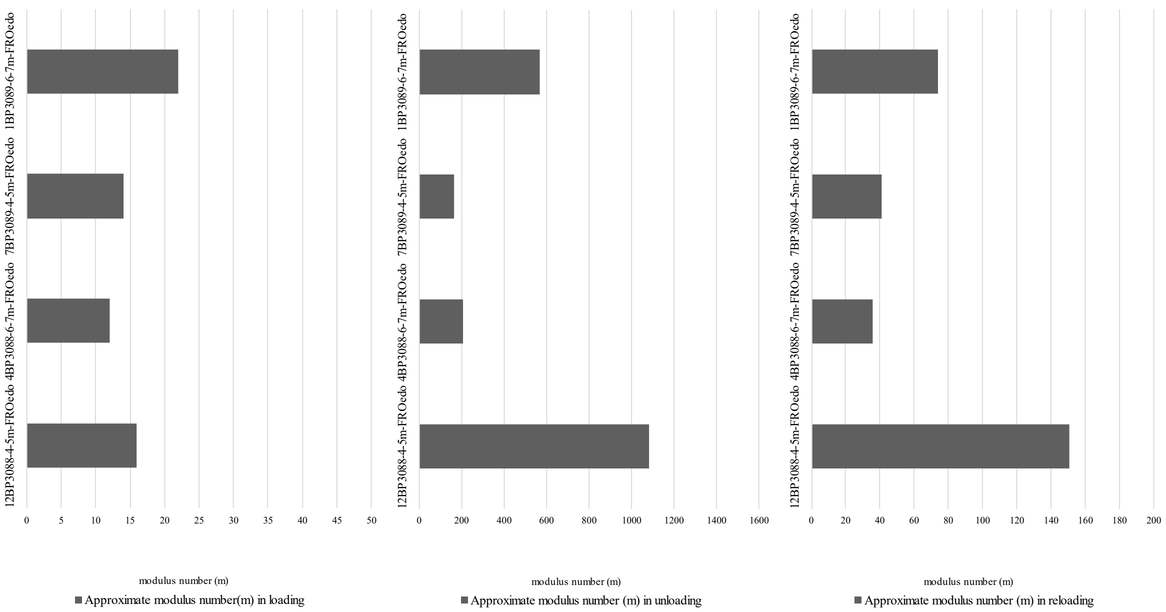


Figure 5-23 Comparison of modulus number in loading, unloading, and reloading increments on Klettelva cylinder samples- ***FROedo***

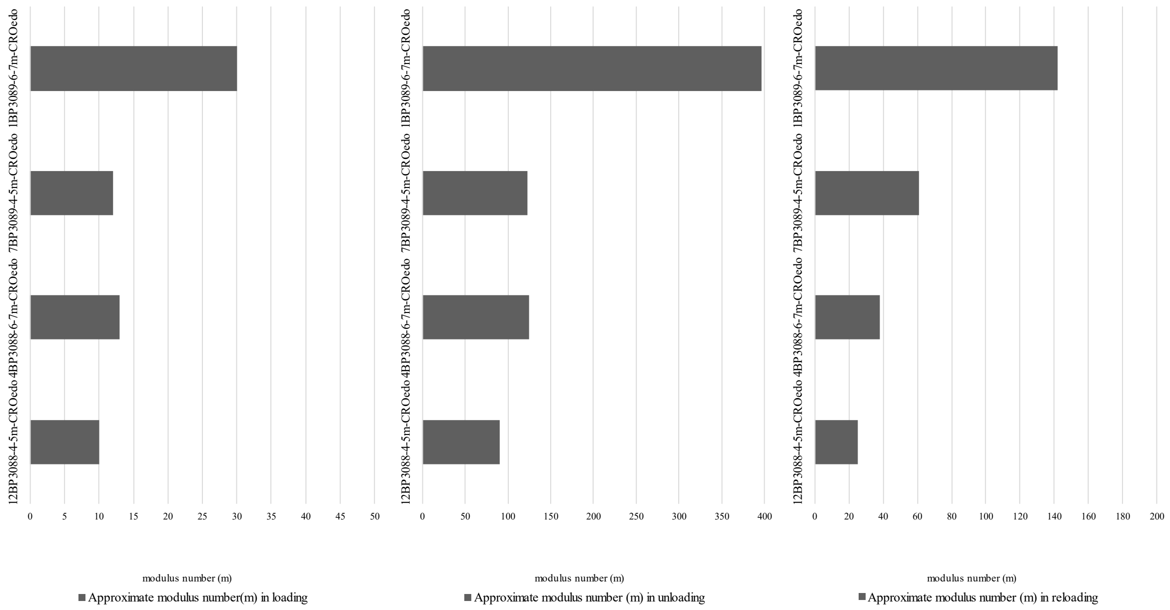


*Figure 5-24 Comparison of modulus number in loading, unloading, and reloading increments on Klettelva cylinder samples-**CROedo***

5.2.3 Time resistance

The main objective of this section is to present time resistance numbers in order to be able to assess whether the samples exhibit significant deformation over time. Time-dependent properties of soil, including continuous settling (creep), can be estimated by analyzing time resistance and creep resistance numbers. Time resistance (R), which is the reciprocal of the rate of strain, has been calculated by dividing the change in time over change of strain using Equation 2-9 as discussed in theory. Generally, there is typically increases of time resistance over time which the results have confirmed the increase over time.

To obtain smoother and simplified data, the approach of a time running window has been employed. The time running window approach, also known as the sliding window approach or rolling window approach, eases the process of capturing the general trend in data. In this approach, a fixed-size window or a time interval moves over a sequence of data points, and computations or analyses are performed on each window as it slides through the data. The window size and sliding step determine the amount of overlap between successive windows. By considering a window of data points at a time, it allows for the examination of local characteristics and enables the detection of changes or patterns over time.

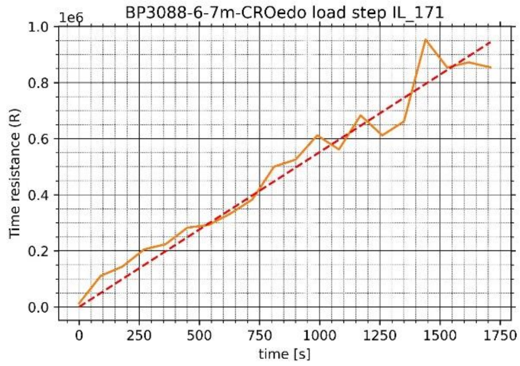


Figure 5-25 One example showing how creep number has been calculated.

The fixed-size window in this study has been selected as 20 logs of data which corresponds to almost every 1.5 to 3 minutes. One can consider other window sizes for the approach if not a good trend discovered. The effect of the size of the window (every 10,20 or 30 log) will be discussed in the next Chapter. The rough prediction of the time resistance and its evolution over time (R-t graphs) with averaging rate/window size of 20 can be found in Appendix E.

Creep resistance number (r_s) as the linear relationship between time resistance (R) and time (t), which is also observed as the slope of the graphs of time resistance versus time (R-t graphs) have been calculated for all graphs and summarized in the graphs below. Figure 5-25 illustrates an example of obtaining time resistance/creep number.

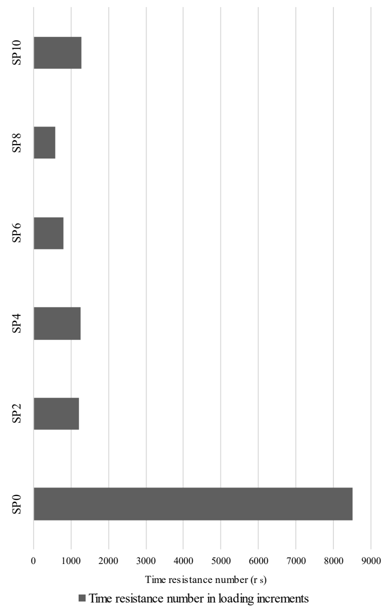


*Figure 5-26 Selected representative creep number in loading increments on built samples-**CROedo***

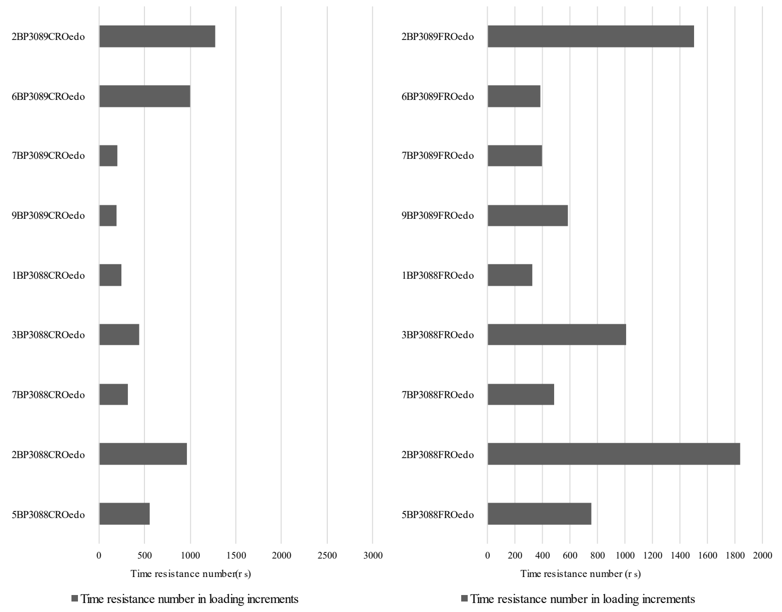


*Figure 5-27 Comparison of Selected representative creep number in loading increments on Klettelva bag samples-left figure: **CROedo-** right figure: **FROedo***

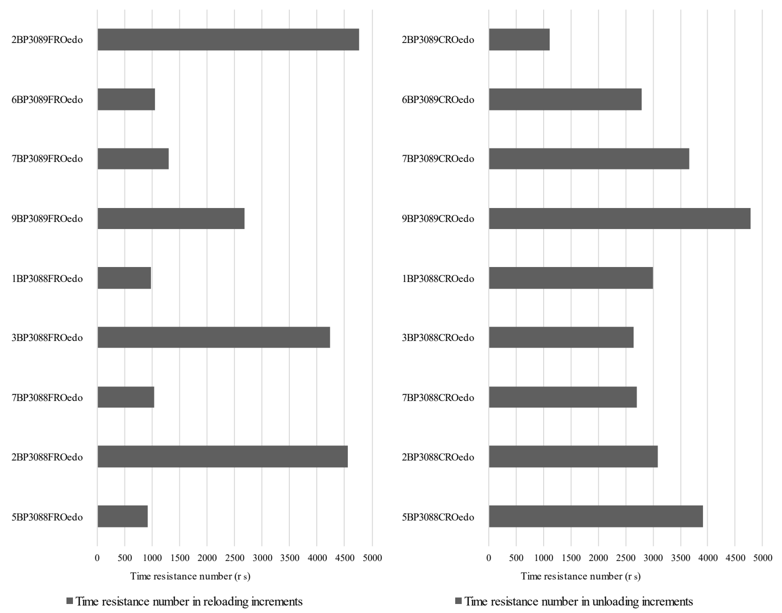


*Figure 5-28 Comparison of Selected representative creep number in reloading increments on Klettelva bag samples- left figure: **CROedo-** right figure: **FROedo***

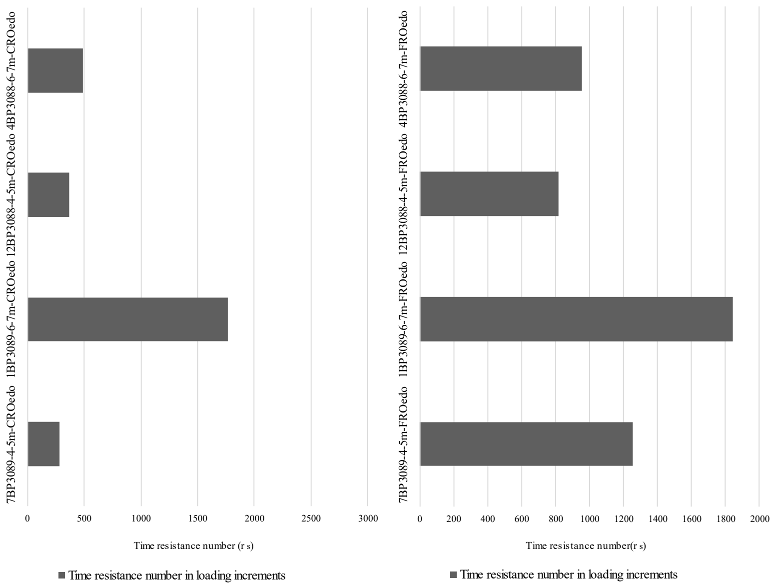


*Figure 5-29 Comparison of creep number in loading increments on Klettelva cylinder samples left figure: **CROed**o- right figure: **FROedo***

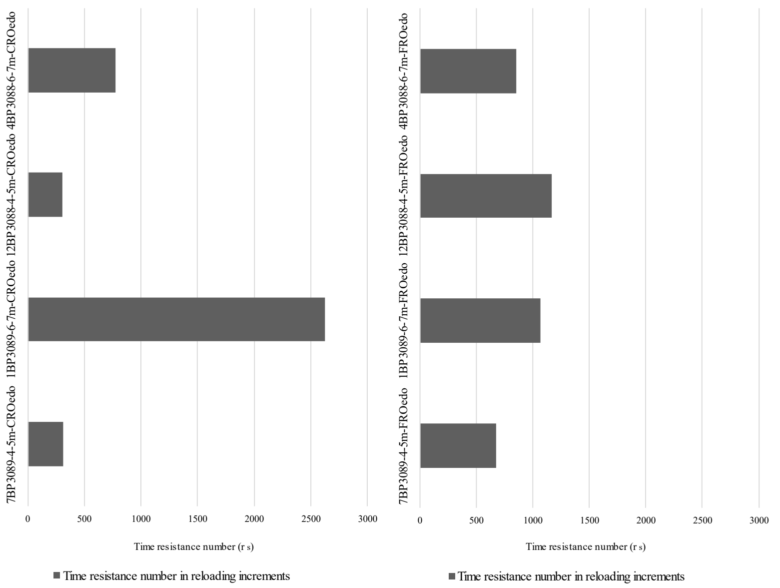


*Figure 5-30 Comparison of creep number in reloading increments on Klettelva cylinder samples left figure: **CROed**o- right figure: **FROedo***

Chapter 6 Discussion

This study tried to analyze the mixtures of various soil samples which are mixed with organic content from different perspectives which included evaluating the physical properties such as water content and grain size analysis in addition to a major part of the study which was the oedometer test.

In Chapter 5, the initial results of these tests were visualized, however, it is important interpret the results such that a practical base could be found. This is extremely important since, as mentioned in Chapter 1, one of the most important motivations for this study is the real-life construction of the new E39 Betna-Hestnes road. More specifically, when it comes to design purposes, geotechnical engineers usually tend to use more readily available relationships and parameters. Therefore, this Chapter would try to build a foundation for finding possible relationships using different aspects of the materials and tests used and conclude on the practical methods such as preloading with a more specific focus on the creep properties of such soils.

Moreover, the focus when interpreting the relationships is on the oedometer results with the other tests acting a support role. As mentioned earlier, two separate groups of Fixed ring oedometer (FROedo) and Compressible ring oedometer (CROedo) tests were conducted. Additionally, three separate groups of materials were also tested using these methods. It is of crucial importance to note that here the focus will be *mainly* on the cylinder samples that were tested using CROedo. The reason for this choice is that first, the cylinder samples were better preserved before testing, and second, the CROedo is believed to have overcome some the shortages in FROedo for sandy materials.

6.1 Deformation trends

When analyzing the oedometer tests, the first and foremost graph to analyze is the time-deformation curve. This study aimed to analyze the samples using both FROedo and CROedo. Total deformation of a sample during an oedometer test depends on the loading and the total initial height of the sample. For this study, while the total height of the FROedo tests were the actual standard size of 20mm, the height of the CROedo tests varied closely around 50mm. Therefore, rough comparison could be made for all CROedo samples and FROedo samples separately.

Examining the time deformation curve of the “built samples” generally revealed the direct effect of organic contents in the deformation. The effect seems also notable when referring to these samples such that for example for a sample with 2% peat, the total deformation is almost double the pure sand, while this number grows to four times the pure sand for the sample with 6% organic content.

The same conclusion could also be observed generally for both types of Klettelva samples. However, for these samples, comparison is not as straightforward as for the built samples. When interpreting the samples from Klettelva, it is essential to consider multiple factors that can influence the results, including the soil type (grain size distribution), water content, and the contribution of peat. Peat percentage does not solely determine the outcomes. This can be exemplified by the disparity in deformation between 2BP3088 and 2BP3089, despite both samples containing the same peat percentage, where the latter exhibits more deformation.

These differences can be attributed to variations in sample collection locations within Klettelva and differences in grain size distribution. For example, it is anticipated that specimen 9BP3089 will exhibit a higher degree of deformation due to its approximately 9% peat content. However, it is plausible that the presence of varying soil composition and clay content may result in a stiffer response than initially anticipated. Another crucial factor to consider is the sample preparation procedure, which may have a more pronounced effect on tests conducted with the CROdeo. The author made efforts to prepare the samples with the same density calculated from the cylindrical samples; however, it proved to be a challenging task for Klettelva samples. Numerous parameters hindered proper compaction and hindered the achievement of the desired density during sample preparation. These parameters include:

- **Organic content:** Peat, being an organic material, tends to retain water and undergo swelling when saturated. This can disrupt the compaction process and hinder the achievement of desired density.
- **Water content:** Excessive water content in the mixture can contribute to increased swelling.
- **Particle size distribution:** The presence of different-sized particles in the soil mixture can affect its compaction behavior. Inadequate grading or a significant variation in particle sizes may result in poor compaction and uneven distribution of forces during tamping.
- **Type of soil:** Certain types of soil, such as highly plastic clays, can exhibit swelling behavior when in contact with water. The interaction between water and clay minerals can cause the material to expand, making compaction challenging.

Table 6-1 shows a summary of the total deformation for the tested materials of Klettelva and compare their water content, organic and nature of the soil.

Table 6-1 summary of total deformation at 200 kPa for all tested Klettelva materials with corresponding characteristic of samples

Type of the sample	Sample label	Water content (%)	Organic content (%)	Fine portion (%)	Coarse portion (%)	Total deformation at 200 kPa (mm)
Klettelva bag samples	5BP3088	79	5	92	6	8
	2BP3088	58	2	93	7	4
	7BP3088	96	7	84	16	10
	3BP3088	56	3	98	2	7
	1BP3088	60	1	99	1	10
	9BP3089	94	9	98	2	7
	7BP3089	58	7	99	1	13
	6BP3089	61	6	97	3	12
	2BP3089	49	2	100	0	8
Klettelva cylinder samples	12BP3088-4-5m	90	12	90	10	13
	4BP3088-6-7m	80	4	89	11	8
	7BP3089-4-5m	48	7	76	17	11
	1BP3089-6-7m	40	1	64	36	5

Assessing creep from time-deformation curves

Results obtained from the time deformation curve provide a reasonable level of assurance regarding the adequacy of the time duration allocated for each loading increment. However, for the purpose of observing creep during unloading and reloading, it may be recommended to include additional time slots. It is evident that the deformation changes become negligible within the recommended duration, particularly during the loading phase. Samples containing a high percent of organic content can be an exception and the deformation changes will take longer time to be dissipated over time. The slight slope in the tale of the time deformation curve in the cylinder Klettelva sample with 12% peat confirms this claim, indicating that there might be an inadequate duration for load steps for this sample.

Nonetheless, it is worth noting that a slight slope was observed in the reloading steps of the time deformation curve (Highlighted in Figure 6-1). This observation suggests that extending the duration of the reloading stages may yield more reliable results in terms of the anticipated deformation specifically when samples contain a higher percentage of organic matter. Moreover, when dealing with a mixture of peat and silty sand or silty clay, it is advisable to exercise greater caution when determining the duration of load increments to facilitate better observation and analysis of deformation changes.

Upon examination of Figure 6-1, a notable observation can be made regarding the time deformation curves. Specifically, it can be observed that the end portion of the curves exhibits a nearly flat behavior for the sample labeled as 4BP3088-6-7m. Conversely, in the case of sample 12BP3088-4-5m, the tail of the curve does not exhibit a flat trend. The slope of the tail in the time deformation curve can serve as an indication of the level of creep observed in the field. A steeper slope suggests less creep (corresponding to a higher creep number), whereas a flat tail indicates that the time duration for the applied load increment may have been insufficient to observe significant creep. Despite the sand and silt content being similar in both samples, as indicated by the grain size distribution results, the notable difference in creep behavior can be attributed to the significant peat content in the former sample, which results in higher water content.

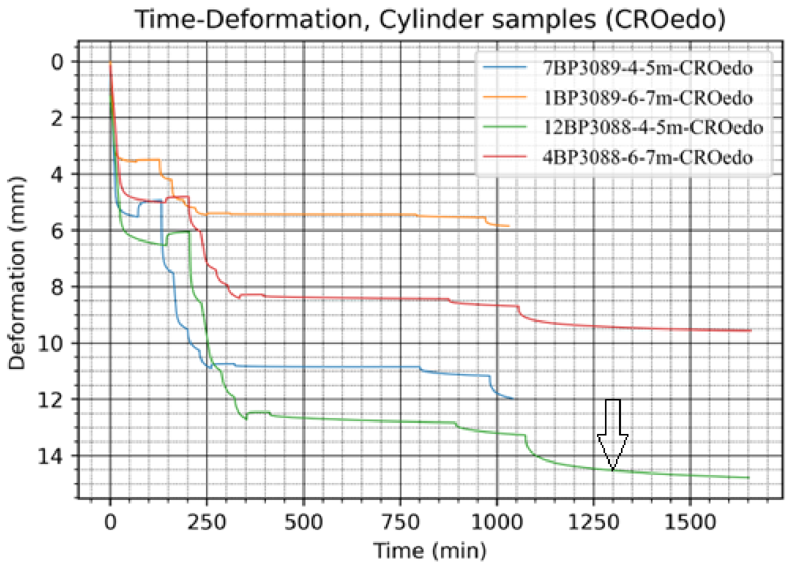


Figure 6-1 Time deformation curve for cylinder samples showing that the slight change of strain over time is increasing as peat content increases.

Particles reformation

Taking a closer look at one of the time deformation curves (Figure 6-2), it is evident that at the beginning of all the tests, there is a significant increase in deformation (marked with black arrows in Figure 6-2). This notable increase in deformation could be attributed to the interlocking of particles with one another. Initially, the particles may undergo rapid rearrangement, leading to significant deformation. However, as the particles gradually adjust and reach a more stable state, the rate of deformation decreases. In other words, as time goes by, the soil particles continue to settle and reorganize, leading to a decrease in deformation rate. Therefore, the beginning of tests is not of much interest, especially in the samples.

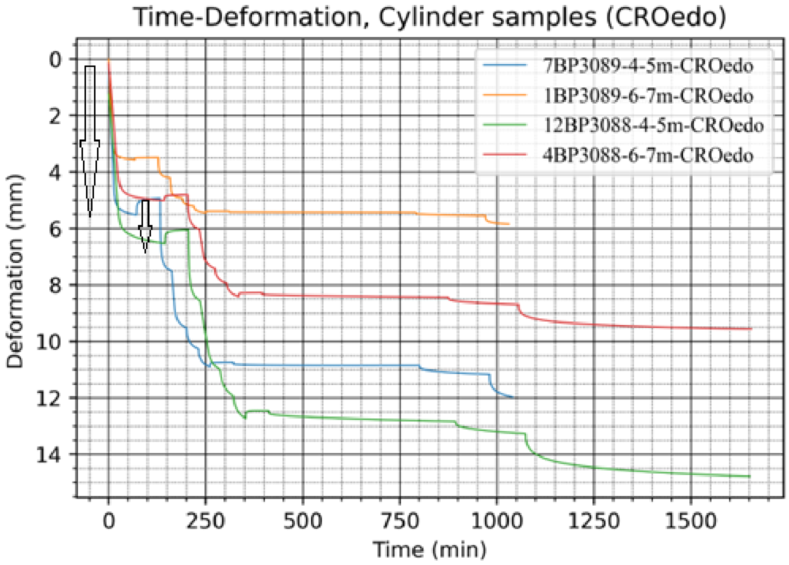


Figure 6-2 Time deformation curve for Klettelva cylinder samples highlighting the rapid deformation in the beginning of tests

6.2 Stress-strain behavior

Stress strain curve as a main tool, shows the behavior of soil in different loading, unloading, and reloading increments. It helps to visually understand the relative degree of compression and/or swelling in oedometer tests. Moreover, the stress-strain curves not only allow for a better direct comparison between different oedometer types, but also open a better insight into the stiffness of the sample under the specified loading, unloading or reloading.

For the built samples, the first impression of change in stress and strain is that the behavior of clean sand without any organic content is according to what is expected. In the case of clean sand, strain levels below 5% are observed under a load of 600kPa. This may result in an important indication that the results from the CROedo could potentially be reliable. Comparing the stress strain of pure sand with previous studies, can confirm the validity of measured deformation in pure sand where range of 2-4% strain observed at the same load level (Ellis et al., 1995; NTNU, 2015).

Furthermore, a significant amplification in strain is observed in built samples when the peat content exceeds **4%.** This finding serves important to geotechnical engineers, signaling the need for possible further sensitivity analysis when dealing with soil containing peat content surpassing even 4%. It is crucial to note that a substantial increase in strain can be anticipated under such circumstances.

When testing the built samples and bag samples, the ultimate stress on the load was set to over 600 kPa, however, in real road constructions this number is typically around 200 kPa. Hence, the interpretation in this study would mainly try to investigate the strain rates at 200 kPa for all samples. With this respect, the strain percentage may be increased as high as double the clean sand with the addition of just 2% peat. This emphasizes the pronounced impact of peat on the soil's response, as it significantly enhances its deformability under moderate loads.

Figure 6-3 to 6-5 present a summary of the attained strain percentages under a 200 kPa load across all tested samples. This comparative analysis allows for a preliminary assessment of the relative magnitudes of strain exhibited within different experimental scenarios.

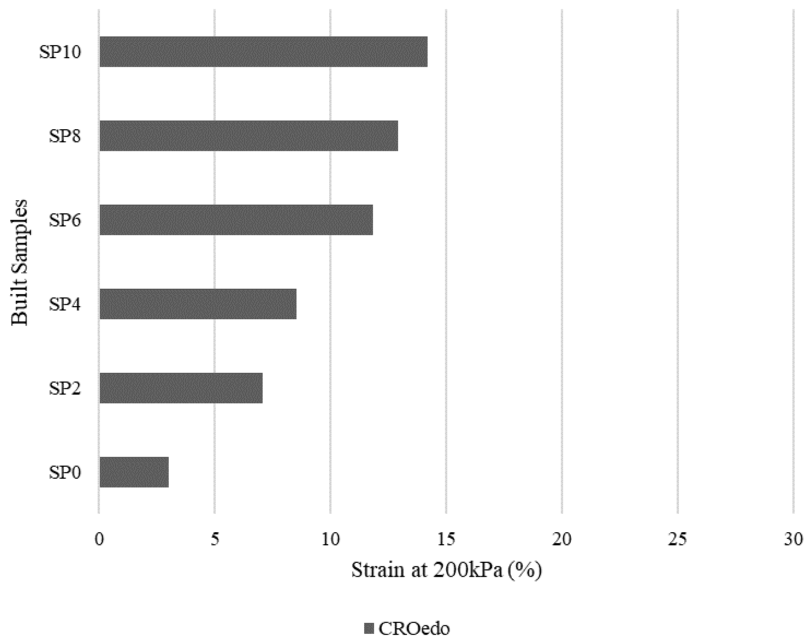


Figure 6-3 strain percentages under a 200kPa load in Built Samples-Tested with CROedo

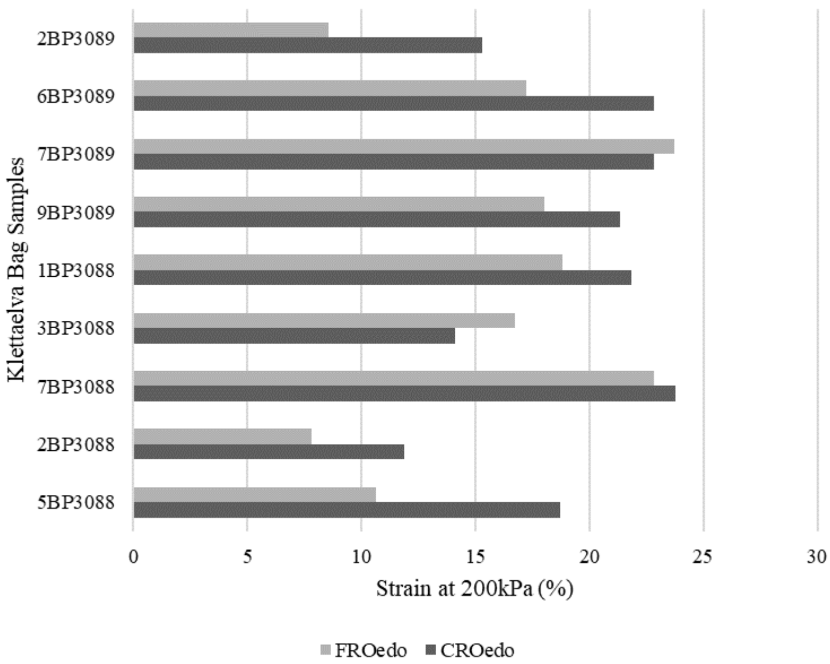


Figure 6-4 strain percentages under a 200kPa load in Bag samples of Klettelva-tested with both CROedo & FROedo.

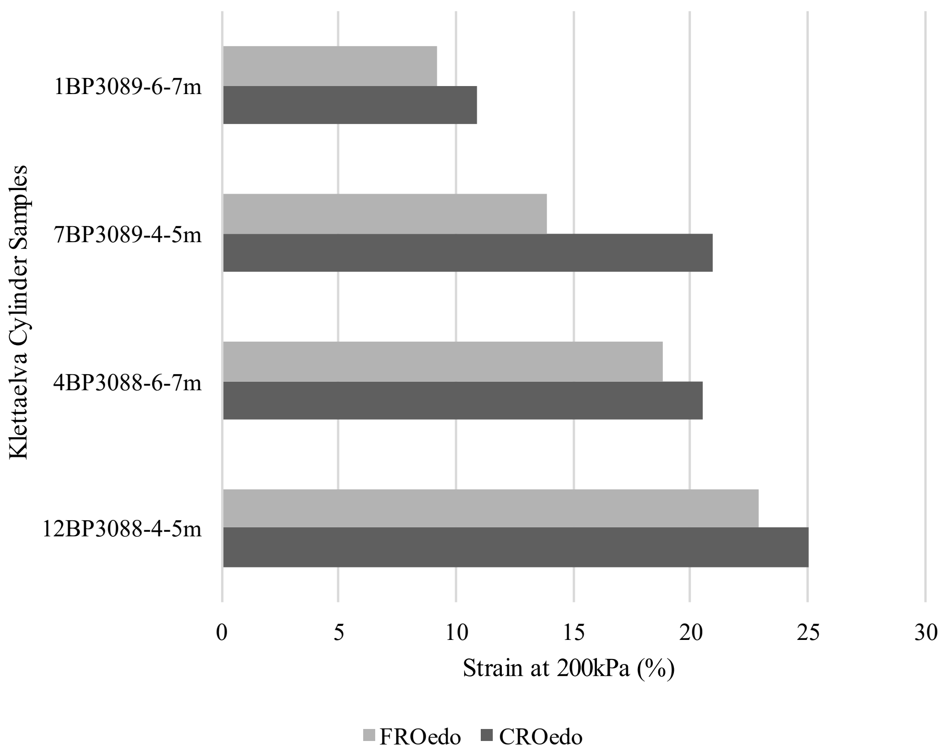


Figure 6-5 strain percentages under a 200kPa load in cylinder samples of Klettelva- tested with both CROedo & FROedo

It shall be noted that the reported percentage of peat content is based on weight percentage. This substantial presence of peat has a profound impact on transforming the behavior of the soil, shifting it from a sand-like nature to exhibiting characteristics more like to peat. This distinction is of utmost importance in understanding and analyzing the response of such soil formations in geotechnical engineering applications.

With respect to the stress history, it is noteworthy to highlight that the number of contact points within the soil skeleton is profoundly influenced by the size distribution of its constituent grains. In soils containing larger grains with limited contact points, even small changes in load can lead to significant changes in effective stress. This can cause the soil to crush until it reaches equilibrium. In contrast, the deformation pattern of soils composed of smaller grains is more complex and may not necessarily involve crushing. small clusters of clay particles are formed, and the soil skeleton will behave as a stiff structure with unities that partly collapse and partly move relative to each other when loaded. Almost no contact between the clay particles occurs since the particles are surrounded by a thin film of adsorbed water, in which the particles flow.

Hence, it is imperative to consistently consider the grain size distribution (GSD) when attempting to interpret the stress-strain behavior of soils. Moreover, it is crucial to acknowledge that the present study incorporates diverse materials characterized by distinct GSDs (See Appendix C).

It is pertinent to assert that a direct comparison of results between CROedo and FROedo may not be completely valid due to the different state of disturbances in the samples. For instance, the cylinder samples employed in CROedo testing represent completely remolded samples, attempting to mimic the in-situ conditions, while the cylinder samples utilized in FROedo testing exhibit less disturbance.

The simplified version of stress-strain curves, as depicted in the preceding Chapter, has been constructed based on the inclusion of final logged data for each incremental step. This approach can be deemed as a judicious methodology, considering the available information.

Unloading properties

The stress-strain behavior during unloading reveals that initial stages of unloading show minimal deformation. Even in samples with organic content exceeding 6%, the deformation at the onset of unloading is negligible. This observation suggests that preloading can have an acceptable influence, particularly when dealing with soils containing organic content. However, it should be noted that as unloading progresses below 100 kPa, swelling becomes more apparent, and this phenomenon should be taken into consideration. Therefore, caution must be exercised during unloading in shallow surface conditions to account for these effects (See Figure 6-6).

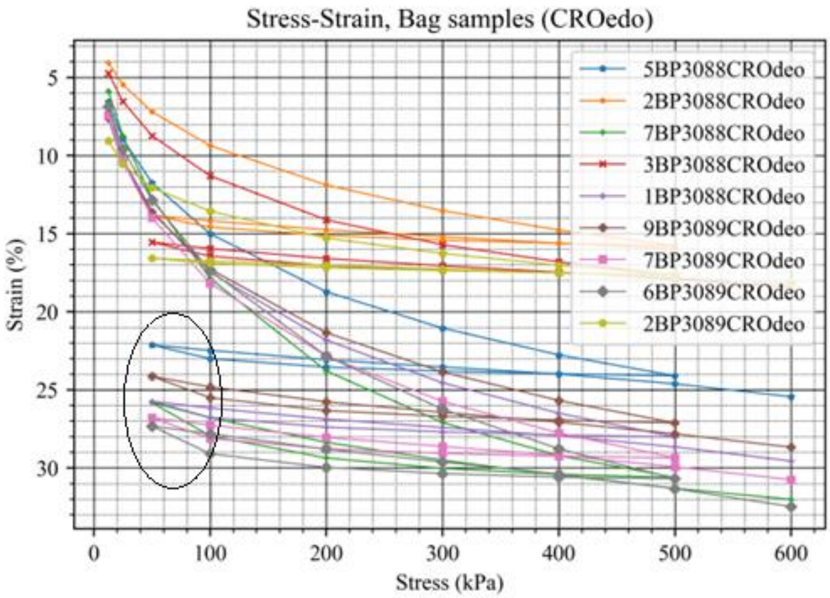


Figure 6-6 Stress strain curve of Klettelva samples- circle marks heaving and swelling.

Table 6-2 summarizes the total degree of swelling in unloading of all samples. Comparing the degree of compressibility during loading with the degree of swelling during unloading provides insights into the behavior of the samples. It is observed that the swelling values are relatively low, suggesting that swelling may not pose significant issues in the field. However, it is important to note that if unloading continues below 100 kPa, the swelling becomes more pronounced. For instance, in the case of Built samples containing 10% peat mixed with sand, the deformation change is around 2% if unloading is limited to 100 kPa. However, deformation can increase up to 5% if unloading progresses to 20 kPa. This highlights the increase in swelling deformation with further unloading below a certain threshold.

Table 6-2 Summary of observed degree of swelling in unloading increments.

Type of the sample	**Sample label**	**Water content (%)**	**Organic content (%)**	**Fine portion (%)**	**Coarse portion (%)**	***Swelling (%)**
Built samples	SP0	16	0	63	37	0.8
	SP2	20	2	63	37	0.6
	SP4	20	4	62	38	0.8
	SP6	20	6	61	39	1
	SP8	25	8	64	36	1.05
	SP10	30	10	63	37	1.9
Klettelva bag samples	5BP3088	79	5	92	6	1.7
	2BP3088	58	2	93	7	0.9
	7BP3088	96	7	84	16	2.2
	3BP3088	56	3	98	2	1.2
	1BP3088	60	1	99	1	1.3
	9BP3089	94	9	98	2	2
	7BP3089	58	7	99	1	2.3
	6BP3089	61	6	97	3	0.91
	2BP3089	49	2	100	0	0.7
Klettelva cylinder samples	12BP3088-4-5m	90	12	90	10	0.5
	4BP3088-6-7m	80	4	89	11	0.3
	7BP3089-4-5m	48	7	76	17	0.3
	1BP3089-6-7m	40	1	64	36	0.2

** Swelling refers to the change in deformation observed during unloading increments. Change in strain is the change rate from 500kPa to 100kPa for bag Klettelva samples and built samples, and from around 200kPa to less than 120kPa for Klettelva cylinder samples.*

***Numbers are roughly estimated based on stress strain curves.*

**** Unloading under 100kPa has not been considered in swelling estimation for Klettelva bag samples.*

The maximum change in deformation in unloading increments observed in FROedo tests is typically less than 0.3%, as indicated by the nearly flat line in the unloading portion of the stress-strain curve. Thus, one can claim that CROedo tests were more successful in capturing the degree of swelling in unloading.

Reloading properties

The strain rate during reloading exhibits significantly lower values compared to the strain rate during loading increments, which confirms the effectiveness of the preloading technique when dealing with soils that have low organic content. It should be noted that after reaching the last maximum stress, relatively same the loading strain rates could be observed in reloading strain rates (Table 6-3).

Table 6-3 Summary of rate of change in deformation in reloading increments for Klettelva samples

Type of the sample	Sample label	Water content (%)	Organic content (%)	Fine portion (%)	Coarse portion (%)	Deformation changes in reloading (%)
Klettelva bag samples	5BP3088	79	5	92	6	3.28
	2BP3088	58	2	93	7	2.40
	7BP3088	96	7	84	16	6.30
	3BP3088	56	3	98	2	2.50
	1BP3088	60	1	99	1	3.80
	9BP3089	94	9	98	2	4.60
	7BP3089	58	7	99	1	6.70
	6BP3089	61	6	97	3	5.10
	2BP3089	49	2	100	0	2.00
Klettelva cylinder samples	12BP3088-4-5m	90	12	90	10	5.60
	4BP3088-6-7m	80	4	89	11	3.11
	7BP3089-4-5m	48	7	76	17	1.80
	1BP3089-6-7m	40	1	64	36	0.92

6.3 Stiffness

Taking the slope of the stress-strain curve and expressing stiffness can give the impression of the ability of a material to resist deformation or displacement under an applied load. Initial interpretation of findings in almost all oedometer samples approves that the oedometer modulus gets lower when there is more contribution of peat in the samples, indicating more deformation and less stiff material as peat-content increases.

As mentioned in Chapter 2, this would mean that it is convenient to use the Janbu's method to express the stiffness behavior. This usually involves finding proper ranges of "modulus number (m)" and "oedometer modulus (M)", respectively. Generally, for normally consolidated clay the typical value of the modulus number is m=8-25. In the case of sand, and silt, this number will be in the range of m=50-100 (silt) and m=100-500 (sand) (Janbu, 1998). Comparing the results for modulus number with expected ranges, can show that sand containing high organic content (more than 6% peat), may be detrimental and decrease the soil stiffness to a great degree, as it can behave even close to clay or even much softer.

The analysis of sand showed that modulus number of pure sand is in the range of 150, which approves the range proposed by Janbu. This fact may be an indication of correctness of proposed apparatus (CROedo) and further interpretations can be safely proceed with respect to this fact.

As mentioned in Chapter 2, finding the modulus number is calculated by the assumption that stress exponent is 0.5, in both sand and mixture of sands and peat, yet by observing the stiffness figures in previous Chapter, specifically Klettelva cylinder samples, one can claim that samples with finer grains tend to behave more like an over consolidated clay and therefore considering the stress exponent of 0.5 could be replaced with more fitted value. Yet, considering the stress exponent of 0.5 seems acceptable for some samples like SP2 with 2% peat and pure sand in built samples. Investigation on more fitted number as stress exponent to find modulus number especially in sand with organic content, can be done if one has more samples as another further study.

It may be worth to note that finding modulus number in unloading and reloading phases, using $M = m\sqrt{\sigma_a \sigma'}$ for all load increments and stress exponent of 0.5, still seems not a good fit for the graph. Yet, parabolic curve can be considered a fair fit in modulus curve (M-σ'), when considering only the load increments up to approximately 300 kPa. This understanding shows that a fair interpretation for finding a representative modulus number, especially in unloading, shall be in a range with an acceptable fitted curve and equation.

It shall be noted that one of the effective parameters in unloading is the continuous presence of water. Swelling can be prevented if the sample is not submerged in water. Designing a cup, for keeping the sample submerged with water during the test, in the modified oedometer reassured this concept and helped to obtain better results especially in unloading increments of tests.

Loading properties

The analysis of oedometer modulus curves in built samples reveals significant findings regarding the stiffness behavior of different soil compositions. Clean sand exhibits considerably higher modulus number in loading, approximately 6.5 times greater than sand mixed with 10% peat. This is accompanied by a substantial reduction in modulus values, ranging from less than

200 in clean sand to 30-70 in samples with 2-10% peat, indicating a notable decline in modulus number and stiffness compared to clean sand.

Examining the Klettelva samples reveals distinct modulus numbers for loading, unloading, and reloading increments. The rough estimation of modulus number in loading for Klettelva samples ranges from 10 to 30 in samples with 1 to 12% peat.

Unloading properties

In built samples, during unloading, the built samples demonstrate increased stiffness, approximately 2 to 5 times than in loading. Examining the Klettelva samples tested with CROedo, reveals modulus numbers for unloading increases significantly to 120-1000 in cylinder samples with 4-12% organic matter. Yet in bag samples this number is between 150 and 400.

When tested in the proposed compressible ring oedometer, the unloading modulus numbers are approximately 10 times higher than loading, while the fixed ring oedometer struggles to accurately record deformation during unloading, resulting in a difference between loading and unloading of 20 times or more in Klettelva samples. This discrepancy may arise from the increased side friction in the fixed ring oedometer during unloading increments.

It shall be noted that one of the effective parameters in unloading is the continuous presence of water. Swelling can be prevented if the sample is not submerged in water. Designing a cup, for keeping the sample submerged with water during the test, in the modified oedometer reassured this concept and helped to obtain better results especially in unloading increments of tests.

Reloading properties

In reloading, clean sand shows higher stiffness, being approximately 1.5 times greater than sand with 4-6% peat, and around 4 times greater than clean sand with 10% peat under a 100kPa load. These findings emphasize the significant impact of peat content on the soil's stiffness response in both loading and reloading scenarios.

The modulus number of Klettelva samples in reloading increments exhibit a range of 30-200 when organic content is between 3 to 12%. Regarding reloading, the stiffness of the Klettelva cylinder samples increases by approximately 4 to 5 times compared to loading increments, with slightly higher ratios observed when using the FROedo test.

Overall, a noticeable decreasing trend in modulus numbers can be observed as the peat contribution increases across all samples. This trend holds true for both the conventional ring oedometer (CROedo) and the fixed ring oedometer (FROedo), with consistent stiffness rankings observed in nearly all samples and increments, particularly during loading.

6.3.1 Effect of grain size distribution on stiffness

The grain size distribution of soil samples holds significant importance as it profoundly impacts their compression and swelling behaviors. Coarser materials with a higher proportion of sand generally exhibit greater compressibility due to their loose packing and larger void spaces, facilitating increased deformation under applied loads. Conversely, the presence of silt and clay fractions influences swelling behavior as these fine-grained particles possess higher surface area and water retention capacity. Consequently, samples containing elevated silt and clay contents are prone to more substantial swelling when exposed to moisture due to their increased ability to absorb and retain water. Moreover, the arrangement and interactions between different particle sizes within the samples, influenced by the distribution of grain sizes, play a crucial role in determining their overall behavior. For example, the presence of silt and clay particles in the interstitial spaces between sand grains can modify the packing and interlocking behavior, potentially reducing compressibility, and influencing swelling behavior. Therefore, it is important to consider all factors together and interpret the behavior of samples.

In the current study, a single type of poorly graded sand comprising 97% sand and only 1% silt was utilized. It is important to recognize that conducting tests with different types of sand, each possessing distinct compositions, may yield divergent findings. However, investigating the impact of various sand types was beyond the scope of the present study. On the other hand, the Klettelva samples exhibited considerable variability in their composition, with sand content ranging from 60% to 90% and silt content ranging from 10% to 40%. In addition, at deeper depths, clay content ranging from 1% to 7% was also observed in the samples. These variations in composition are summarized in Table 6-4 and had a significant influence on the study findings. Therefore, future research endeavors should consider the effect of material type and composition to obtain a more comprehensive understanding of the subject matter.

Table 6-4 Overview of different composition of samples (based on Pariometer results-For detailed information refer to Appendix C)

Sample	Sand content (%)	Silt content (%)	Clay content (%)
BP3088-4-5m	**90**	**10**	**1**
BP3088-6-7m	**89**	**11**	**0**
BP3089-4-5m	**76**	**17**	**7**
BP3089-6-7m	**64**	**36**	**0**
Built samples	**97**	**1**	**2**

The present study, in conjunction with previous research conducted by Johari et al. (2015), provides further confirmation of the influence of grain size distribution on the compressibility of materials and the ability to predict settlement in oedometer testing. Johari's work specifically focused on peat soil and also highlighted the substantial impact of soil particle size. It was observed that samples containing larger particles tended to exhibit a higher fiber content in comparison to samples with smaller particles. This finding underscores the importance of considering the size distribution of soil particles in assessing the compressibility and settlement characteristics of materials, particularly in the context of peat soil (Johari et al., 2015).

6.3.2 Effect of water content on stiffness behavior

The depicted results in Figure 6-7 to Figure 6-9, display the average modulus values of the samples alongside their corresponding water content. The observed relationship between water content and material stiffness indicates that as the water content increases, there is a consistent trend of stiffness increase across all loading, unloading, and reloading increments. This trend can be characterized as a uniform decline in stiffness with the progressive increase in water content.

As the water content in materials increases, they tend to become less stiff due to the following reasons. Firstly, water acts as a lubricant between particles, reducing the inter-particle friction and allowing for easier movement. This results in a decrease in stiffness as the resistance to deformation decreases. Secondly, water fills the void spaces between particles, effectively increasing the volume and reducing the overall density of the material. This leads to a decrease in stiffness since the material becomes less compacted. Additionally, water molecules have a cohesive property, enabling them to form bonds with the material's particles, which in turn weakens the forces between them and contributes to a decrease in stiffness. Moreover, water can also modify the strength of chemical bonds within the material, affecting its overall stiffness. Therefore, the increase in water content leads to a decrease in stiffness because of reduced friction, increased void space, weakened inter-particle forces, and potential alterations to chemical bonds.

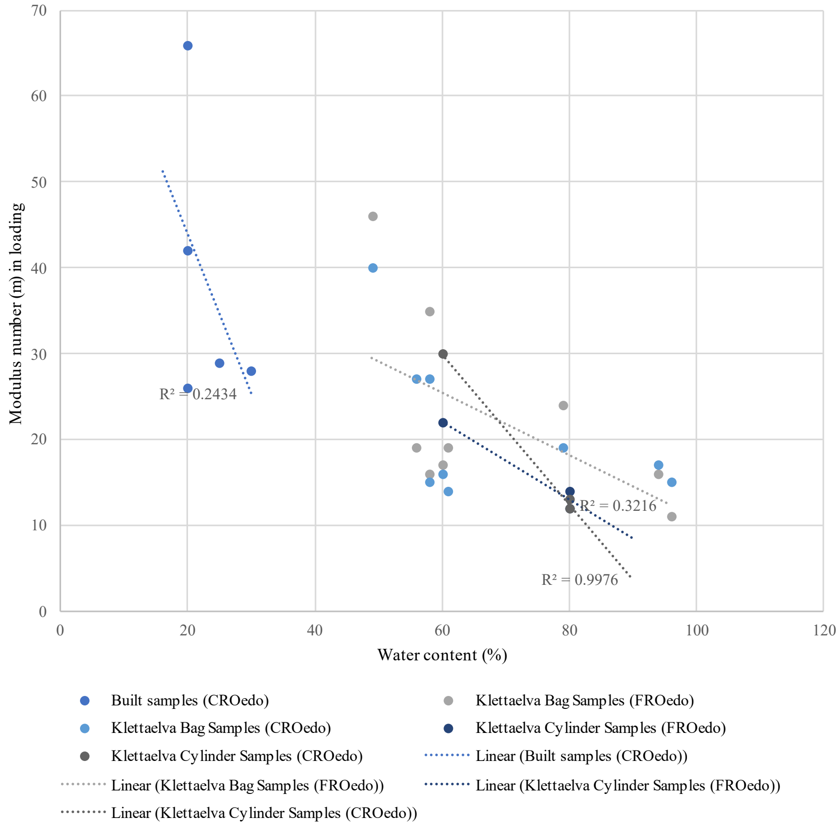


Figure 6-7 water content and average modulus number of all samples in loading increments.

All Klettelva samples exhibit a consistent trend, particularly in loading and reloading increments, indicating a general increase in stiffness with higher water content. Notably, when examining the modulus numbers during unloading increments for cylinder samples tested with CROedo, a higher stiffness is observed overall. This disparity could potentially be attributed to certain factors that warrant further investigation. Nevertheless, the overall trend of decreasing stiffness as water content increases remains evident across these samples. Moreover, it is worth noting that the built sample displays a steeper slope, particularly in loading increments, which could potentially be attributed to the specific composition of materials used in its construction. Further analysis is required to fully comprehend the underlying reasons for these observations.

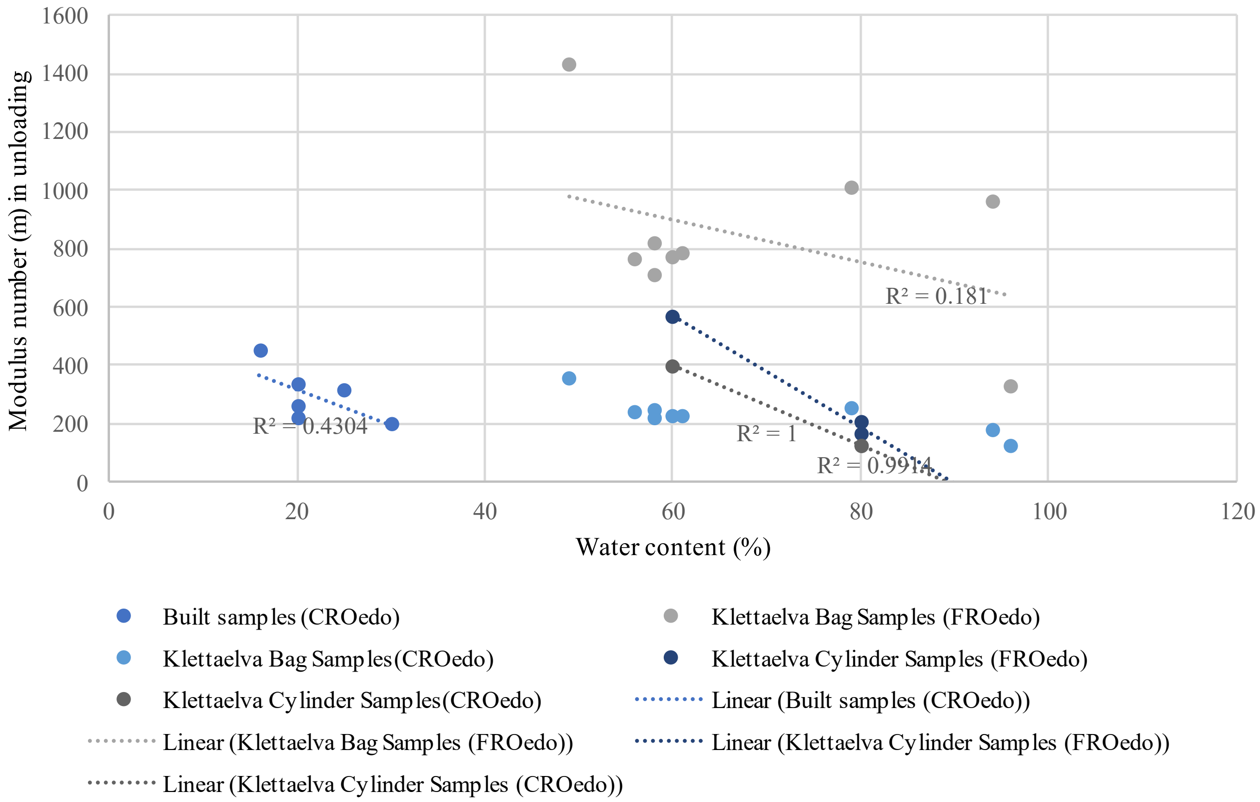


Figure 6-8 water content and average modulus number of all samples in unloading increments.

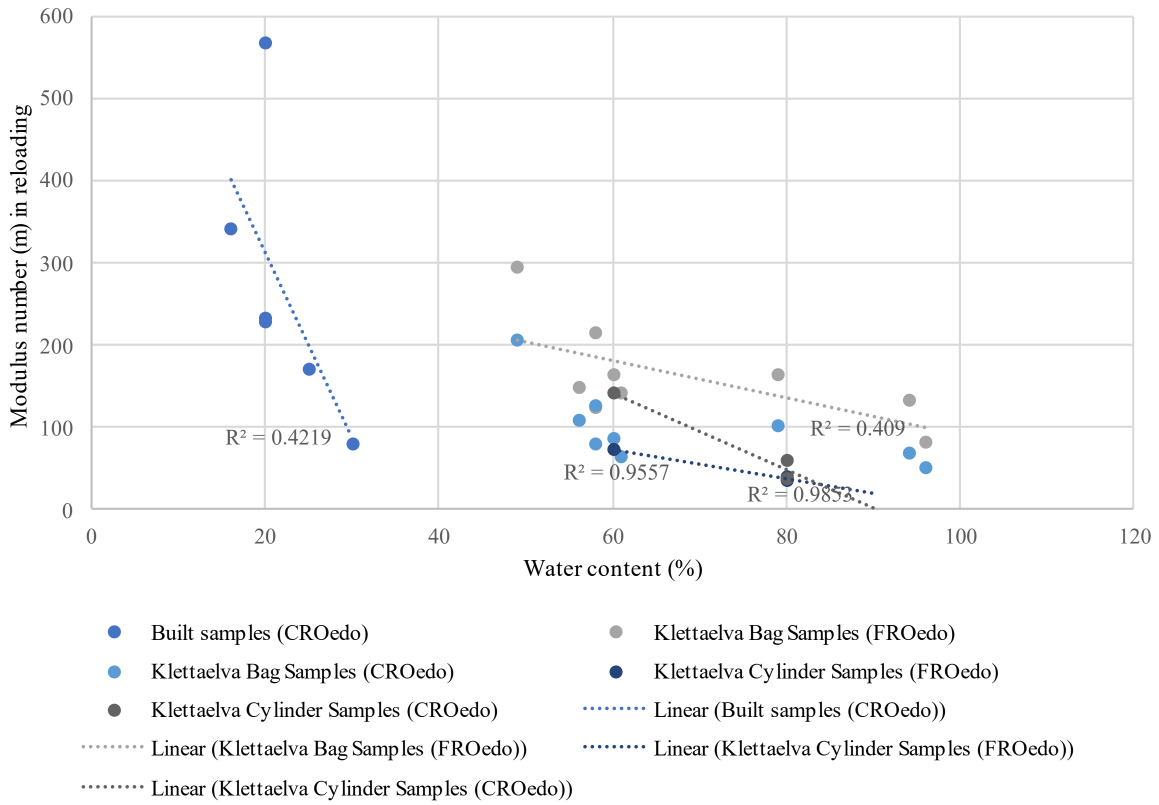


Figure 6-9 water content and average modulus number of all samples in reloading increments

Indeed, it is crucial to consider the grain size distribution when interpreting stiffness data. In the present study, it is noteworthy that the built samples primarily consist of sand (97%) during the decomposition process. On the other hand, the Klettelva material at approximately 6 meters depth comprises approximately 65% sand and 35% silt. Consequently, the distribution of particles and their interlocking arrangement within the Klettelva samples will vary based on the range and distribution of grain sizes. This variation in grain size distribution can significantly influence the stiffness properties of the materials, as the interparticle interactions and overall packing characteristics will be influenced by the specific arrangement of grains. Hence, it is important to consider the influence of grain size distribution when interpreting the observed stiffness trends in the studied materials.

6.3.3 Effect of peat percentage on stiffness behavior

The figures presented in this section depict the samples along with their corresponding peat content and stiffness values, aiming to investigate the influence of peat content on material properties and the correlation between stiffness and organic content. A consistent downward trend is observed across all samples, indicating that as the organic content increases over time, the stiffness of the material decreases. This provides empirical evidence supporting the notion that there exists an inverse relationship between organic content and material stiffness.

As organic content increases in soil, it undergoes several changes that contribute to a decrease in stiffness. Firstly, organic matter acts as a sponge, retaining moisture and lubricating the soil particles. This increased moisture content reduces friction and makes the soil less stiff. Additionally, organic matter aids in the formation of soil aggregates, creating larger clumps. These aggregates provide pore spaces for better air and water movement, resulting in less compacted and stiff soil. Lastly, organic matter helps to reduce soil compaction, which in turn decreases the soil's density and stiffness. These effects may vary depending on soil characteristics and the quantity and quality of organic matter added.

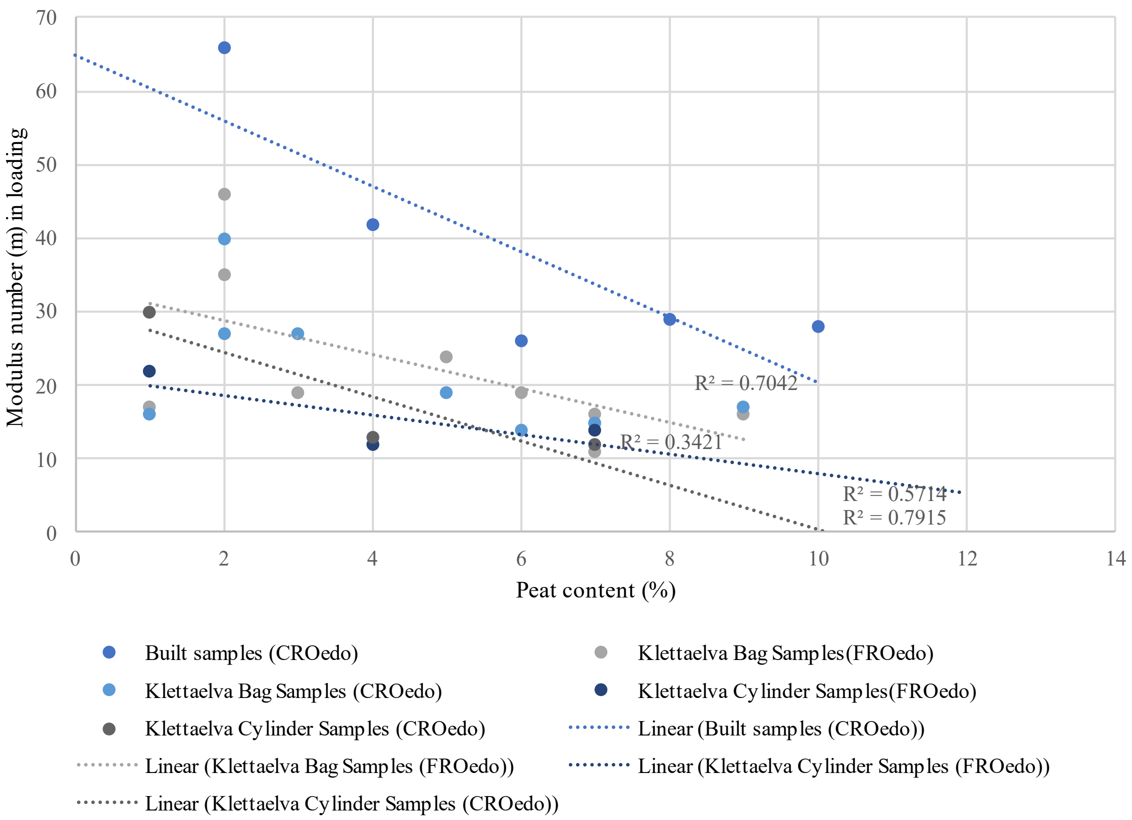


Figure 6-10 peat content and average modulus number of all samples in loading increments.

It is essential to acknowledge that the compositional variability in these samples extends beyond the sole consideration of peat content. To attain a comprehensive understanding of the samples, one must pay attention to the full array of soil specifications. Among these factors, the grain size distribution of the samples assumes significant importance, as mentioned beforehand (Appendix C outlines the grain size distribution of samples in the current study).

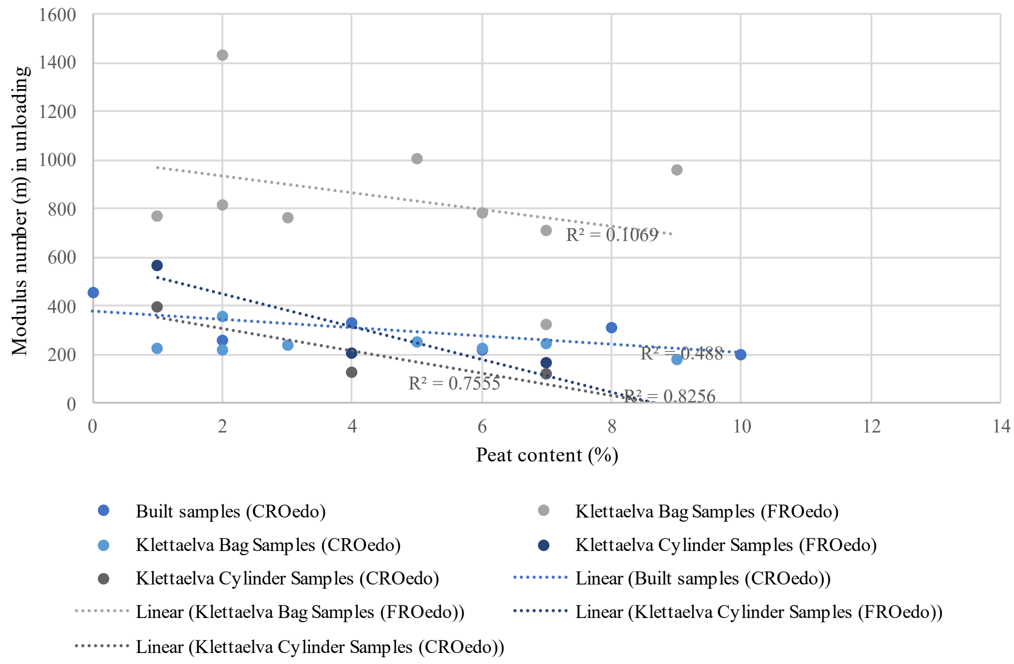


Figure 6-11 peat content and average modulus number of all samples in unloading increments.

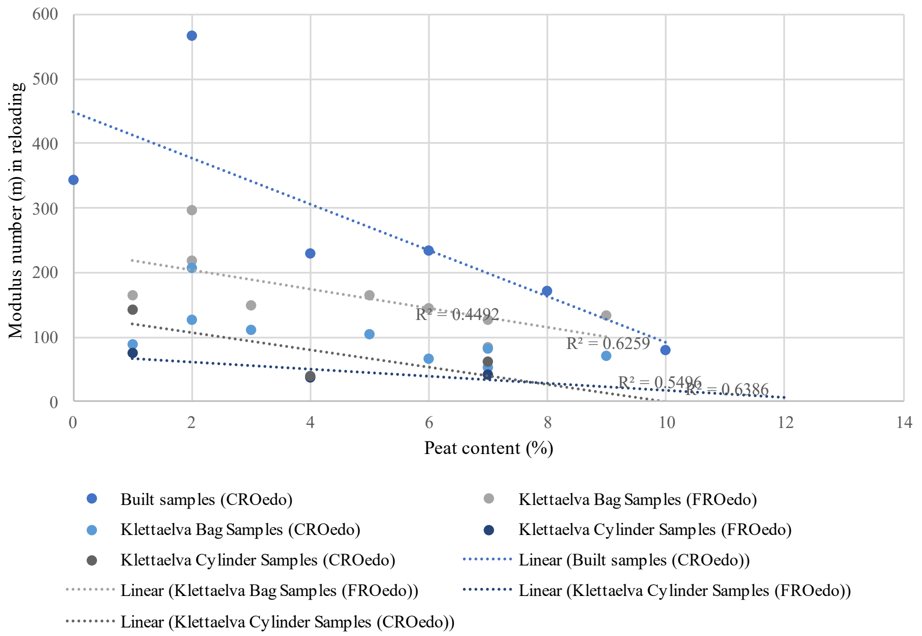


Figure 6-12 peat content and average modulus number of all samples in reloading increments.

6.4 Time resistance (Creep)

As mentioned earlier, one of the most important objectives of this study is to investigate creep properties of soils mixed with organic matters. For this purpose, the main tools and interpretations from the oedometer tests include the observations of time resistance (R) and time resistance number (r_s). Initial obtained results of these parameters were visualized in Chapter 5. As a general principle, higher values of creep number indicate less creep over time. According to previous tests and studies (NTNU, 2015), resistance number (r_s) could vary significantly for different types of soils. For normally consolidated clays, this value may be as low as 100-500, while sands, according to their grain size, may vary from 1000-10000. Over consolidate clays sit somewhere between these values and usually have a time resistance number of 1000-5000.

For this study, initial interpretation is that time resistance (R) is increasing with time continuously, which is according to expectation. It shall be noted that all calculations have been made in each increment separately with a fair range of smoothening with the approach of a fixed size running window.

For the clean sand, the value of time resistance number varies between 4000 to 8000 which indicates a reasonable test was conducted. Nonetheless, when peat is mixed with sand, the value is substantially decreased. Time resistance number goes as down as 450-1000 for these samples indicating the huge effect of creep compared to the bare sand. This obvious decrease is furthermore relevant for all samples with peat, regardless of the peat content.

For the Klettelva samples, the effect of peat content is also obvious. According to both bag and cylinder samples, the value for low peat contents (up to 2%) may vary around 2000; however, this value could be decreased as low as 500, which indicates that the creep properties could be significant for such samples.

In accordance with the provided information, significant magnitudes of creep number suggest that there is no substantial alteration in the behavior of the soil, thereby exhibiting minimal deformation over time within each measured interval. Of particular significance is the examination of creep numbers during unloading stages, or more precisely, the swelling of the soil. The experimental outcomes for constructed samples during unloading stages reveal higher creep numbers in clean sand, as anticipated, when compared to sand mixed with peat. However, creep values in the presence of sand mixed with peat, within the range of 2-8% peat content, are found to be reduced to a quarter of the values observed in clean sand. Generally, In unloading, a delayed swelling is observed in soil containing organic content, yet in the beginning of the unloading stages, swelling cannot be observed.

The degree of smoothing applied to time resistance graphs, or in other words, the window size used for data consideration and analysis, has a direct impact on the measurements and calculations. Including excessive data, utilizing small window sizes, or employing less smooth graphs can result in an inability to capture the overall trend accurately. To evaluate the efficacy of the running-window approach, a sensitivity analysis was conducted on two samples for each specific loading, unloading, and reloading increment. In general, there was a minimal reliance on averaging R values, and no significant changes were observed in the range of creep numbers. However, during unloading increments, the smoothing and averaging of time resistance exhibited a more pronounced influence. This assertion is supported by the example of sample BP3089-4-5m-CRoedo, as depicted in Figure 6-13, which serves as empirical evidence confirming the aforementioned claim.

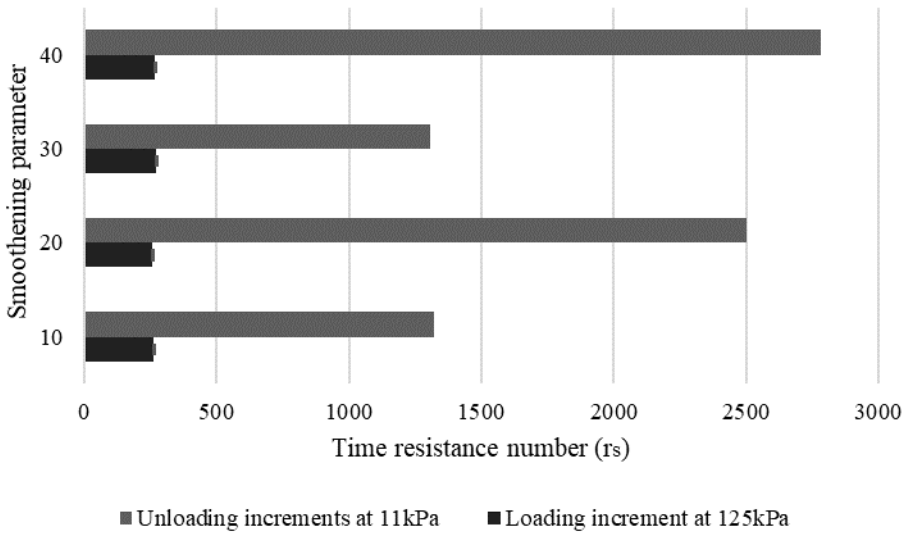


Figure 6-13 Comparison between creep numbers and effect of averaging and smoothening in BP3089-4-5-CROedo

Based on the observed r_s values, a preliminary estimation of creep during loading, unloading, and reloading can be made (Equation 6-1). In the case of pure sand, the estimated creep is negligible at approximately 0.048%. Similarly, when 2-4% peat is added to the sand, the observed creep is not significant, amounting to approximately 0.3%. However, a slight variation in creep behavior is observed in samples containing organic content within the range of 8-10%.

This discrepancy can be attributed to the dominance of peat in influencing the soil behavior when the peat content is around 10% or more.

Equation 6-1 $\varepsilon = \frac{1}{r_s} ln \frac{t_2}{t_1}$

$$clean\ sand\ \&\ r_s = 8500\ \rightarrow \varepsilon = \frac{1}{8500} ln \frac{60}{1} = 0.048\%$$

$$sand + 2 - 4\% peat\ \&\ r_s = 1200 \rightarrow \varepsilon = \frac{1}{1200} ln \frac{60}{1} = 0.3\%$$

$$sand + 8 - 10\% peat\ \&\ r_s = 600 \rightarrow \varepsilon = \frac{1}{600} ln \frac{60}{1} = 0.68\%$$

Using the same equation and the same procedure for Klettelva samples shows maximum 1.5 to 2% strain n loading, while preloading can decrease the estimated strain to roughly 0.5%. Based on the obtained results, it can be concluded that following the loading and reloading stages, a deformation of less than 1% can be anticipated in the long term. However, the tests conducted with the FROedo demonstrated slightly higher deformation values, which still do not pose significant concerns or problems.

6.4.1 Effect of grain size distribution on creep

The grain size distribution of sandy soils can have an effect on creep behavior and long-term deformation. Sandy soils with a well-graded and uniform distribution tend to have lower creep and deformation due to better interlocking between particles. This helps distribute the load evenly and resist particle rearrangement. In contrast, soils with a wider range of grain sizes or poorly graded distributions may experience higher creep and deformation. The presence of fines within the sandy soil can further influence creep, with fines acting as lubricants and increasing particle movement, yet in the current study a comprehensive assessment of all sample characteristics is necessary to accurately interpret the results.

As an example, in analyzing the behavior of the 1BP3089-6-7m-CROedo sample, it is anticipated that a lower creep number would be observed compared to other cylinder samples. This expectation arises from the presence of a higher proportion of fine particles in the sample. However, it is important to note that relying solely on grain size distribution (GSD) analysis may not be sufficient in this context, as the other samples in the study exhibit variations in peat content and water content and got lower creep numbers.

6.4.2 Effect of water content on creep

The depicted results in Figure 6-14 display the average time resistance number of the samples alongside their corresponding water content. The observed relationship between water content and material creep number indicates linear regression in which as the water content increases, there is a consistent trend of creep number increase.

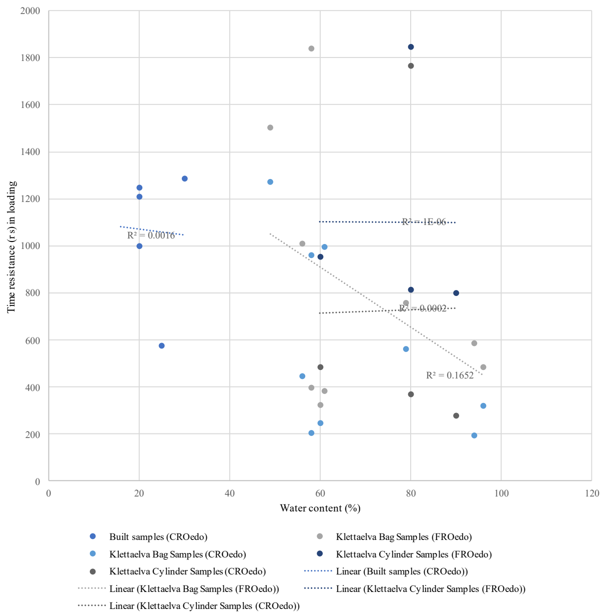


Figure 6-14 water content and average time resistance number of all samples in loading increments

6.4.3 Effect of peat percentage on creep

Figure 6-15 presents the average time resistance number of the samples along with their corresponding peat percentage. The results indicate a relationship between water content and material creep number, suggesting a linear regression pattern. As the water content increases, there is a consistent trend of an increase in the creep number.

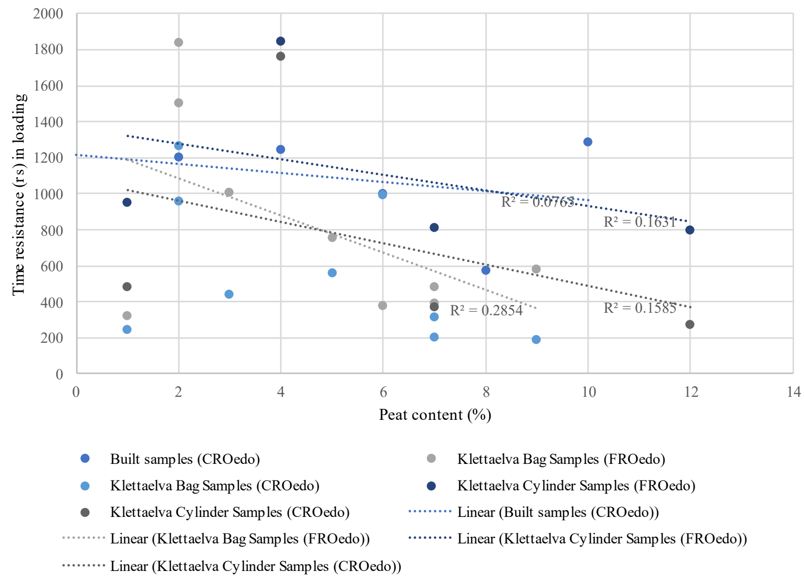


Figure 6-15 peat content and average time resistance number of all samples in loading increments

6.5 Effect of test conditions

6.5.1 Effect of loading rate

In practice, it is important to recognize that preloading procedures can span longer durations, which vary depending on the specific project requirements. For instance, in the present study, a preloading stage of 1-6 months was anticipated, while the oedometer testing was completed within a maximum duration of 24 hours. When referring to creep behavior, it is crucial to consider long-term effects over extended periods, such as 5-10 years, as the results obtained should be applicable for predictions within such timeframes. Consequently, the duration of the applied load during testing holds significance. In situations where the creep parameters are of interest, it is essential for the load duration to considerably exceed the time required for primary consolidation to be fully accomplished. Considering the predominant inclusion of sand or silt materials, which exhibit rapid consolidation, the findings derived from this study may be regarded as valuable for aiding in the initial estimation of long-term creep behavior.

However, it is important to note that the current study did not endeavor to investigate the impact of load duration on the outcomes of oedometer tests. Rather, the focus was on establishing a reasonable load duration within the available timeframe for the thesis and obtaining satisfactory results.

6.5.2 Effect of remolded samples and intact samples

The state of disturbances of a sample, particularly in the case of coarse materials with lower cohesion, can have significant effects on oedometer testing results. Coarse materials generally exhibit less inter-particle bonding and cohesive forces compared to finer-grained soils. As a result, the sample's initial state of disturbance, such as its degree of compaction or particle arrangement, can affect its behavior during oedometer testing. The applied load in oedometer testing causes the sample to undergo deformation, including consolidation and settlement. However, the initial state of disturbance can influence the sample's ability to consolidate and its subsequent compressibility behavior.

Additionally, the state of disturbance in oedometer testing, particularly in poorly compacted or loosely arranged coarse samples, can influence settlement values due to the availability of larger void spaces for compression leading to larger settlements, whereas denser compacted samples with minimal void spaces may exhibit lower settlement values. Overall, the state of disturbances in oedometer testing, particularly in coarse materials with less cohesion, can impact the sample's initial void ratio and subsequent compressibility behavior.

The current study is limited in its ability to provide a detailed comparison between the samples due to insufficient data, which hinders the establishment of reliable correlations and interpretations. Further studies incorporating more comprehensive and repeatable tests are necessary to address this limitation. key reasons for the inability to compare the samples are as below:

- The absence of intact samples suitable for the proposed 15cm diameter apparatus (CROedo).
- Load increments applied during the tests on bag samples and cylinder samples were not identical.
- Cylinder samples could be obtained as less remolded samples, albeit not considered as intact samples.
- The remolded bag samples may not precisely represent the specific depth due to significant remolded state.
- The peat content and water content differed between all of the samples.

6.5.3 Effect of loading rate

This study utilized incremental loading in oedometer testing. However, to reach each load increment, a constant rate of strain was employed, followed by subsequently holding the load. This approach offers certain advantages over fast jumps between load increments in oedometer tests. This approach allows for a more gradual application of stress, allowing the sample to adjust and attain a more stable state before moving to the next load increment. By doing so, it helps to minimize the impact of sudden changes in stress on the sample, potentially providing more reliable test results.

6.5.4 Effects of modifications on the standard oedometer

To assess the efficiency and performance of the proposed oedometer, it is essential to compare the laboratory investigation results with monitored data collected in the field.

A relevant comparison between the two sets of data was not feasible in the current study due to several factors. Firstly, the location of the Klettelva project where Sweco conducted settlement monitoring differed from the sampling site for this study. Moreover, the observed settlements in the field were focused on layers with clay, which further complicates the comparison and limits its relevance. Nevertheless, as mentioned in section 6.2, a reasonable comparison could be made between the tested pure sand with the results from other studies. Based on this, the performance of the CROedo is expected to be acceptable; however, conducting additional tests on a broader range of samples using both the floating ring oedometer and the proposed apparatus would be a beneficial idea. Such a study would need to be pursued in further research to assess the performance of standard oedometer modification.

6.6 Other considerations

The presence of silt or organic content in soil can pose challenges in road construction due to the potential for frost heave during freezing and thawing cycles. While a 4% peat content may not be deemed problematic, it is important to note that this study did not consider the freezing and thawing effects in such soil types. Frost heave can cause significant damage to roads, resulting in uneven surfaces and reduced load-bearing capacity. Therefore, it is advisable to conduct further investigations specifically focusing on freezing effects in soils with silt or organic materials to better understand their behavior and associated risks, enabling more informed decision-making in road construction projects.

Chapter 7 Conclusion

Soils containing organic matter have been recognized as problematic in geotechnical projects due to their high compressibility, which can result in substantial settlement. Because of this, when organic materials are discovered in shallow soil surfaces, the conventional method is to excavate them. Although it may seem that geotechnical engineers should avoid using materials with a high organic content, more research, and evaluations in situations where the organic content is less than 10% and at greater depths may help to develop more ecologically friendly and affordable alternatives. As preloading can be used as one of the methods to release the expected settlement in early phases of such geotechnical projects, long term creep is still a question mark. This study tried to assess the degree of compressibility of soils with low organic content specifically in cohesionless soils within an experimental study with oedometer testing on both built samples and project-based samples.

In order to investigate such soils in a laboratory setting, it is necessary to possess intact representative samples and suitable equipment designed for cohesionless materials to accurately test geotechnical parameters. However, the majority of studies pertaining to cohesionless soils primarily utilize remolded samples due to the significant expenses associated with obtaining intact samples, such as freezing techniques that entail various considerations and costs. Additionally, when examining the settlement properties of these soils, conventional fixed ring oedometers encounter difficulties due to the inherent characteristics of cohesionless materials. In light of this challenge, the present study endeavored to address the issue by proposing an oedometer equipped with a compressible set of rings.

It is noteworthy that, despite the utilization of remolded samples in this study, the proposed approach represents an advancement in addressing the constraints inherent in conventional testing techniques applied to cohesionless soils with organic content. In other research studies, when even intact sampling methods are employed, it is crucial to consider the potential impact of the sampler's type on settlement outcomes during laboratory testing. Previous work by Long has highlighted that the use of conventional samplers designed for mineral soils may lead to the densification of peat, resulting in an underestimation of the actual settlement. Consequently, employing remolded samples could potentially exacerbate this issue, leading to further inaccuracies in settlement estimations (Long & Boylan, 2013b).

Evaluations were made to characterize the cohesionless soil samples with low organic content, with both conventional odeometer (Fixed ring oedometer) and newly proposed oedometer (Compressible ring oedometer) to assess the stiffness and creep number in samples. The present study encompassed three separate groups of materials: built samples and Klettelva samples in the form of bag and cylinder. The former category, namely the built samples, was primarily employed to investigate general trends and comprehensively understand the behavioral characteristics of cohesionless soils mixed with peat. Conversely, the Klettelva samples focused on project-specific inquiries, rendering it more dependent on contextual considerations. Results and findings could be summarized as follows:

7.1 Summary of results

General comments

- The organic content within the built samples comprised readily available peat from stores, while in the Klettelva samples, it consisted of naturally occurring organic matter found on-site. The author did not conduct a comprehensive assessment of the physical and chemical properties specific to these organic contents. Instead, the study relied on the existing information available regarding their characteristics. Consequently, the investigation did not encompass an in-depth analysis of the impact of different types of organic content that may have on the geotechnical properties studied. Instead, a generalized approach was taken, referring to all organic content as "organic content" or "peat" throughout the study.
- A comparison between the stiffness or other characteristics of the built samples and Klettelva samples may not be considered appropriate due to several factors that introduce significant variations. These factors include differences in grain size distribution, type of organic content, and water content among the samples. Hence, attempting to draw direct comparisons between the built samples and Klettelva samples would be deemed invalid. Therefore, the interpretation of the built samples and Klettelva samples was done separately.
- Peat is known for its low density and high permeability, as a result of the high void ratio. The primary result showed that when exposed to loading, soils containing peat compresses faster than other soils which is consistently aligned with previous researches and expectations (Carlsten, 1988). Peats exhibit a complex and heterogeneous nature, with variations observed in samples from different regions and countries due to factors

such as depositional history. It is imperative to recognize the necessity of conducting geotechnical investigations tailored to specific projects and site conditions in order to obtain accurate and reliable results.

- Settlement for a road filling or embankments on peat depends strongly on the several factors, which two of the most significant contributions are the amount of squeezed water and a secondary information as creep of the peat masses. Both factors are of decisive importance, for the time duration and the amount of final settlement. The relationship between these two contributions depends on wide range of factors including the characteristics of the peat, the thickness of the peat layer and load as mentioned in V220 (vegvesen Vegdirektoratet, 2022). Therefore, one needs to consider that in practice with caution, and considering all affecting parameters is required.

Deformation trends

- Main understing from the results has shown that the addition of even a small percentage, such as 2%, of peat to sand can induce noticeable changes in soil behavior and stiffness compared to pure sand. Remarkably, even a low organic content can significantly enhance the soil's capacity to retain and bind water, consequently resulting in substantial alterations in soil properties and a reduction in stiffness.

Stiffness trends

- In the investigation of sample stiffness, the study's findings revealed that the inclusion of peat in the samples had a notable impact. Specifically, as the peat content increased up to 10%, the stiffness of the samples decreased. This is evident in the results of the modulus number, which decreased from approximately 140 in the case of pure sand to a range of 40 for samples containing 10% peat. Similarly, when analyzing the Klettelva samples, it was observed that the modulus number ranged between 30 and 40 for samples with 2% peat content. However, as the organic content increased to approximately 9%, the stiffness of the samples was reduced by half, as indicated by a decrease in the modulus number. These findings illustrate the significant influence of peat content on the stiffness properties of the soil samples, emphasizing the relationship between organic content and the resulting stiffness characteristics.
- Upon examining the results of the modulus number in the context of reloading, it can be asserted that preloading has demonstrated a positive influence on enhancing the stiffness of the samples. Specifically, the modulus number exhibited an acceptable

degree of increase, reaching approximately 100 or even higher in the case of Klettelva samples.

- Preloading appears to have a more pronounced effect in samples with higher organic content compared to samples with lower organic content. This assertion is strongly supported by the results obtained from the Klettelva cylinder samples.
- The majority of the obtained results consistently confirm the primary trend of decreasing stiffness as water content and peat percentage increase.
- The results obtained from the compressible ring oedometer experiments revealed that the samples exhibited higher strains compared to the relatively identical samples tested using the fixed ring oedometer. This observation suggests that the proposed oedometer apparatus has potentially addressed certain limitations associated with the fixed ring oedometer when dealing with sandy materials. Additionally, the higher values of the creep number obtained from the fixed ring oedometer experiments indicate the presence of friction between the soil grains and the ring, potentially resulting in misleading deformation measurements.

Creep trends

- In order to evaluate the long-term deformation and creep behavior, it is important to note that longer durations of loading would typically yield more reliable results. However, even within the relatively short time span of the present study, the calculated creep numbers exhibit relatively high values indicating promising findings and a good propensity for creep deformation.
- Preloading has exhibited a modest impact on creep behavior, as evidenced by higher creep numbers observed over extended periods. However, it is important to note that the effectiveness of preloading appears to be relatively limited. Analysis of Klettelva cylinder samples reveals that preloading has resulted in a shift in creep numbers from the range of 300-500 to 300-700 for samples containing 4-12% peat after preloading. It should be emphasized that these numbers provide only approximate estimations of the results.
- Caution should be exercised when interpreting these findings, as direct comparison of results between preloading and creep behavior may not yield significant improvements, particularly in the case of the Klettelva samples. Furthermore, it is necessary to consider various factors that may contribute to potential overestimation or underestimation of results. Moreover, the proposed experimental apparatus utilized in this study represents

a novel implementation, and further refinements are warranted to enhance the reliability and accuracy of the obtained outcome.

- To obtain a more comprehensive understanding, it is recommended to conduct direct measurements of embankments in field conditions and subsequently compare them with the study's findings. Such field measurements would provide valuable insights for refining the research methodology and achieving greater precision in the investigation of creep behavior.
- The interpretation of time resistance and swelling behavior in oedometer testing during unloading is a complex task due to several influential factors. These factors include the rearrangement and realignment of particles, the breakage of particle bonds, and the reduction in interparticle contacts, all of which can significantly affect the test results. Stress-induced anisotropy, particle rearrangement, and alterations in pore water pressure distribution further contribute to variations in the measured deformation values during the unloading process. To facilitate a better understanding of swelling behavior and to obtain more reliable data, it may be advisable to increase the duration of the unloading increment. This extended duration allows for a more comprehensive assessment of the degree of swelling and assists in obtaining more accurate and meaningful results. It is important to emphasize that significant swelling may occur under conditions of substantial unloading, underscoring the necessity to consider this phenomenon when unloading in the field.

Final Comments

Based on the modulus numbers and creep numbers obtained in this study, it is suggested that geotechnical investigations should be conducted in-depth for construction projects involving organic sand layers containing more than 4% peat. However, it is important to note that this threshold value is not universally applicable and should be considered as a recommendation specific to the studied site and its characteristics. Different construction projects may exhibit varying soil compositions and geotechnical parameters, necessitating site-specific evaluations. In the case of Klettelva project, it can be anticipated that the long-term deformation would result in approximately 1 to 2% strain, which is deemed acceptable for a road application. Nonetheless, it is crucial to acknowledge that these findings are project-specific and should not be generalized without considering the specific context and conditions of other projects.

7.2 Further work

In order to enhance the proposed compressible ring oedometer, measuring both vertical and lateral stress and strain of the specimen could be a viable option. The idea of measuring lateral strain in addition to vertical strain in oedometer testing can provide valuable insights into the deformation behavior of the soil sample. Implementing this idea can be beneficial in certain scenarios where the lateral response of the soil is of interest, such as when studying the behavior of anisotropic or layered soils, investigating the effect of confining pressure on soil deformation, or analyzing the behavior of soils with potential lateral stress-induced effects. This could be achieved by incorporating an elastic lateral supporting ring, using strain gauges or extensometers that are specifically designed to measure lateral deformation or other methods to monitor and record the lateral strains during the test. A study by Kolymbas and Bauer on a new testing device called the soft oedometer has taken into account the lateral stress and strain, can be inspiring for further studies (Kolymbas & Bauer, 1993).

The proposed oedometer in this study demands skillful preparation of the specimen and can be time-consuming. In the current study, the preparation of the sample was carried out while the ring was positioned in the basin, and the sample was compacted inside the ring. However, one can come up with other alternatives of preparing the sample that could potentially simplify the calculation of soil parameters before and after the test by enabling the monitoring of porosity and void ratio, especially when samples are remolded. This might help to build a sample that closely mimics its properties in situ.

For future investigations, it is advisable to incorporate field data from an existing project, as this would facilitate a comprehensive comparison between laboratory findings and field observations. Such a comparative analysis serves to validate the reliability of the proposed laboratory apparatus and offers valuable guidance for refining and enhancing the development procedures. Future studies could consider conducting further investigations using a variety of specimen mixtures and undisturbed samples, provided that obtaining undisturbed samples becomes feasible.

Appendix A -Klettelva specifications

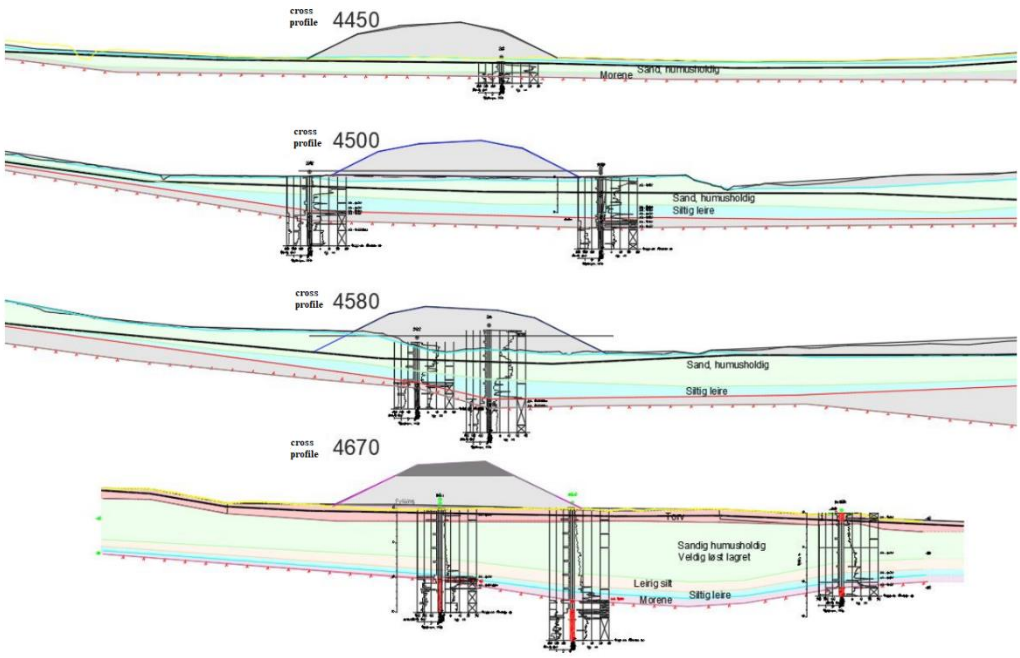


Figure 0-1 Interpreted stratification of the west side of Klettelva

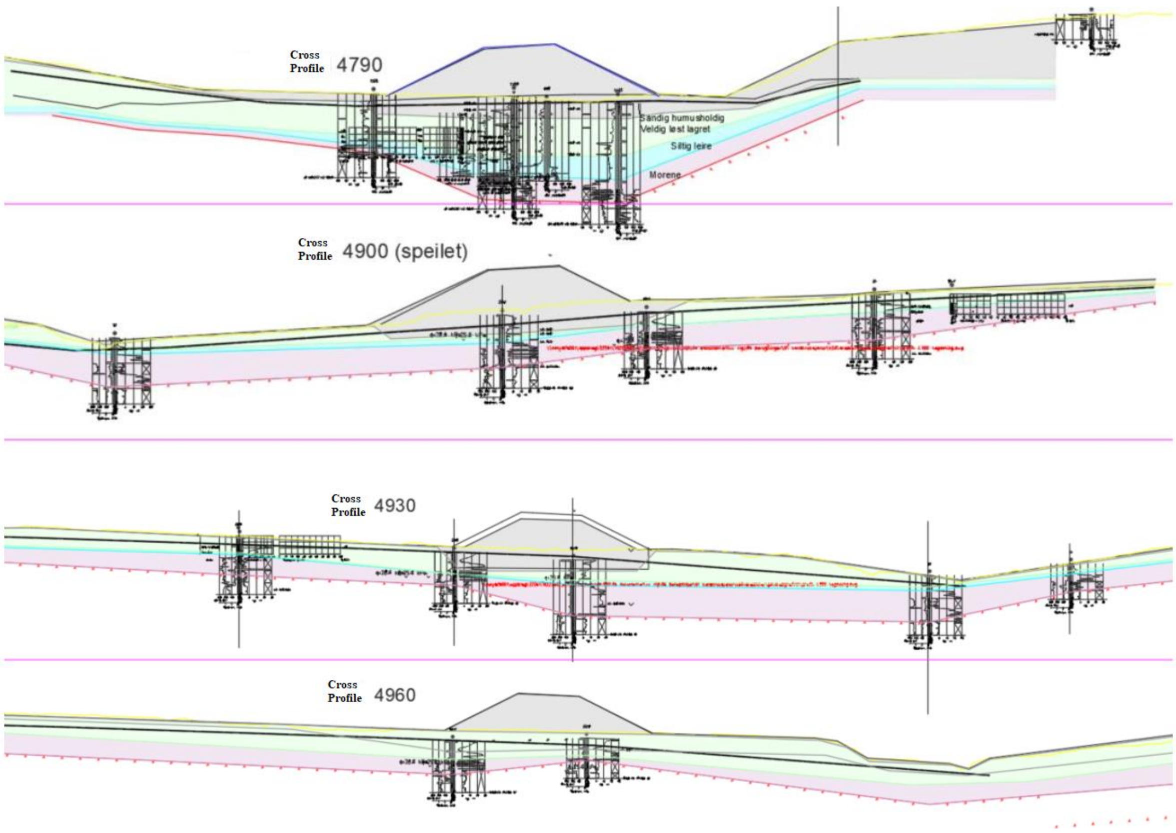


Figure 0-2 Interpreted stratification of the east side of Klettelva

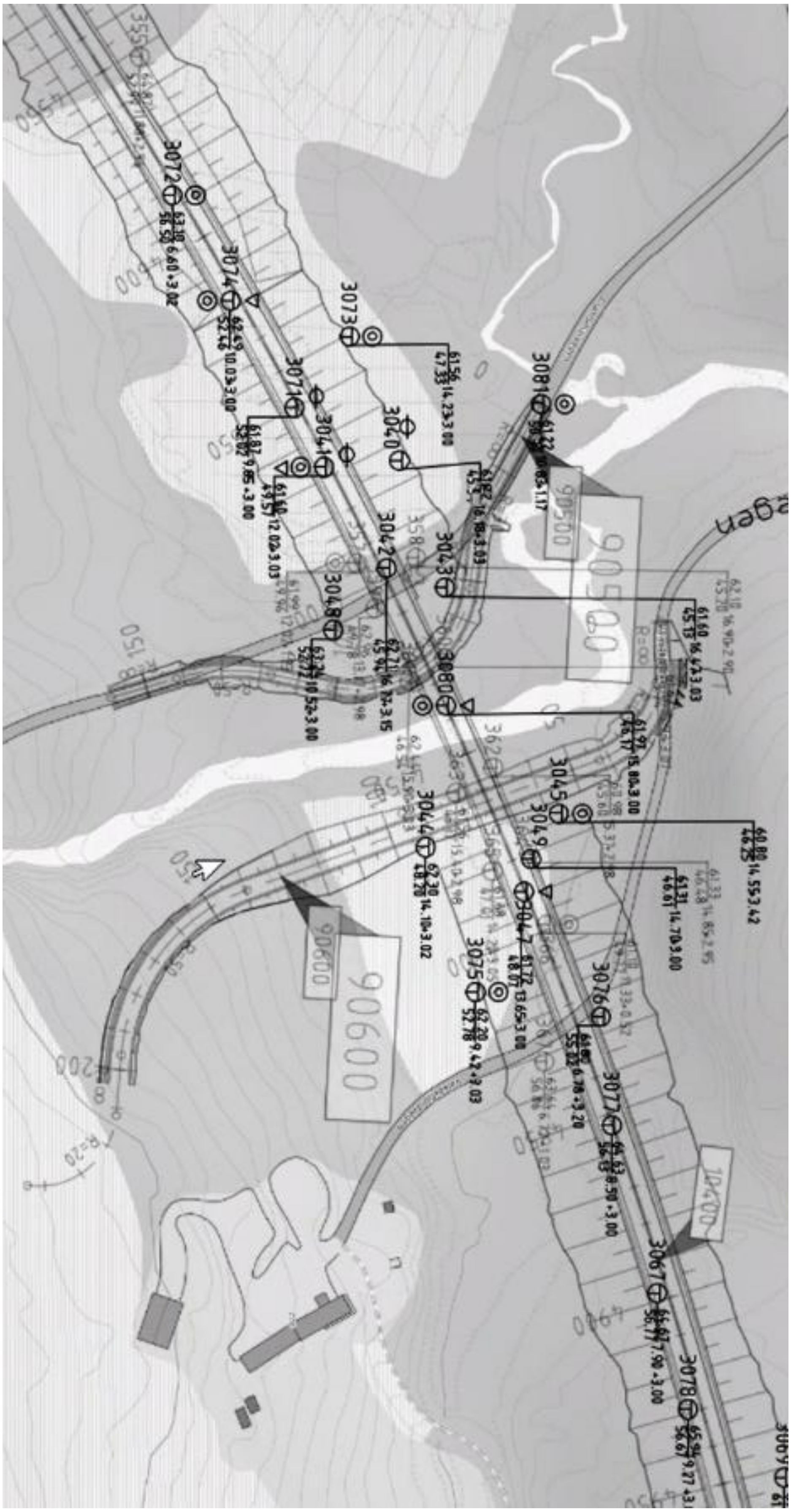


Figure 0-3 Borehole plan for Klettelva

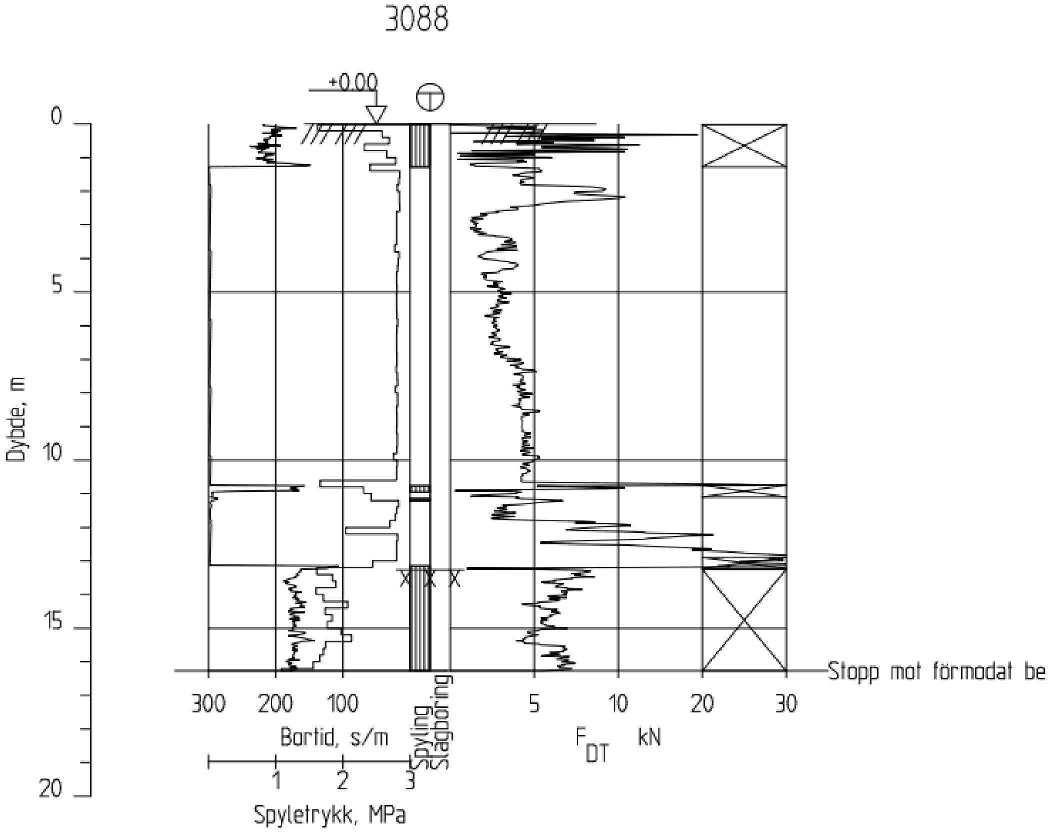


Figure 0-4 Total sounding borehole 3088

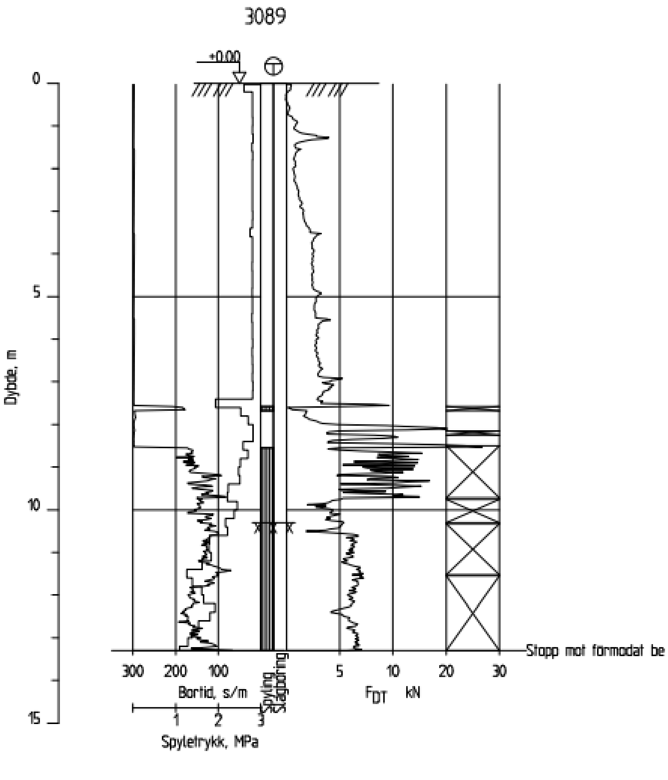


Figure 0-5 Total sounding in borehole 3089

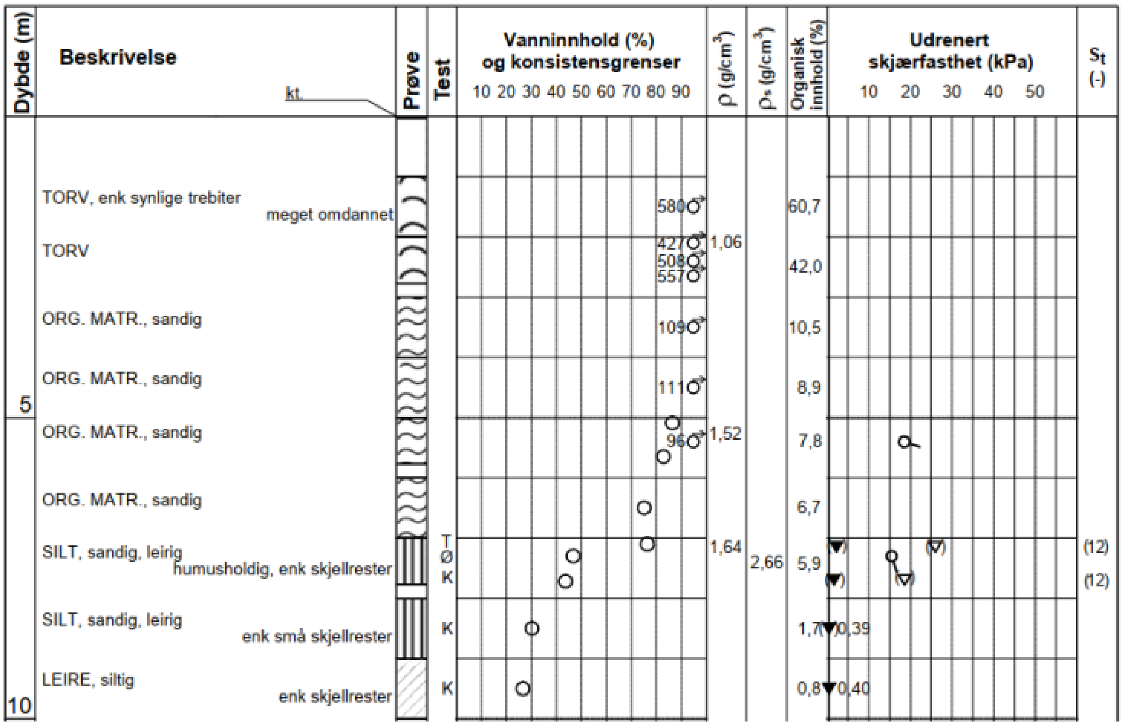


Figure 0-6 Laboratory results for borehole 3041 as the representative borehole with worst case scenario of settlement

Tverrprofil	Gjeldene strekning	Beregnede faser
4500	Berg/løsmasseskjæring - 4500	**Utskifting topplag organisk, ca. 1,0 m, forbelastning** Fase 1: utskifting topplag/torv Fase 2: 3m fylling (2m fra topp) Fase 3: Full fyllingshøyde
4580	4500-4580	**Fasestyrt, masseutskifting, forbelastning** 1,5m overhøyde og 1,5 m høy motfylling av komprimerte stedlige masser. Fase 1: Masseutskifting 2,0 m. Fase 2: 3 m fylling (3,5 m fra topp) + motfylling Fase 3: 5,5 m fylling (1.0 m fra topp) Fase 4: Fylling 1.5 m overhøyde Fase 5: Avtaking overhøyde, bygging vegoverbygning.
4670	4580-4690 (20 m fra landkar vest)	**Fasestyrt, masseutskifting, 1.5 m lette masser, geonett, forbelastning** 1,5 m høyde glasopor, geonett, motfylling 2.5 m komprimerte stedlige masser. Fase 1: 3.5 m masseutskifting Fase 2: 3 m fylling (3,5m fra topp), geonett 120 kN/m + motfylling. Fase 3. 5.5 m fylling (1,0 m fra topp)
		Fase 4: Full fyllingshøyde høyde. Fase 5: Erstatt 1.5 m sprengstein med glasopor. Oppbygning vegoverbygning.
4690 – Landkar vest	Overgang 20m inn mot landkar vest	**Masseutskifting, forbelastning, utkiling med lette masser, geonett. Detaljeres senere.** Kan fylles med sprengstein inntil prosjektert veghøyde. Sprengstein må graves ut senere ved etablering av landkar, og utkiling med lette masser. Fase 1: Masseutskifting 3.5m Fase 2: Fylling 3.5 m fra topp + geonett 120 kN/m. Fase 3: Fylling 1,0 m fra topp. Fase 4: Fylling opptil prosjektert veghøyde. Fase 5: Utgraving landkar og oppbygning utkiling inn mot landkar. Erstatt 1.5-2,5 m sprengstein med glasopor. Oppbygning vegoverbygning.
Landkar øst – 4810	Overgang fra landkar øst og 20-30 m østover	**Masseutskifting, forbelastning, utkiling med lette masser, geonett og motfylling.** Kan fylles med sprengstein inntil prosjektert veghøyde. Sprengstein må graves ut senere ved etablering av landkar, og utkiling med lette masser.
		Fase 1: Masseutskifting 3.5m Fase 2: Fylling 4.5 m fra topp + geonett 120 kN/m, motfylling sørside. Fase 3: Fylling 1.5m fra topp Fase 4: Full fyllingshøyde Fase 5: Utgraving landkar og oppbygning utkiling inn mot landkar. Erstatt 2,0-1,0 m sprengstein med glasopor. Oppbygning vegoverbygning.
4790	4810 - 4930	**Fasestyrt, delvis masseutskifting, geonett, forbelastning, myrrehabilitering fungerer som motfylling.** Fase 1: 3.5 m masseutskifting Fase 2: Fylling, inntil 4.5 m fra topp + motfylling og myrrehabilitering. Fase 3: Fylling, inntil 1.5 m fra topp. Fase 4: Full fyllingshøyde Fase 5: 1.5 m Overhøyde Fase 6: Fjerne overhøyde
4900	4930 - 4970	**Utskifting topplag organisk. 1.0 m overhøyde anbefales for å forsikre seg om at man har fått komprimert masser under fylling. Myrrehabilitering som motfylling der det er aktuelt.** Fase 1: Utskifting topplag organisk Fase 2: Fylling, inntil 4.5 m fra topp + myrrehabilitering. Fase 3: Fylling, inntil 1.5 m fra topp. Fase 4: Full fyllingshøyde Fase 5: 1.5 m Overhøyde
		Fase 6: Fjerne overhøyde
4970	4970 – 5200 (bergskjæring)	Utskifting topplag organisk til faste masser. Lagvis utlegging og komprimering. Ingen geotekniske tiltak.

Figure 0-7 Exact stages of preloading in different terrain profiles

Appendix B Peat specifications in Built samples

Addition per m^3

NPK 12-4-18 micro	1.5 kg
magnesium lime	4.5 kg
chicken manure	7 kg
ferrous sulfate	0.2 kg

Plant ingredients per mg/l

Nitrogen	350
Phosphor	60
Potassium	420
Calcium	350
Magnesium	220
Sulphur	240
Boor	0.5
	0.2
iron	75
manganese	26
molybdenum	0.2
zinc	2.6

Appendix C Grain Size distribution

Mass fractions

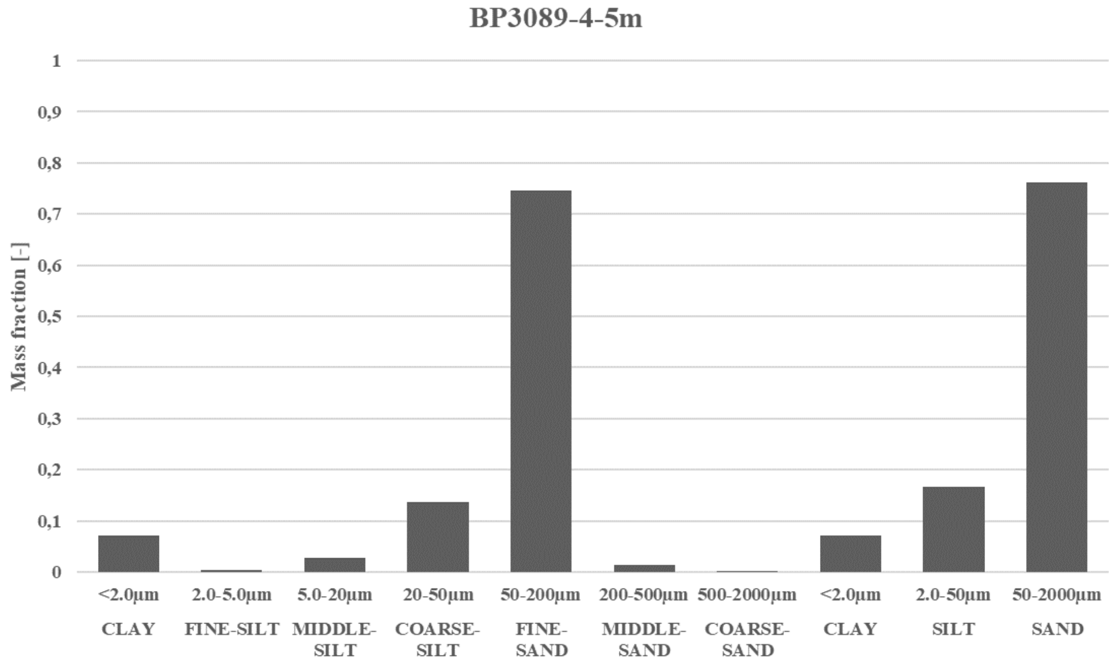


Figure 0-1 Bar chart of soil composition - BP3089-4-5m

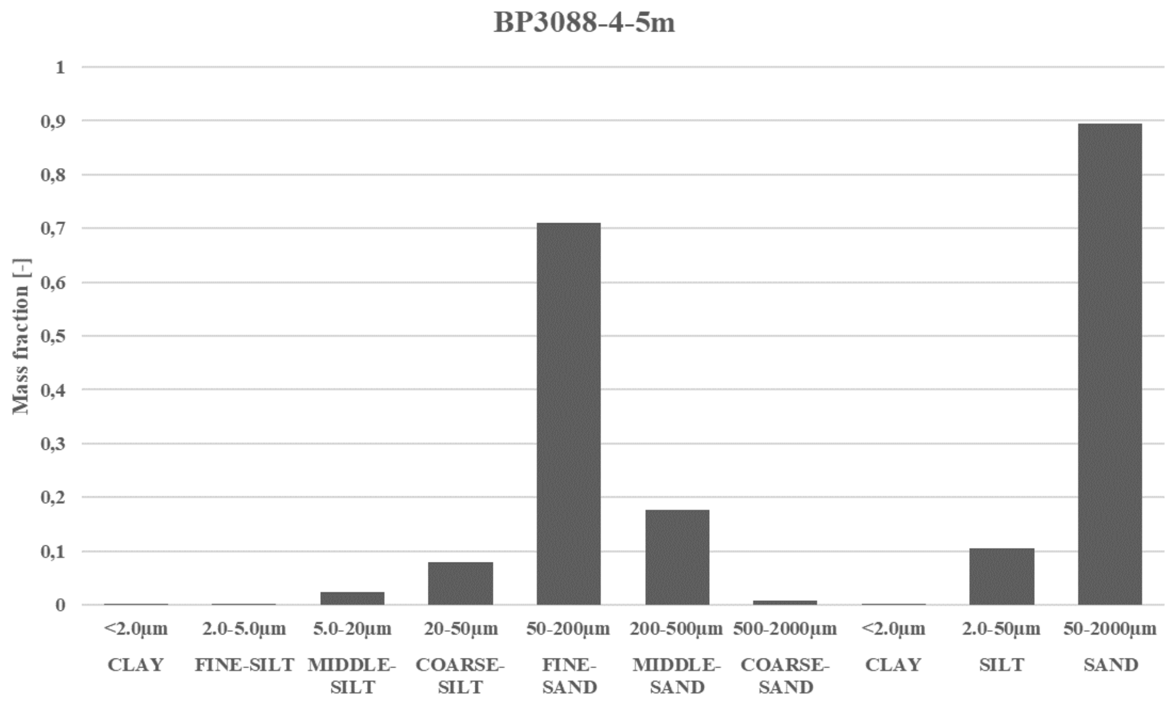


Figure 0-2 Bar chart of soil composition – BP3088-4-5m

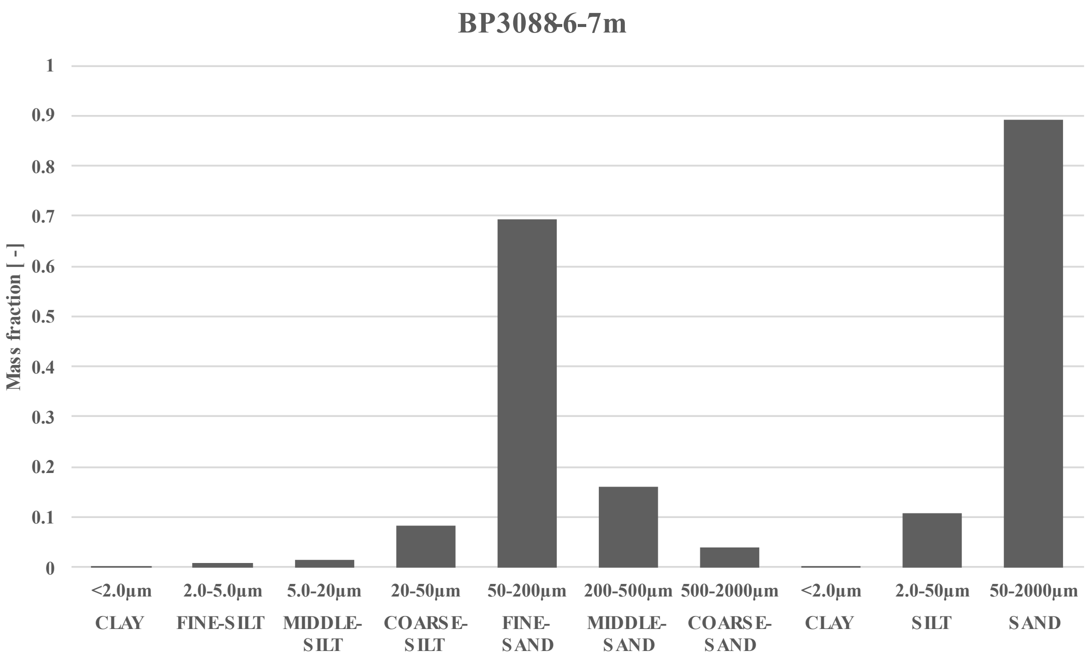


Figure 0-3 Bar chart of soil composition – BP3088-6-7m

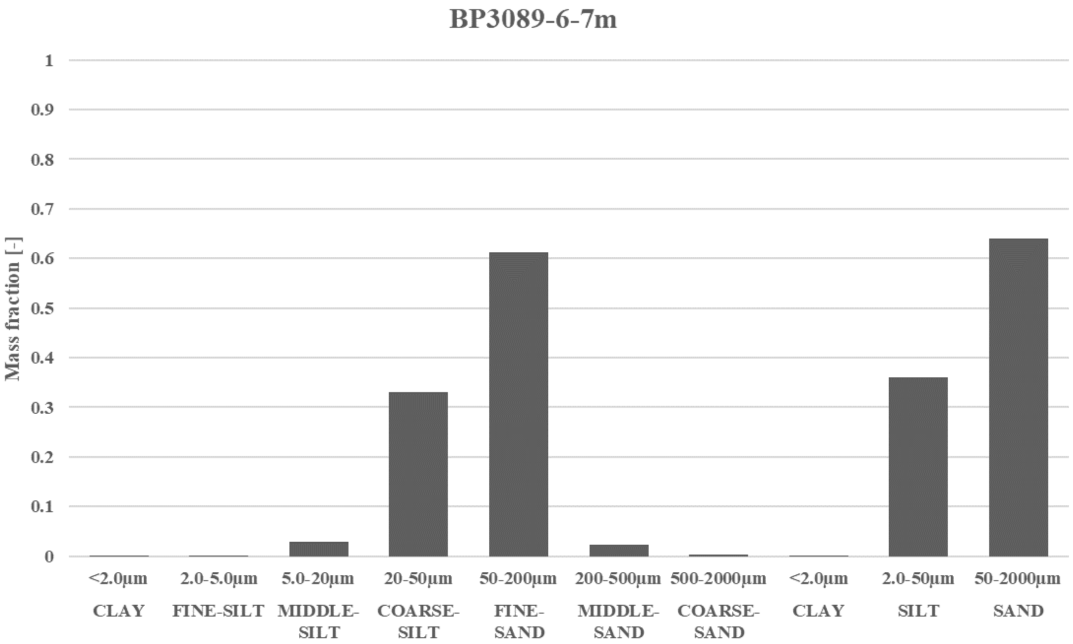


Figure 0-4 Bar chart of soil composition – BP3089-6-7m

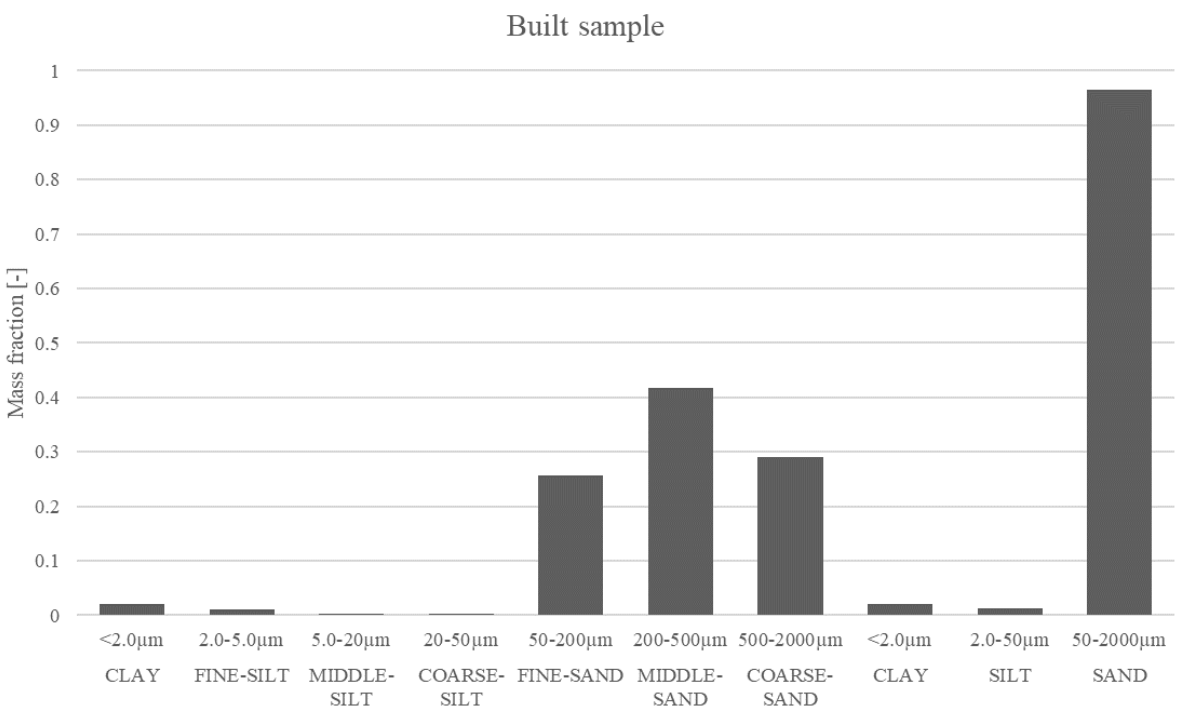


Figure 0-5 Bar chart of soil composition –Sand in Built sample

Soil triangles

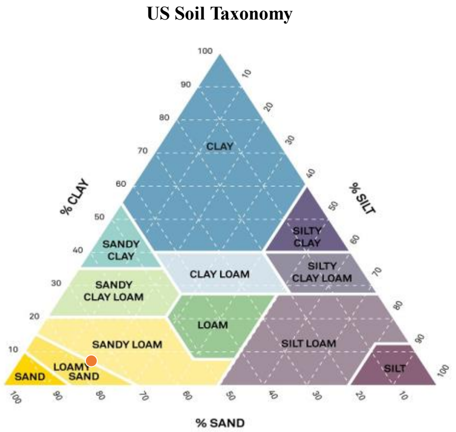


Figure 0-7 BP3089-4-5m

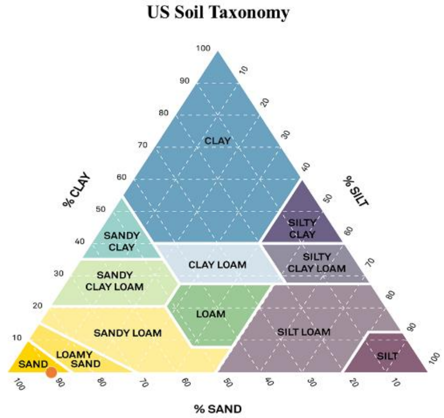


Figure 0-6-BP3088-4-5m

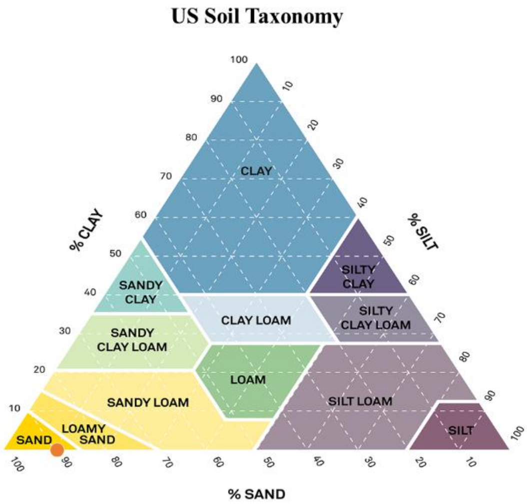


Figure 0-10 BP3088-6-7-m

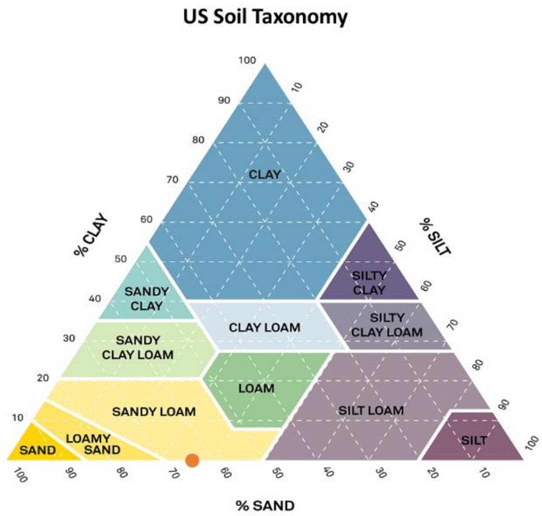


Figure 0-9 BP3089-6-7m

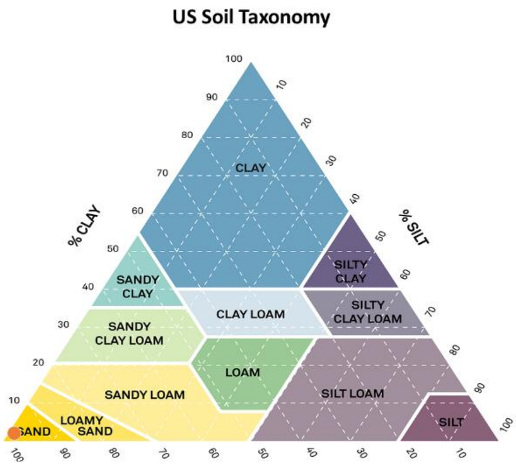
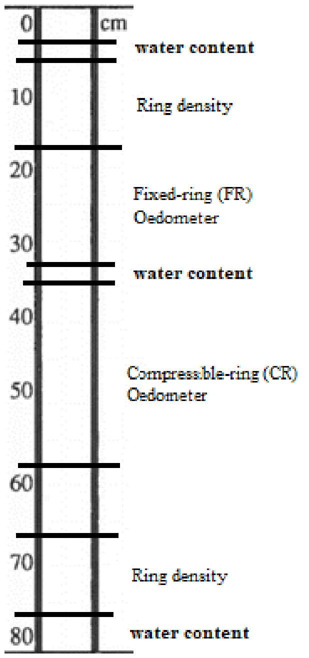


Figure 0-8 Sand in Built samples

Appendix D Cylinder samples specification

	GEOTECHNICAL LABORATORY NTNU	12BP3088-4-5m

INDEX TESTS		**Project: Klettaelva**	
Site:	Klettaelva	Operator: Maryam Fakhari	
Hole no:	BP3088	Sampling date:	1-Mar-23
Sample no:	1	Testing date:	25-Apr-23
Depth, z (m):	4 to 5 m	**Density small sample**	
Ground water level (m):	1		density (gr/cm3)
Length of sample (mm):	66.00	sample1	1.67
Diameter of the sample (cm):	5.40	sample2	1.61
Mass, cylinder w/sample: (gr)	4129.7		
Mass, empty culinder:	1650	Organic cor	
Volume of sample cm3:	1511.546	samples 1	12%
Mass, sample (gr):	2479.7	sample 2	10%
Average density (gr/cm3)	1.640506	sample 3	15%
Detailed location:		**General classification**	



Soil type: Mix of silt, sand and peat-Very Organic so

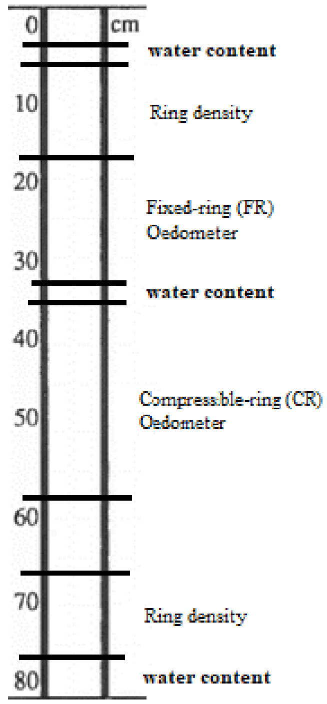
Description:


Remarks:

water content (%)	Sample1	Sample2	Sample3
Cup no:	1	2	3
Total wet mass of sample (gr)	75	320	160
Total dry mass of sample (gr)	40	175	81.5
Mass, water (gr)	35	145	78.5
water content (%)	88%	83%	96%

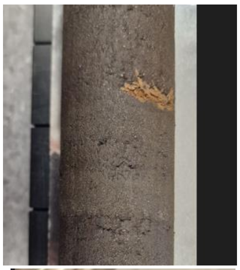
	GEOTECHNICAL LABORATORY NTNU	4BP3088-6-7m

INDEX TESTS		**Project: Klettaelva**
Site:	Klettaelva	Operator: Maryam Fakhari
Hole no:	BP3088	Sampling date: 1-Mar-23
Sample no:	2	Testing date: 27-Apr-23
Depth, z (m):	6 to 7 m	**Density small sample**
Ground water level (m):	1	density (gr/cm3)
Length of sample (mm):	74.50	sample1 1.81
Diameter of the sample (cm):	5.40	sample2 1.75
Mass, cylinder w/sample: (gr)	4787.00	
Mass, empty culinder:	1863.70	Organic cor
Volume of sample cm3:	1706.21	samples 1 4%
Mass, sample (gr):	2923.30	sample 2 3%
Average density (gr/cm3)	1.71	sample 3 4%
Detailed location:		**General classification**



Soil type: Mix of silt, sand and peat

Description:

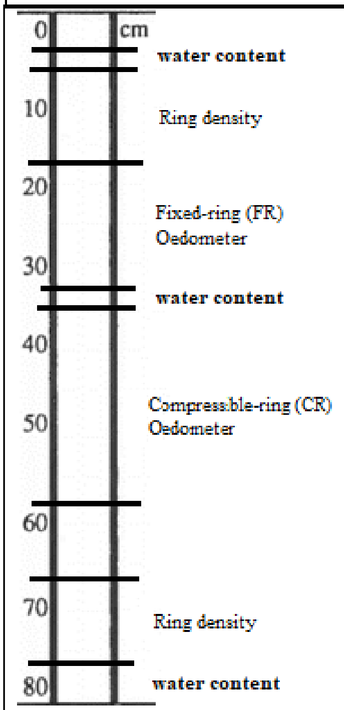


Remarks:

water content (%)	Sample1	Sample2	Sample3
Cup no:	1	2	3
Total wet mass of sample (gr)	70	320	150
Total dry mass of sample (gr)	39.5	175	81.5
Mass, water (gr)	30.5	145	68.5
water content (%)	77%	83%	84%

	GEOTECHNICAL LABORATORY NTNU	7BP3089-4-5m

INDEX TESTS		**Project: Klettaelva**	
Site:	Klettaelva	Operator: Maryam Fakhari	
Hole no:	BP3089	Sampling date:	1-Mar-23
Sample no:	3	Testing date:	29-Apr-23
Depth, z (m):	4 to 5 m	**Density small sample**	
Ground water level (m):	1		density (gr/cm3)
Length of sample (mm):	74.5	sample1	1.81
Diameter of the sample (cm):	5.4	sample2	1.75
Mass, cylinder w/sample: (gr)	4460		
Mass, empty culinder:	1631	Organic cor	
Volume of sample cm3:	1706.215	samples 1	8%
Mass, sample (gr):	2829	sample 2	7%
Average density (gr/cm3)	1.658056	sample 3	7%
Detailed location:		**General classification**	



Soil type: Mix of silt, sand and peat

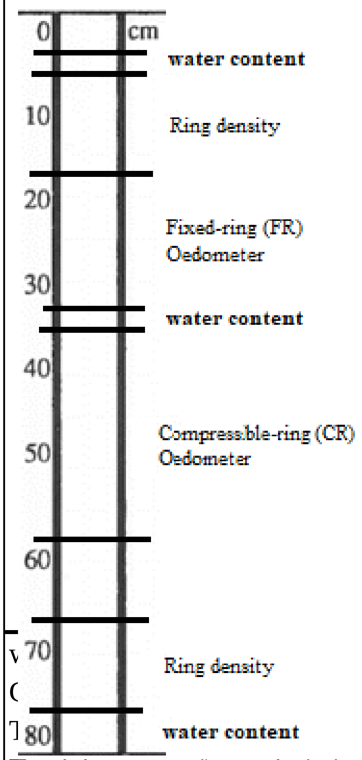
Description:

Remarks:

water content (%)	Sample1	Sample2	Sample3
Cup no:	1	2	3
Total wet mass of sample (gr)	50.2	25	121
Total dry mass of sample (gr)	33.41	15.32	81.5
Mass, water (gr)	16.79	9.68	39.5
water content (%)	50%	63%	48%

	GEOTECHNICAL LABORATORY NTNU	**1BP3089-6-7m**

INDEX TESTS		**Project: Klettaelva**	
Site:	Klettaelva	Operator: Maryam Fakhari	
Hole no:	BP3089	Sampling date:	1-Mar-23
Sample no:	4	Testing date:	29-Apr-23
Depth, z (m):	6 to 7 m	**Density small sample**	
Ground water level (m):	1		density (gr/cm3)
Length of sample (mm):	74.5	sample1	1.89
Diameter of the sample (cm):	5.4	sample2	1.91
Mass, cylinder w/sample: (gr)	5016		
Mass, empty culinder:	1710	Organic cor	
Volume of sample cm3:	1706.215	samples 1	1%
Mass, sample (gr):	3306	sample 2	2%
Average density (gr/cm3)	1.937623	sample 3	1%
Detailed location:		**General classification**	



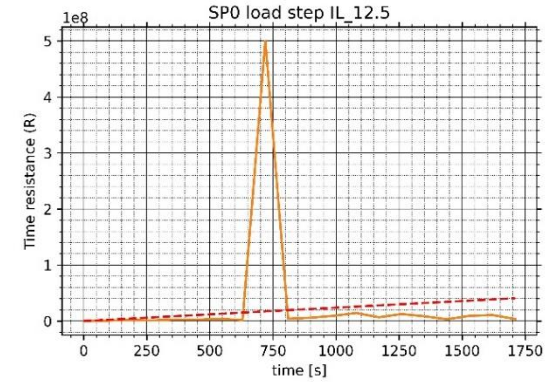
Soil type: Mix of silt, sand and peat

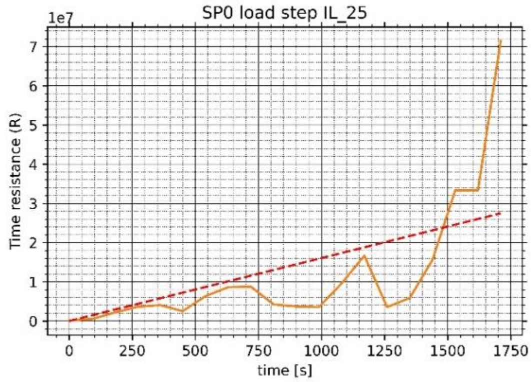
Description:

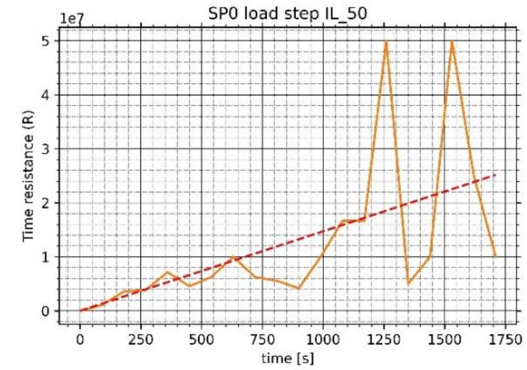
	Sample1	Sample2	Sample3
v	1	2	3
(	50.2	25	121
Total dry mass of sample (gr)	33.41	15.32	81.5
Mass, water (gr)	16.79	9.68	39.5
water content (%)	50%	63%	48%

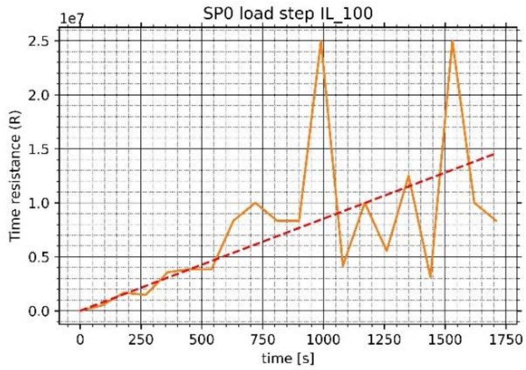
Appendix E Time resistance graphs

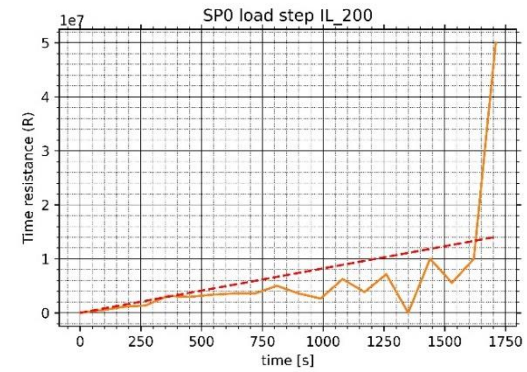
Built samples-SP0

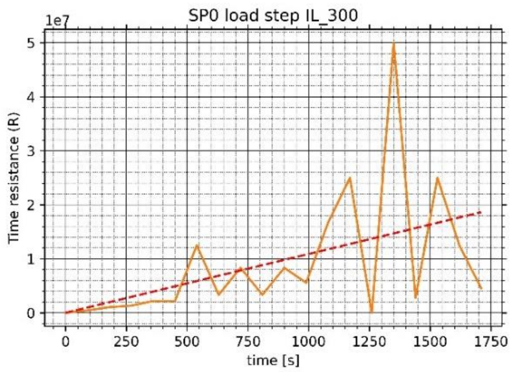


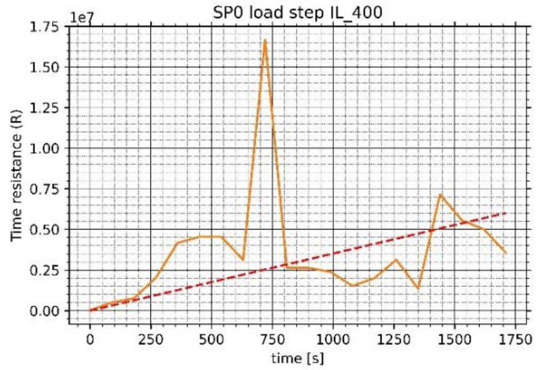


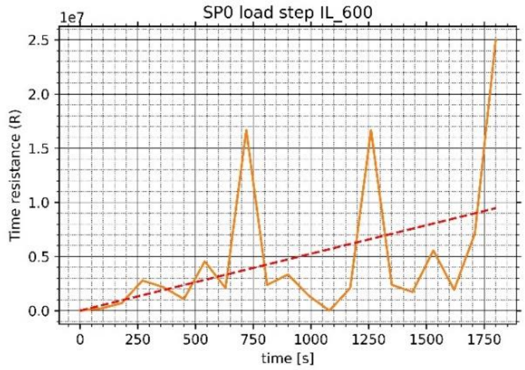




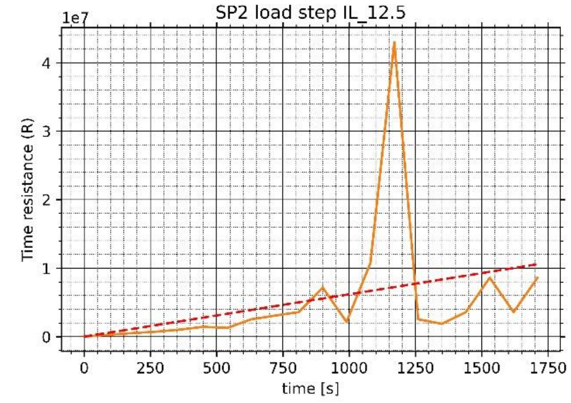


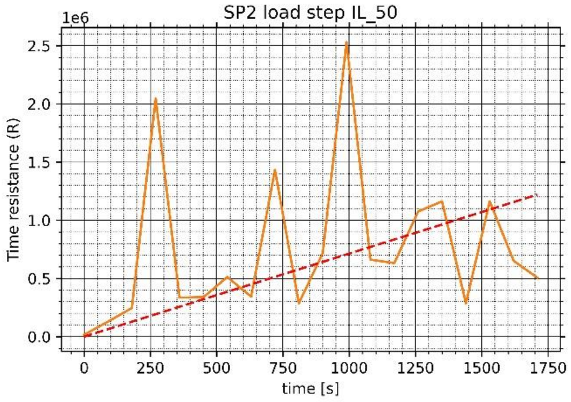


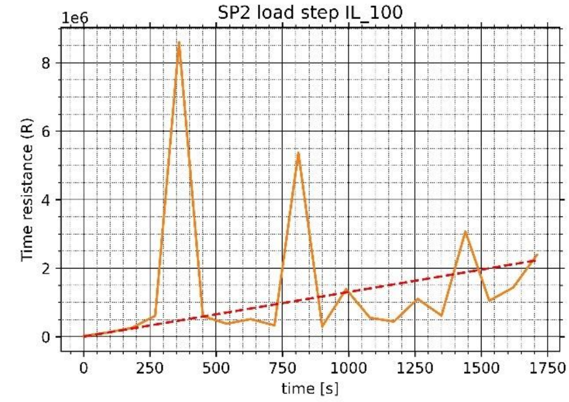


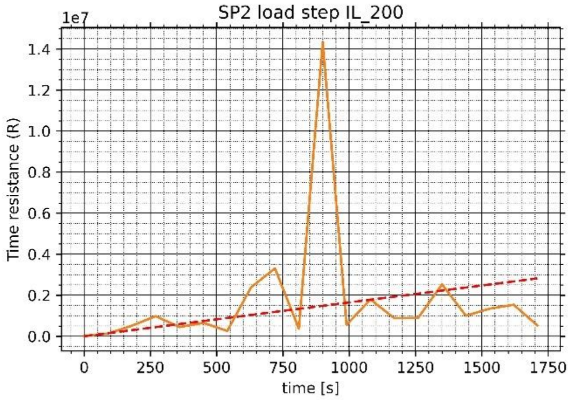


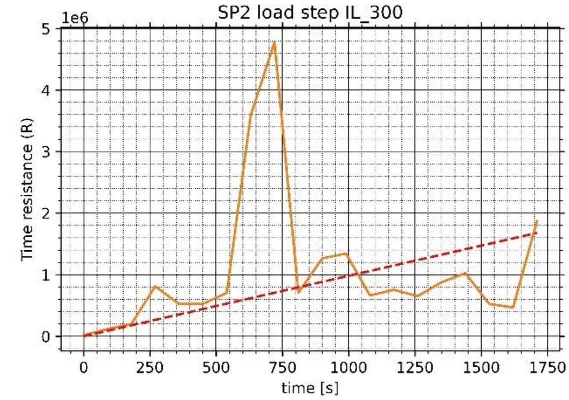
Built samples-SP2

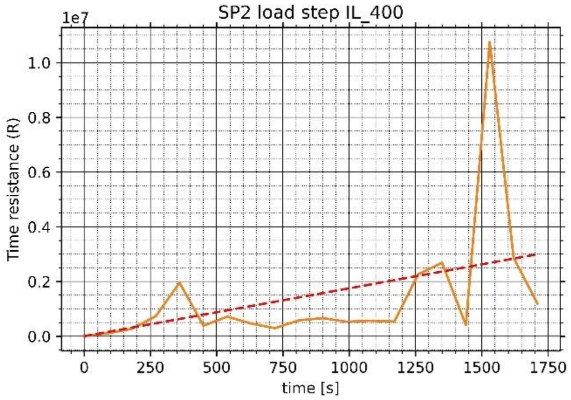


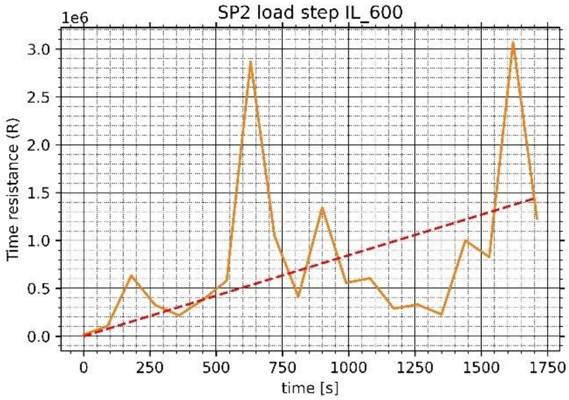




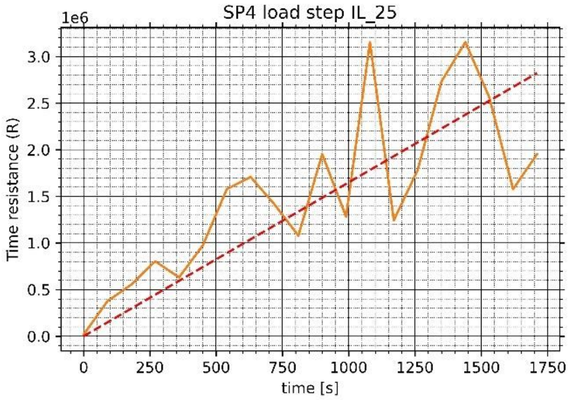


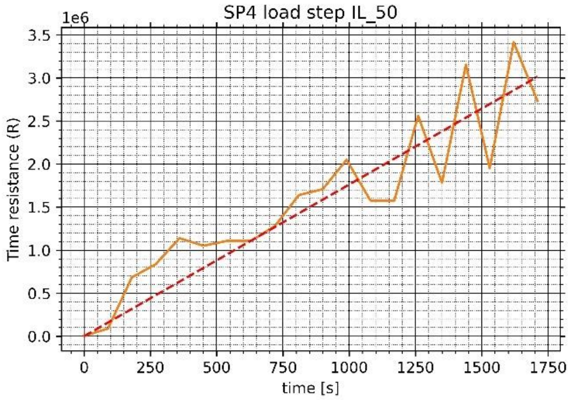


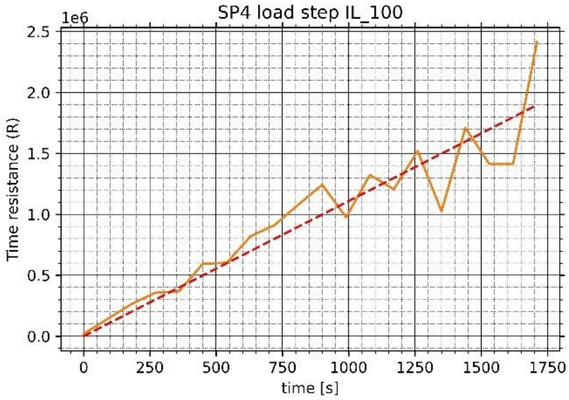


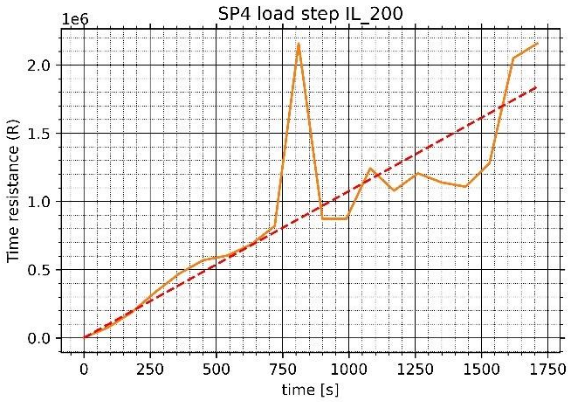


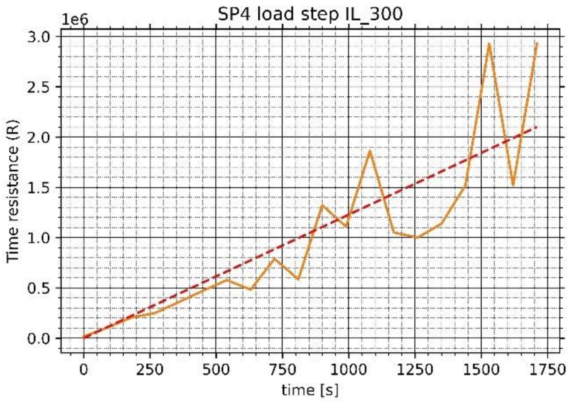
Built samples-SP4

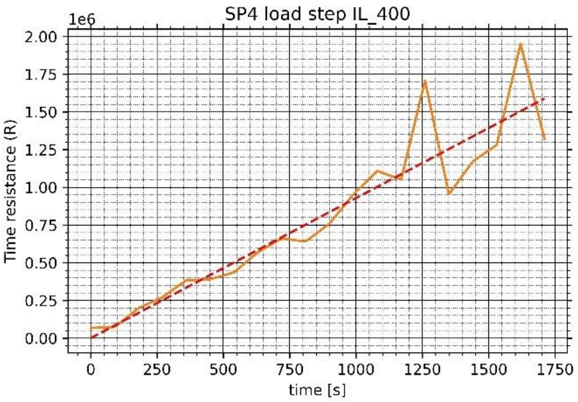


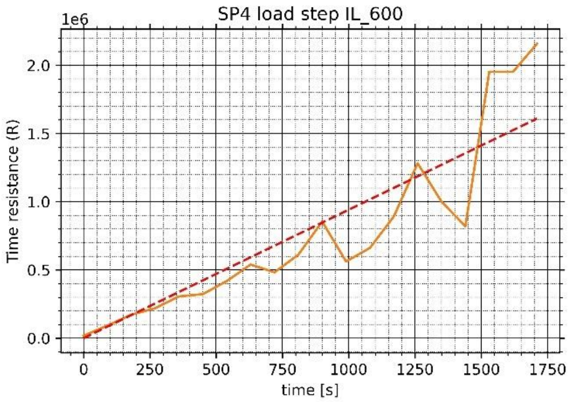




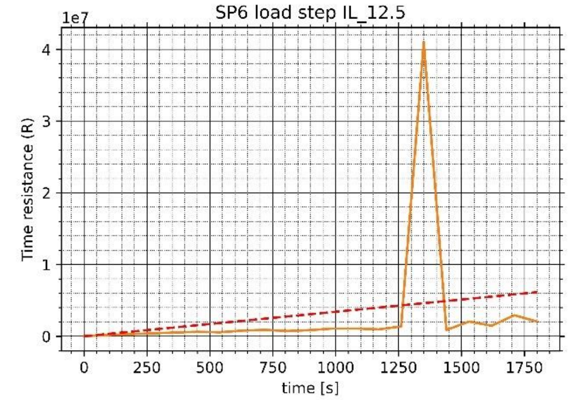


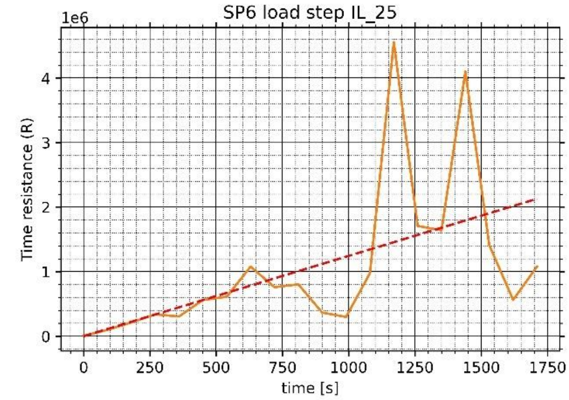


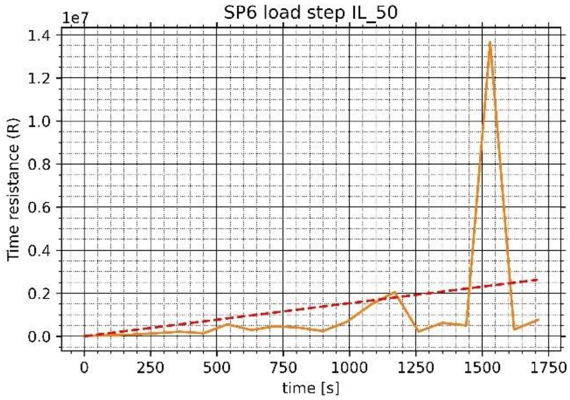


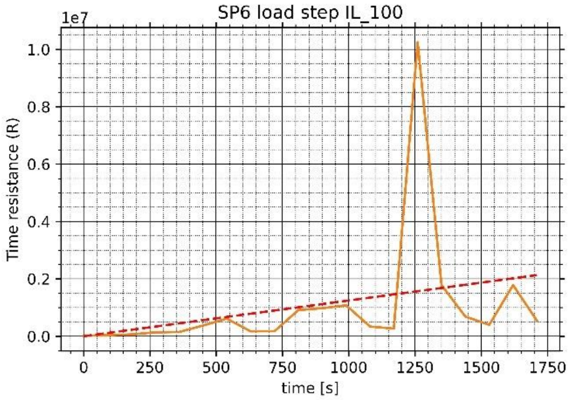


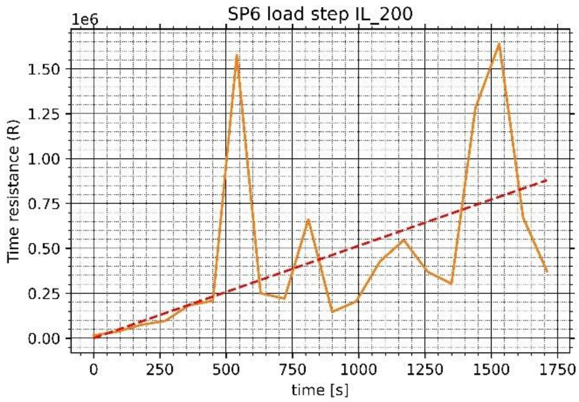
Built samples-SP6

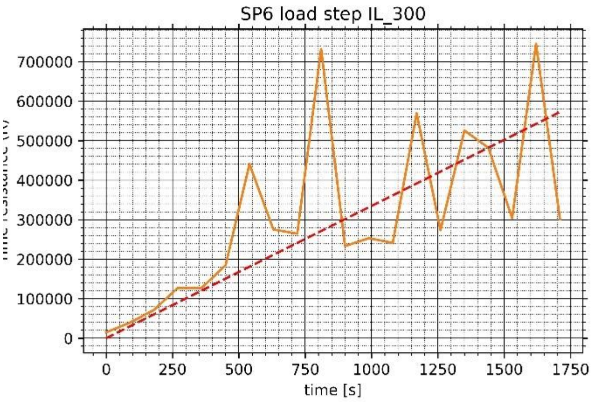


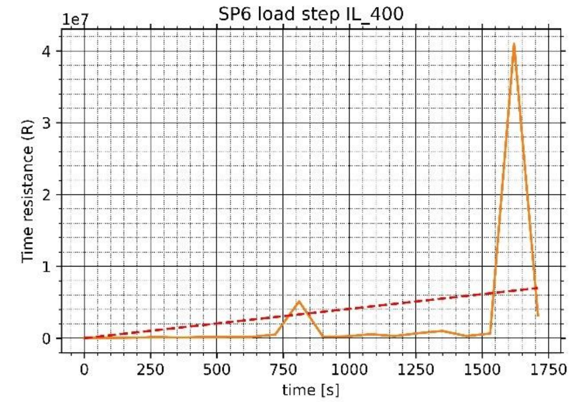


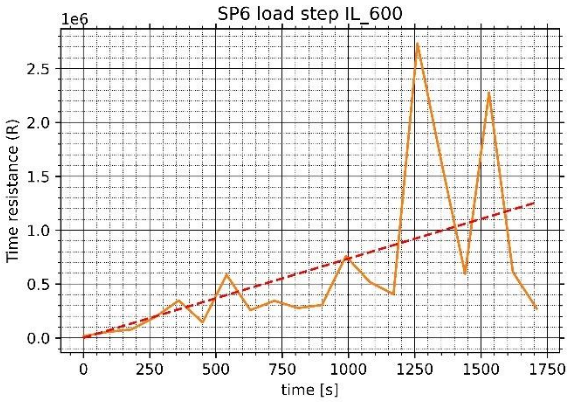




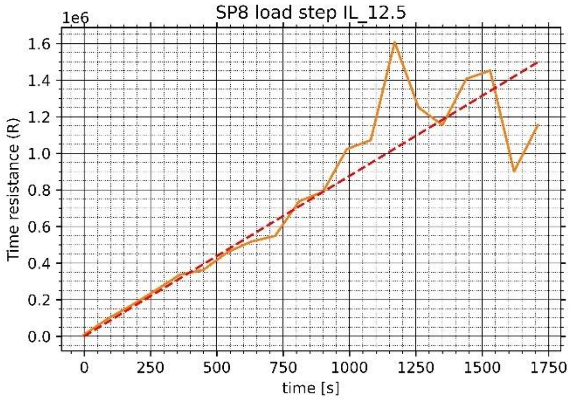


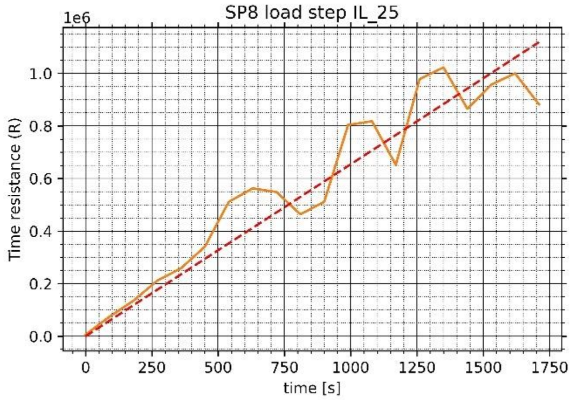


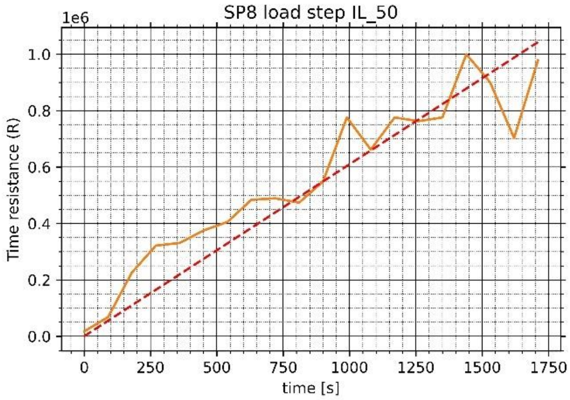


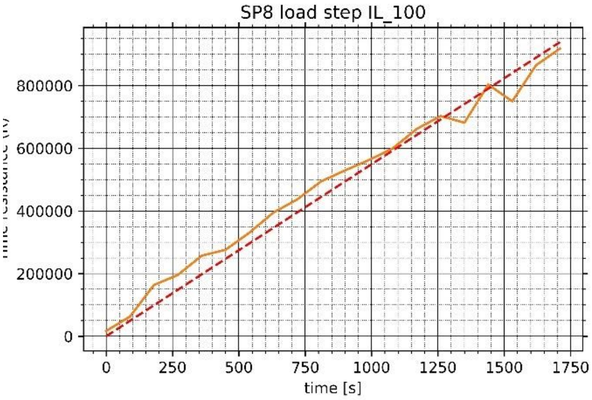


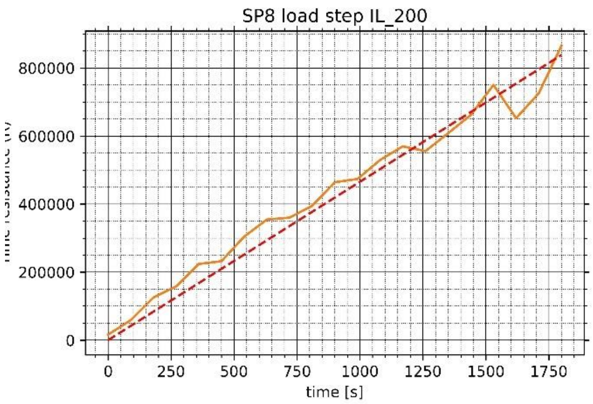
Built samples-SP8

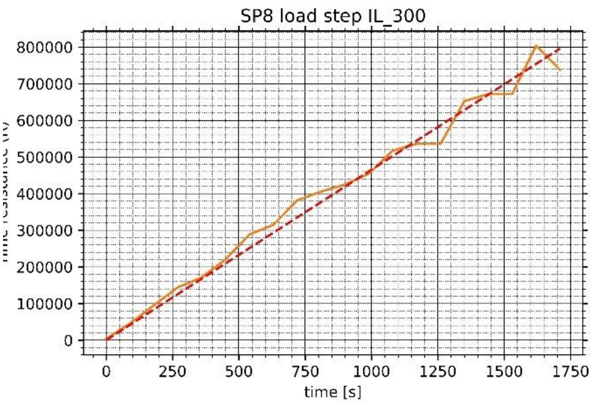


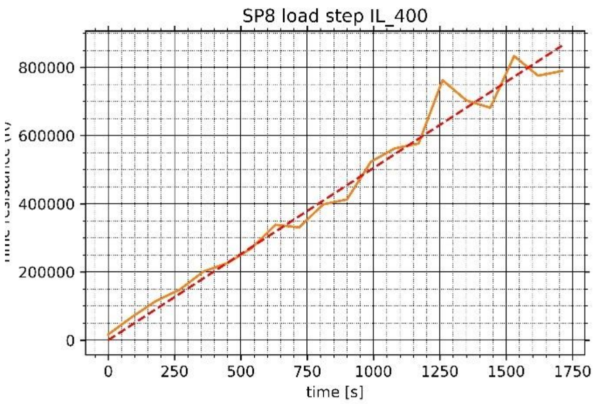


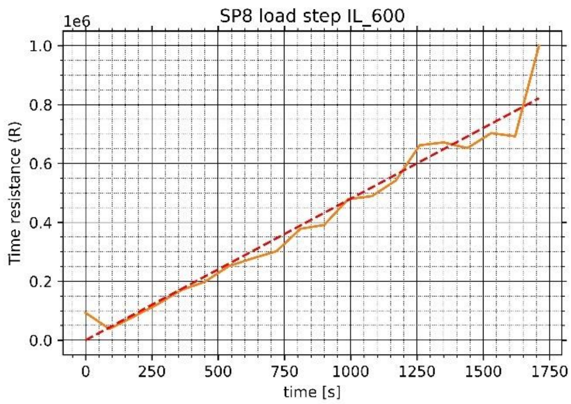




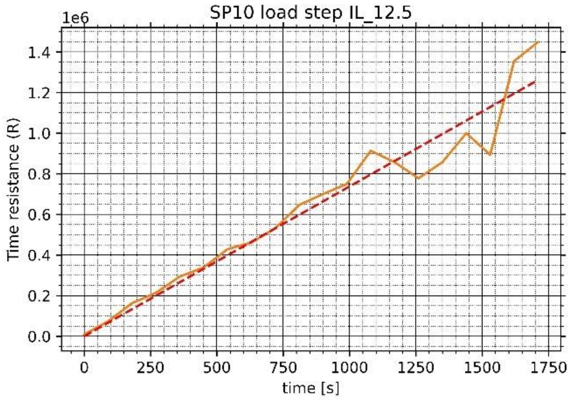


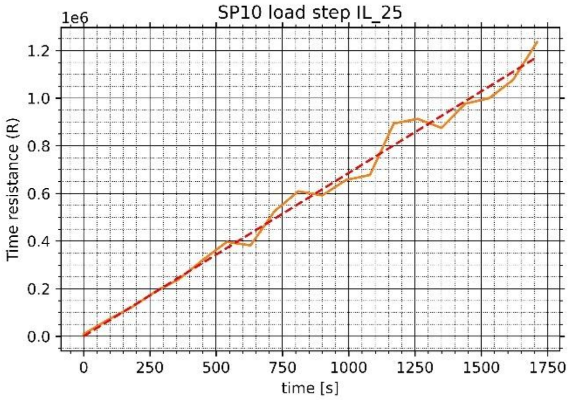


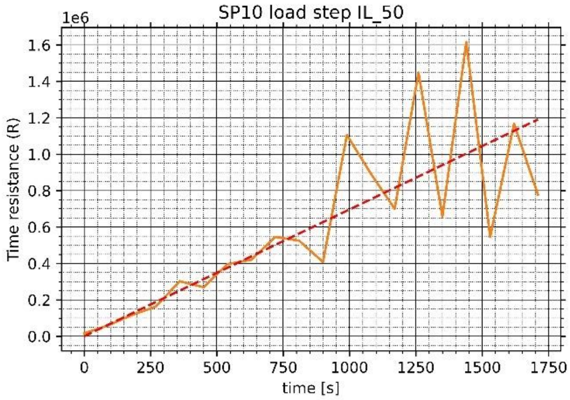


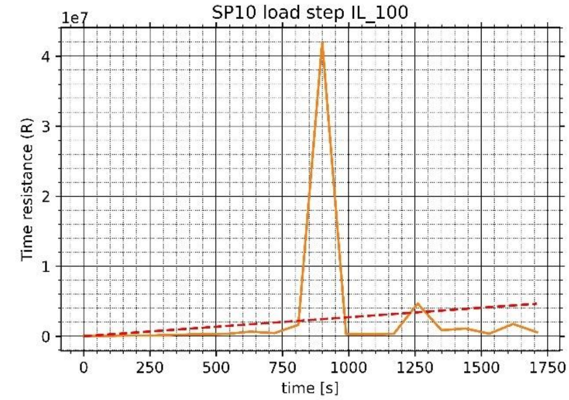


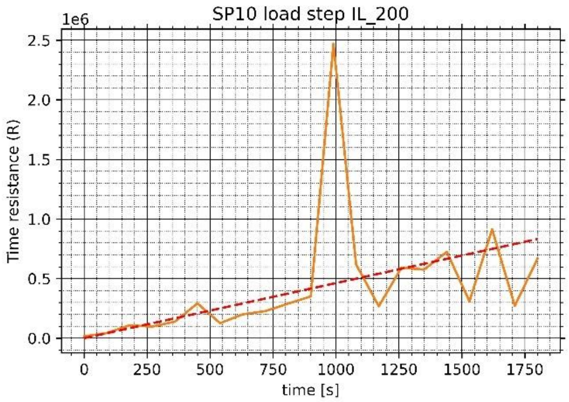
Built samples-SP10

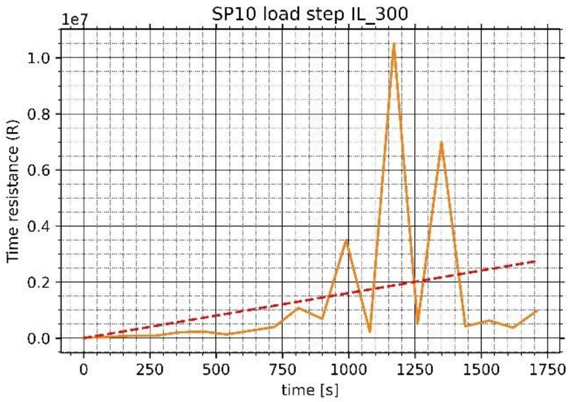


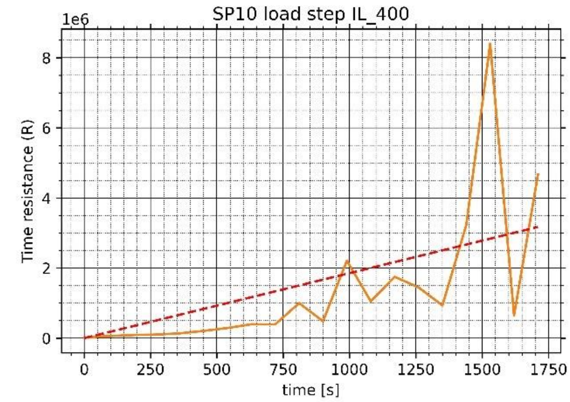


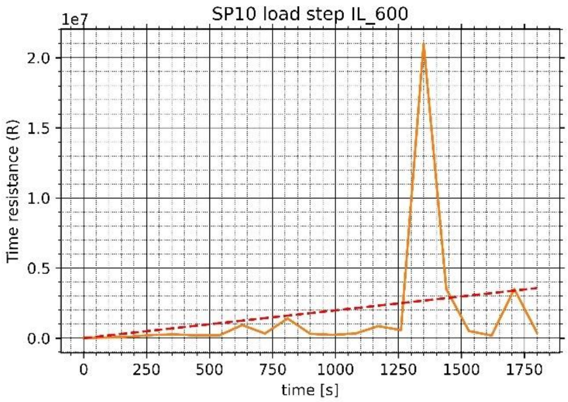




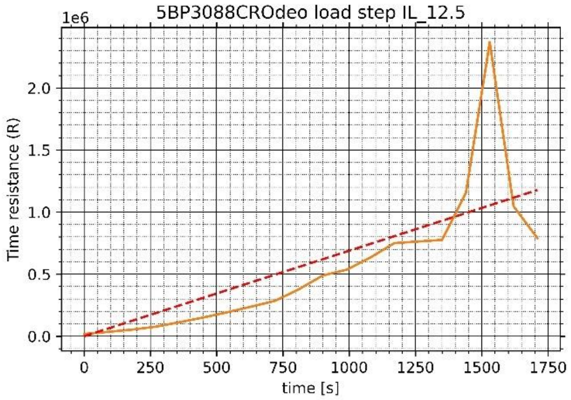


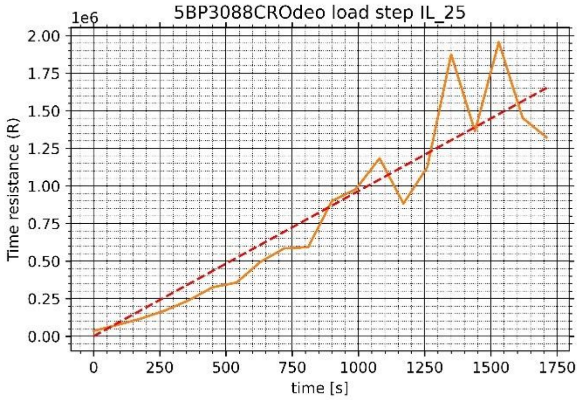


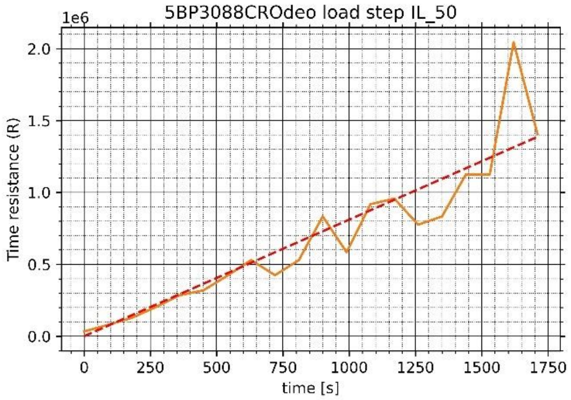


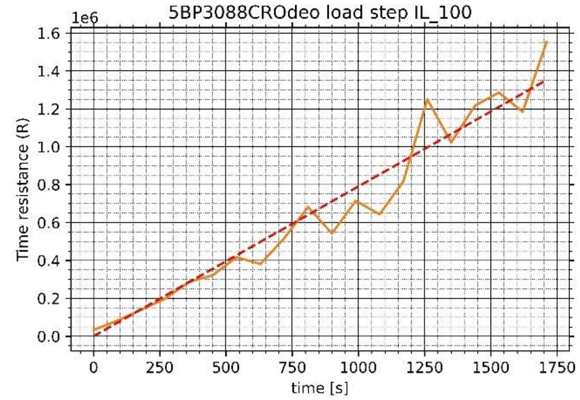


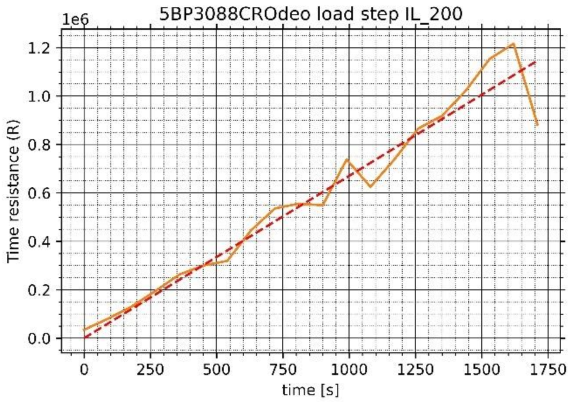
Klettelva bag samples-5BP3088CROedo

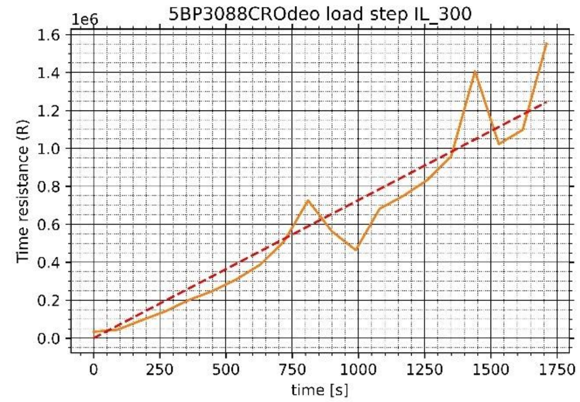


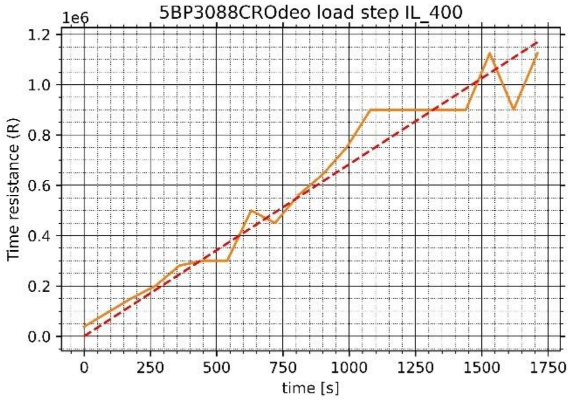


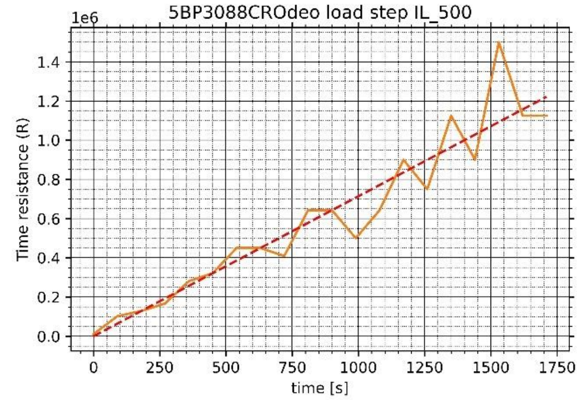




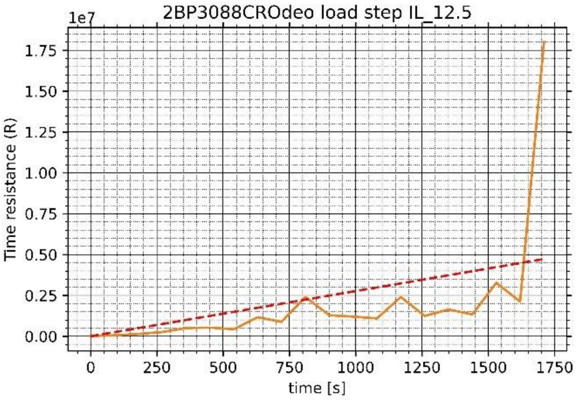


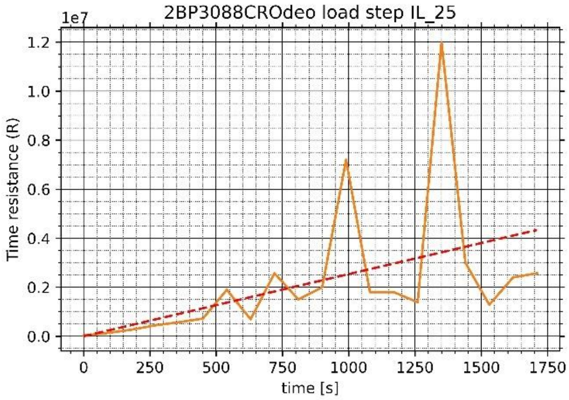


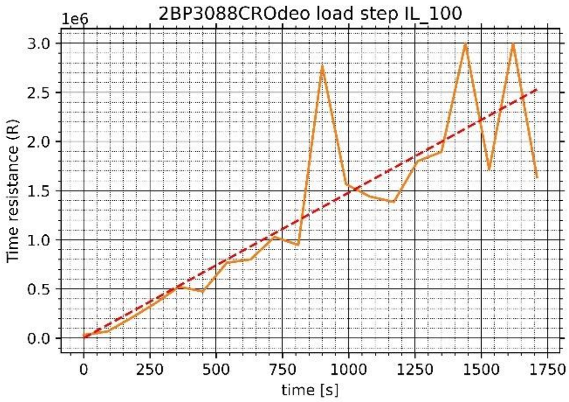


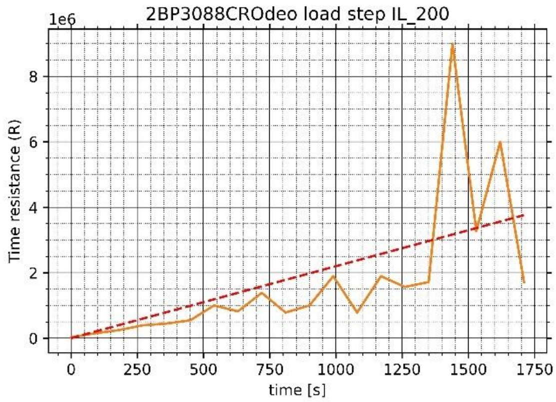


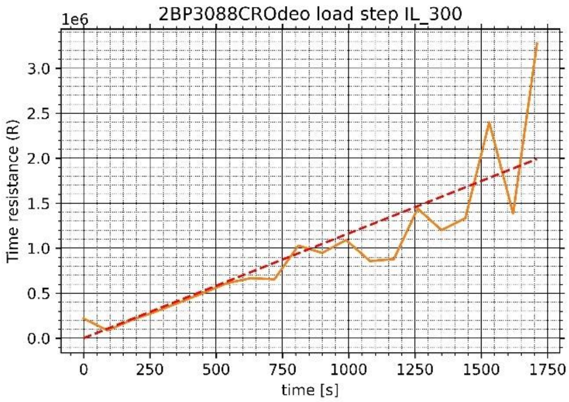
Klettelva bag samples-2BP3088CROedo

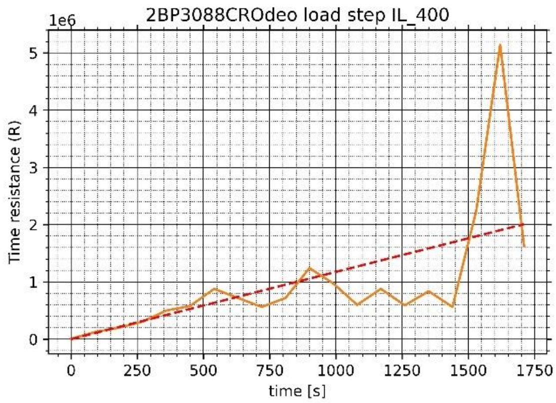


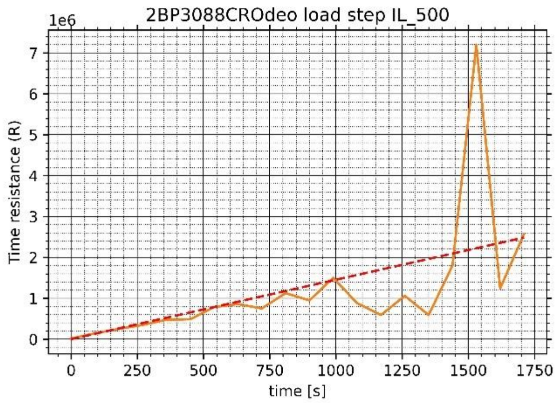




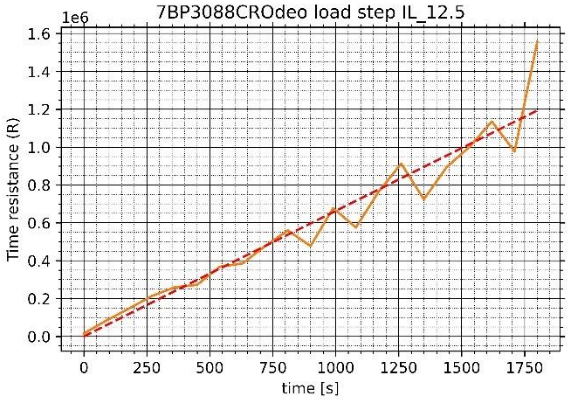


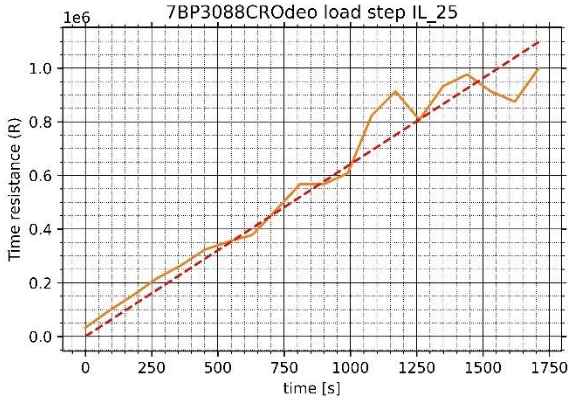


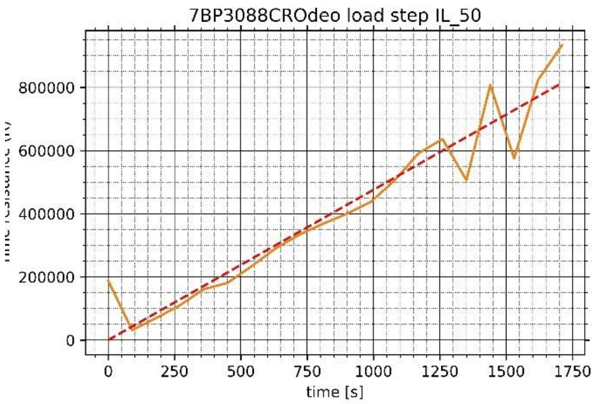


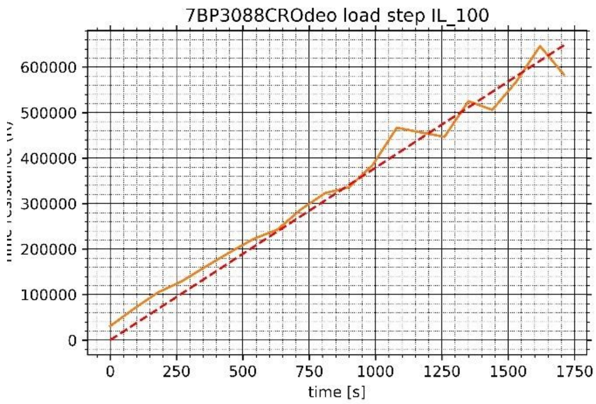


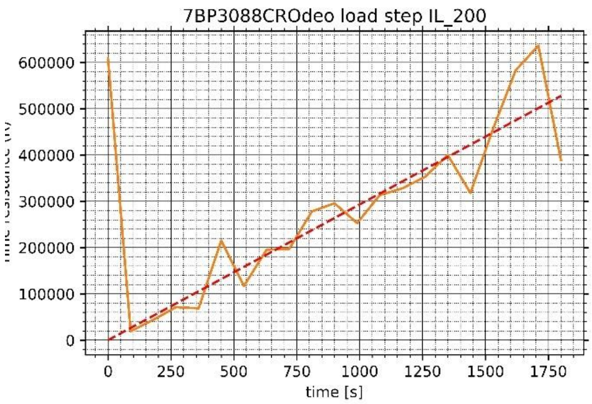
Klettelva bag samples-7BP3088CROedo

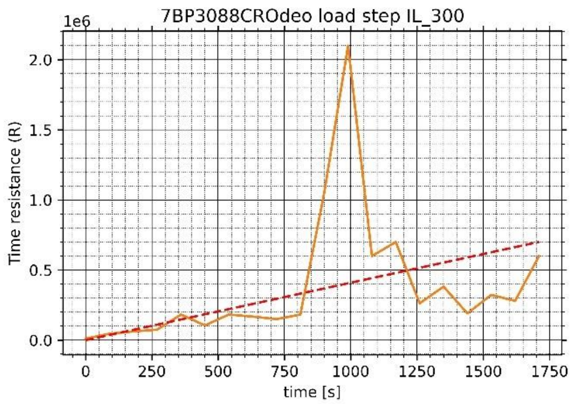


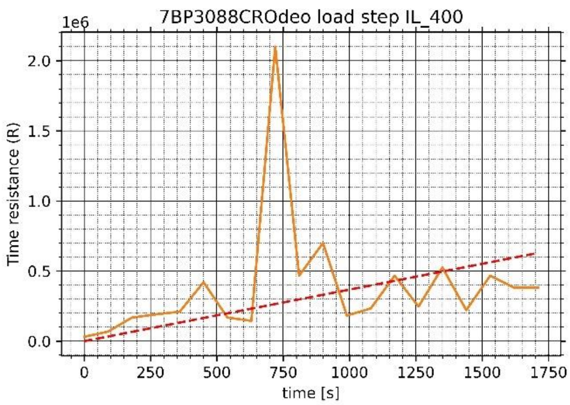


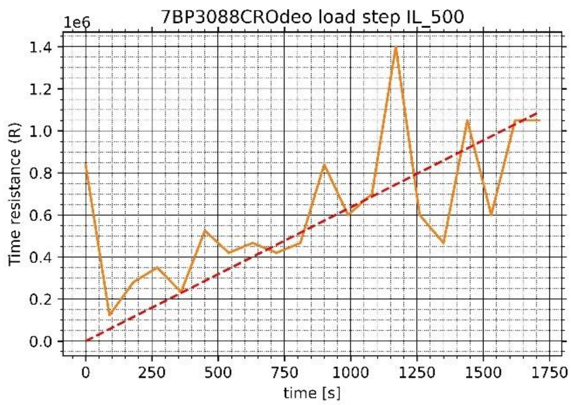




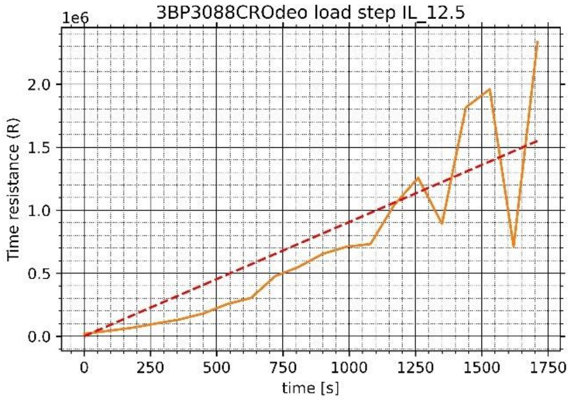


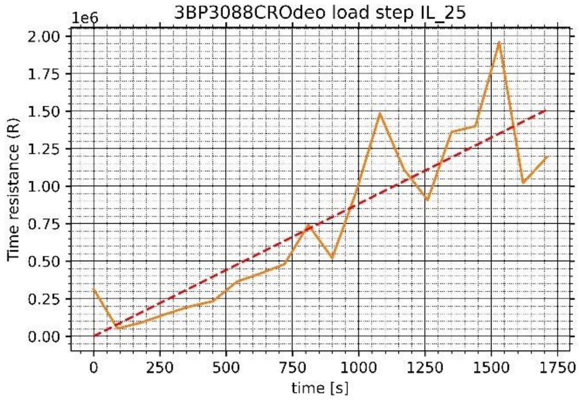


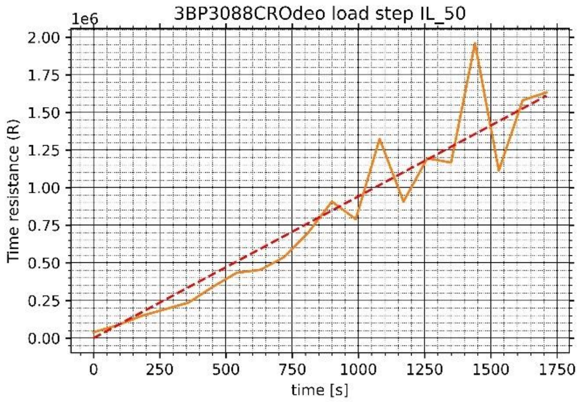


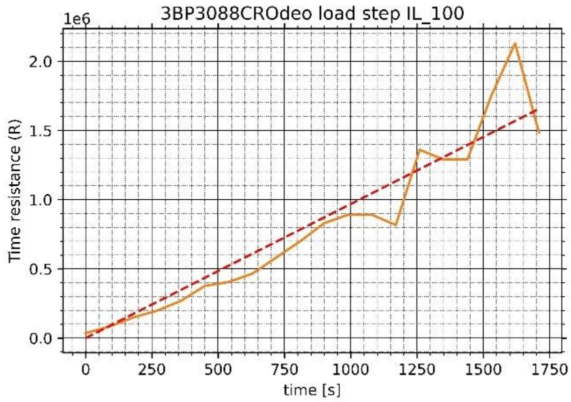


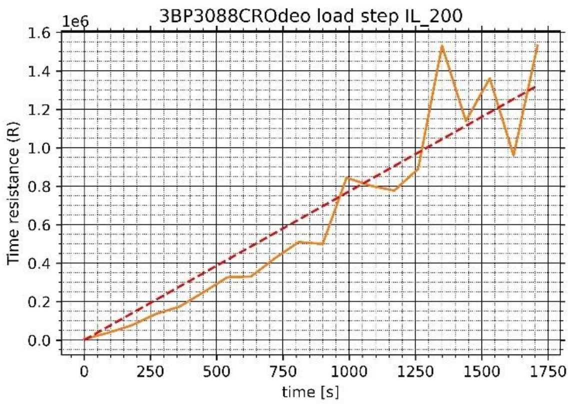
Klettelva bag samples-3BP3088CROedo

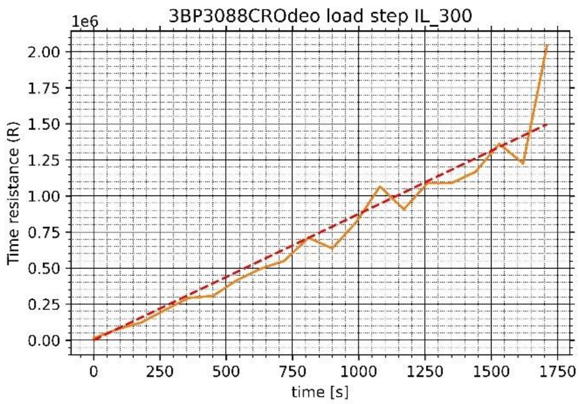


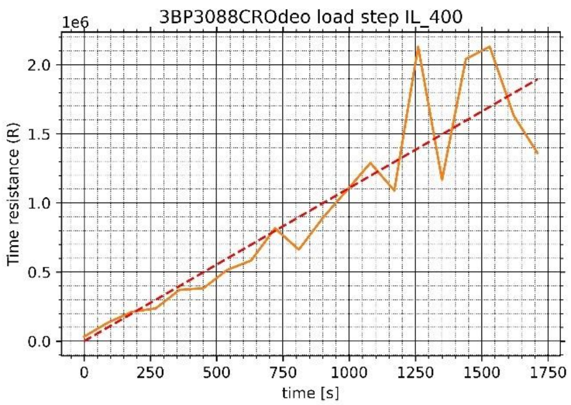


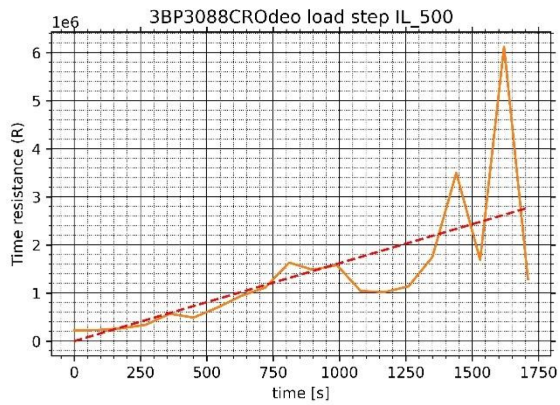




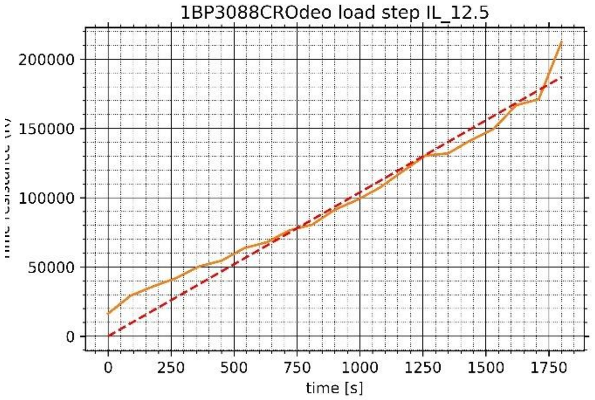


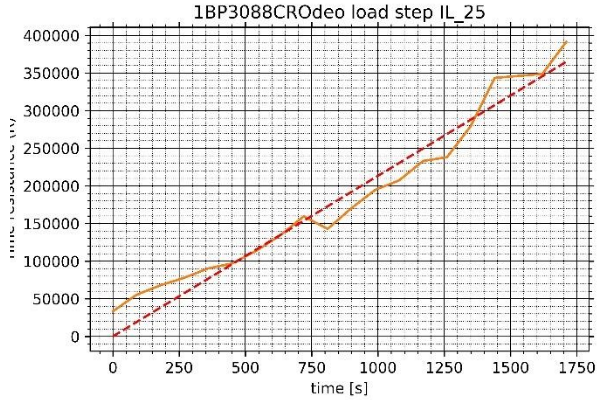


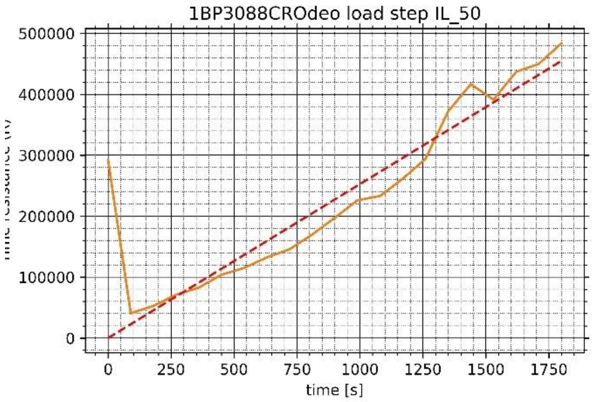


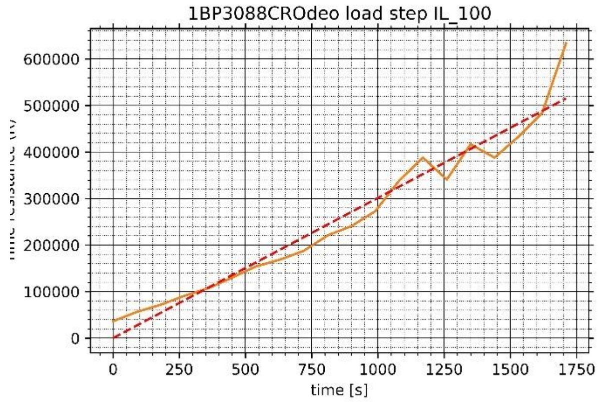


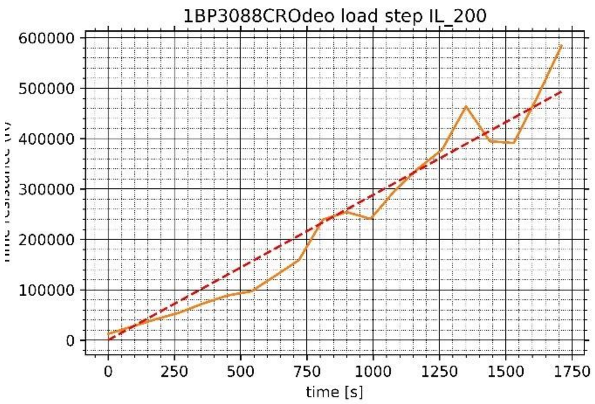
Klettelva bag samples-1BP3088CROedo

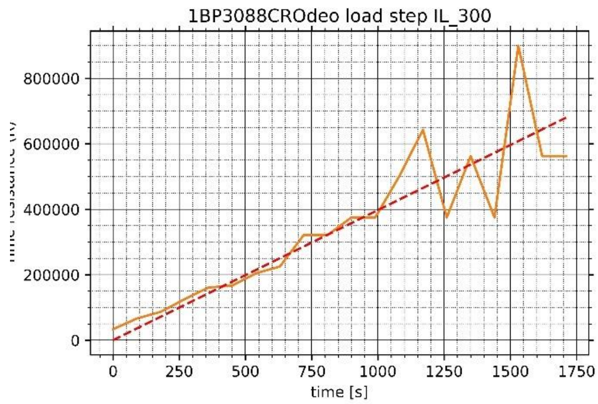


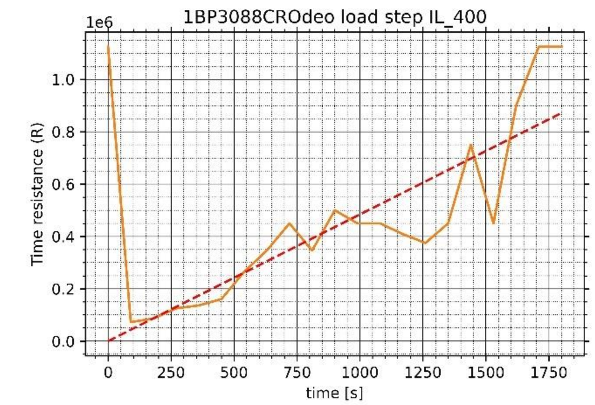


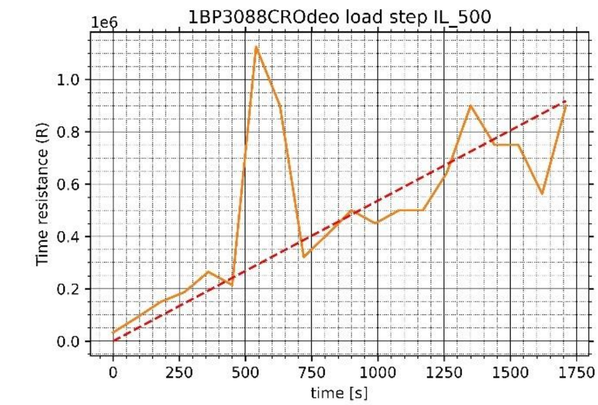




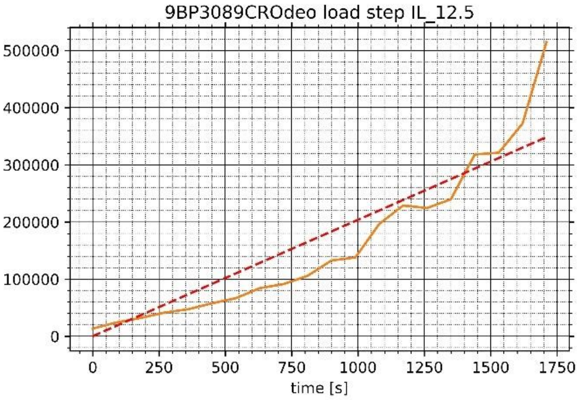


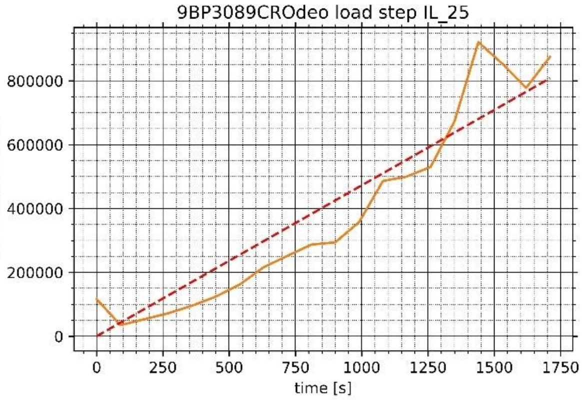


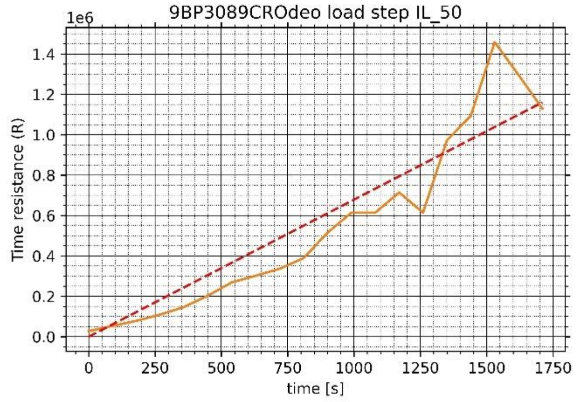


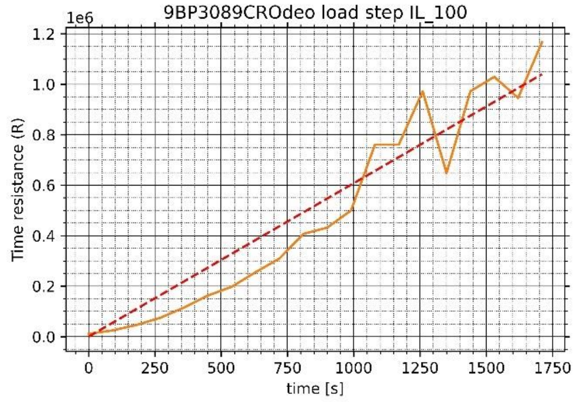


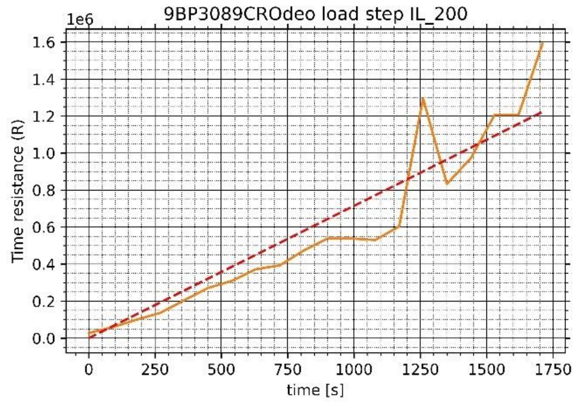
Klettelva bag samples-9BP3089CROedo

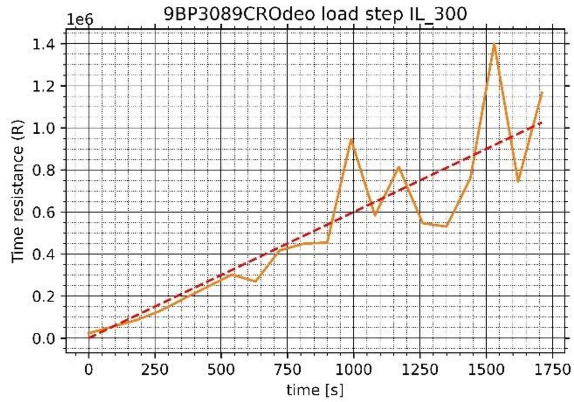


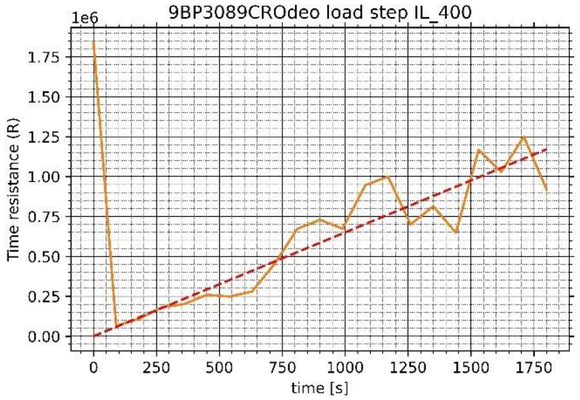


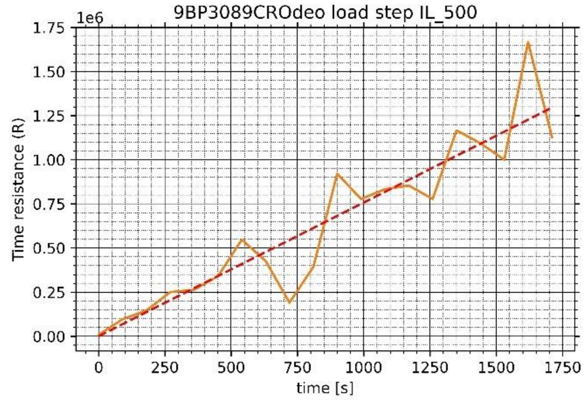




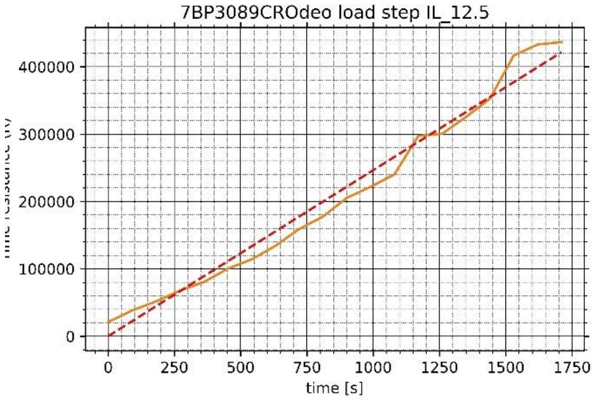


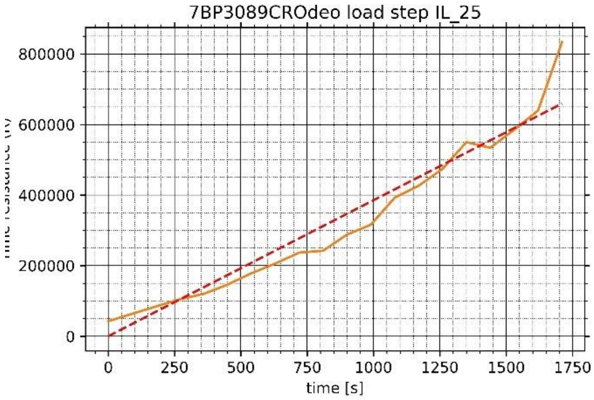


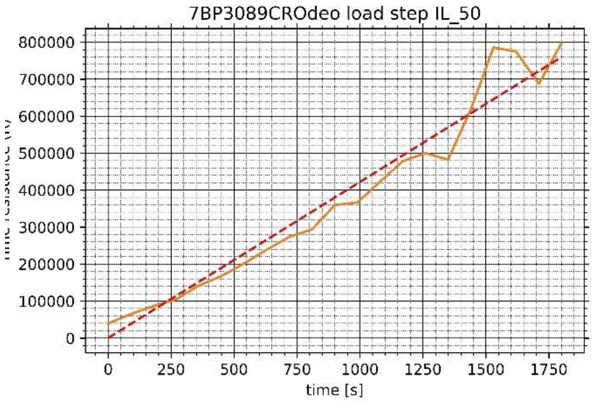


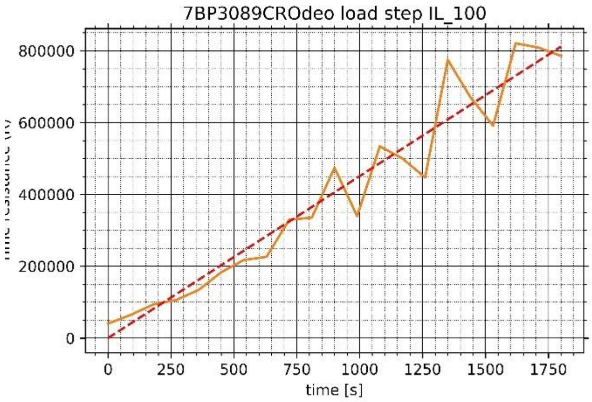


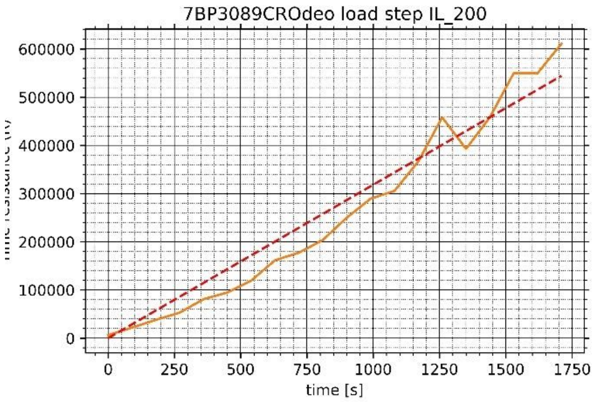
Klettelva bag samples-7BP3089CROedo

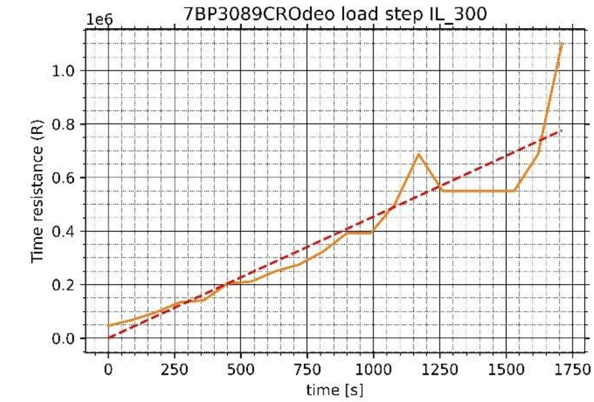


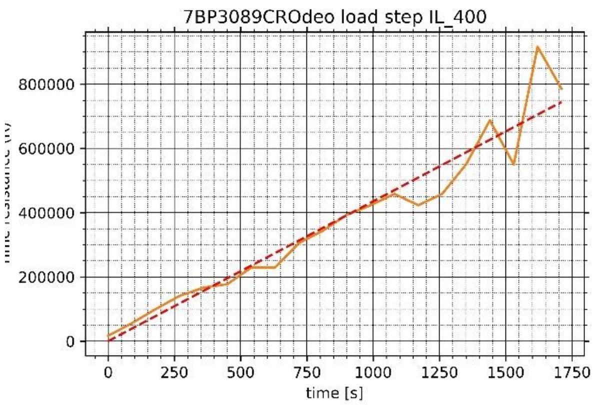


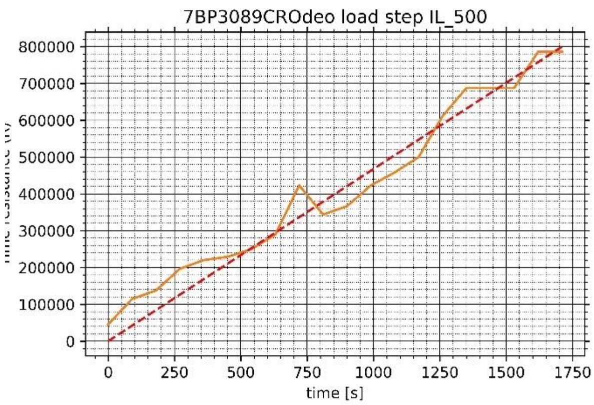




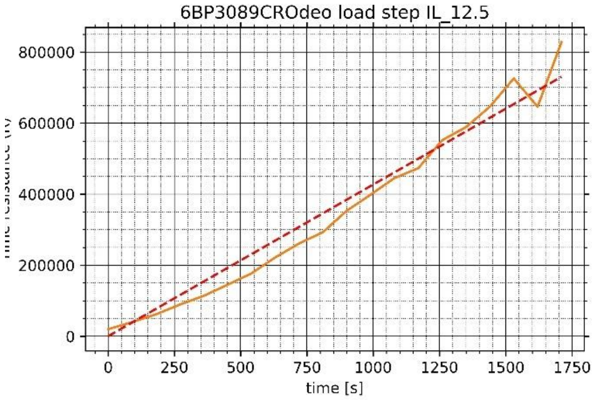


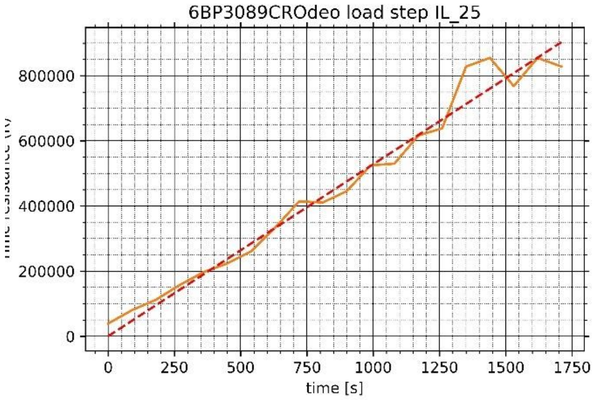


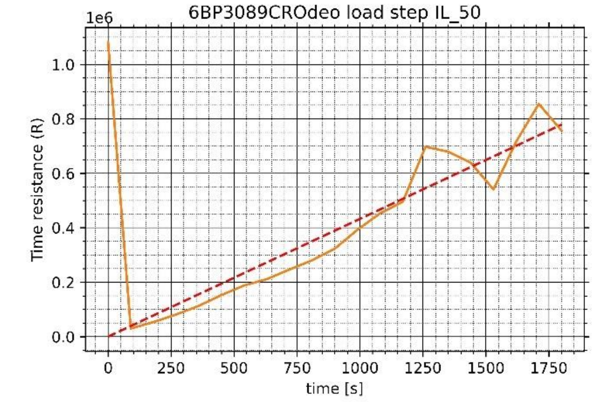


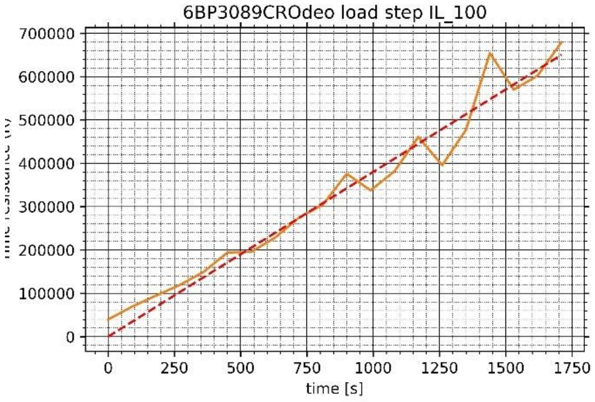


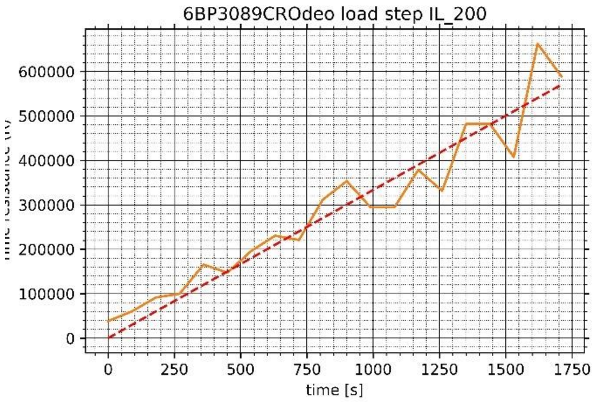
Klettelva bag samples-6BP3089CROedo

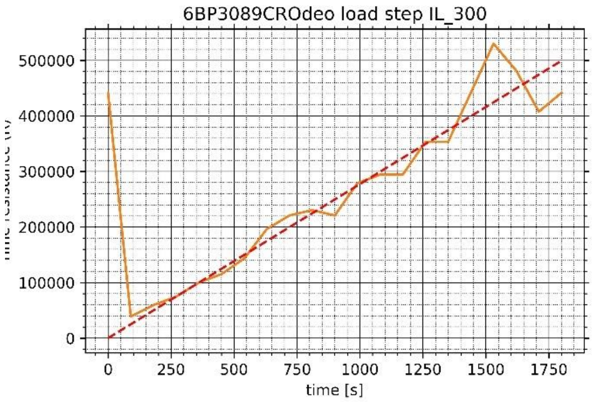


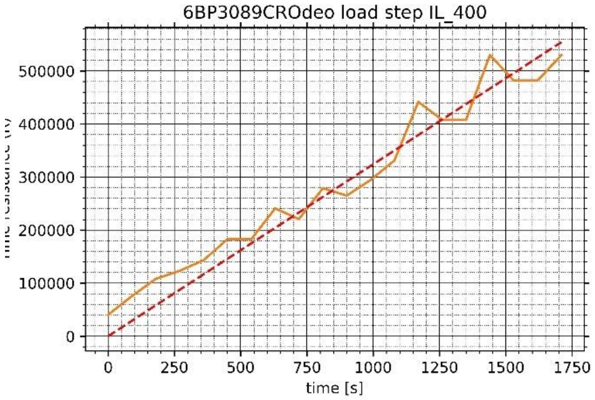


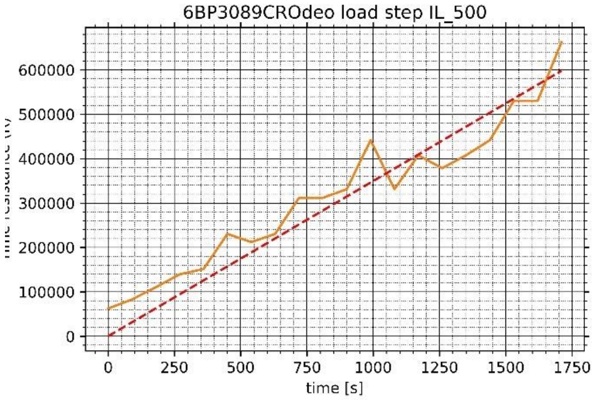




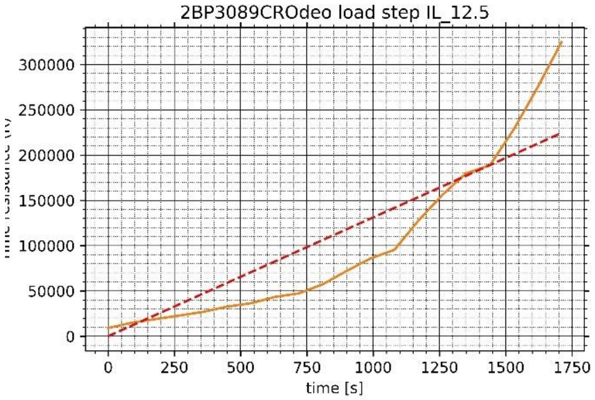


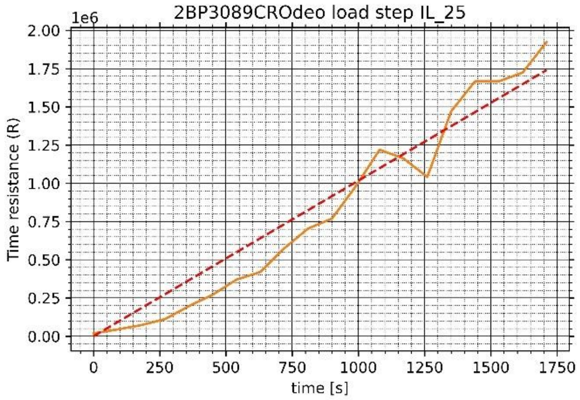


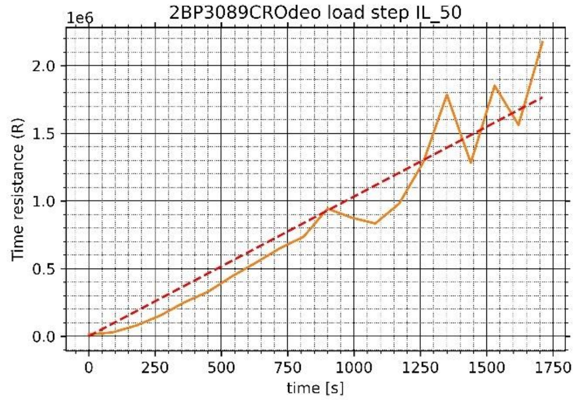


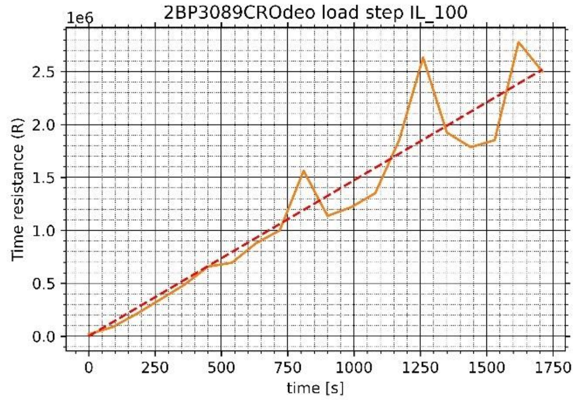


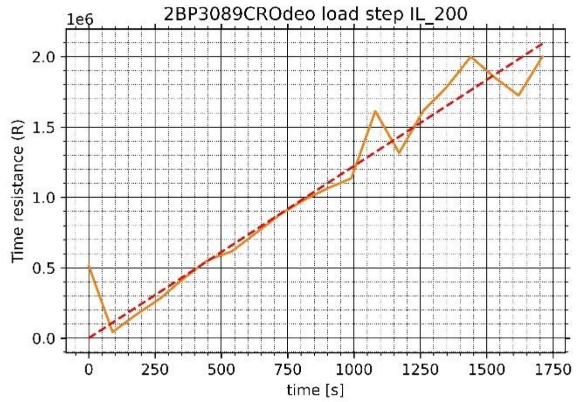
Klettelva bag samples-2BP3089CROedo

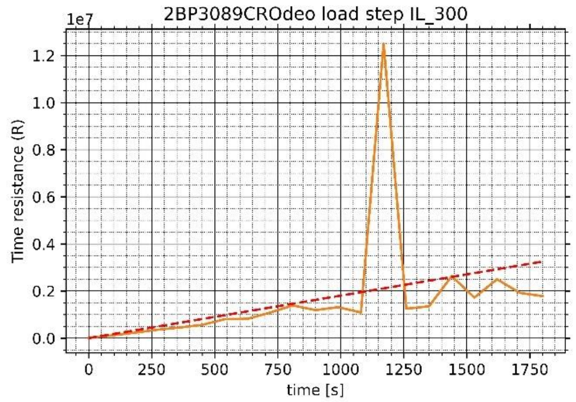


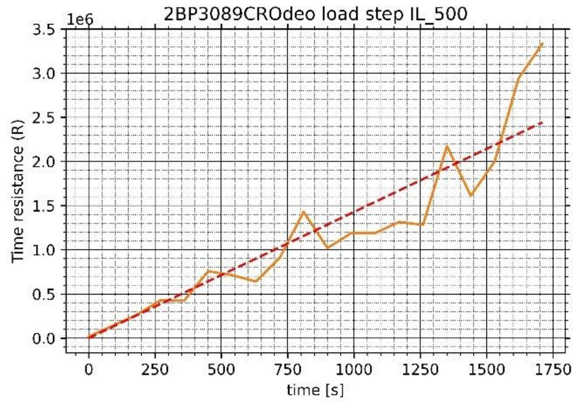




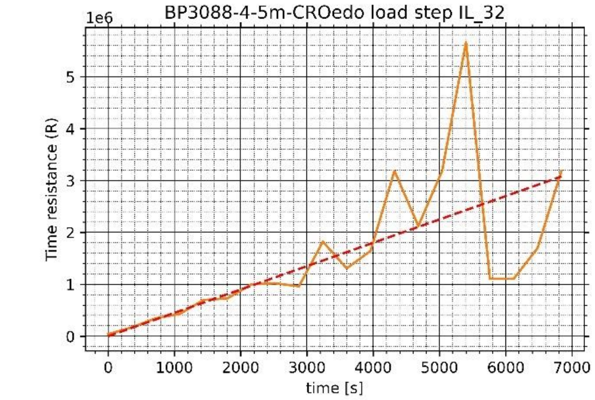


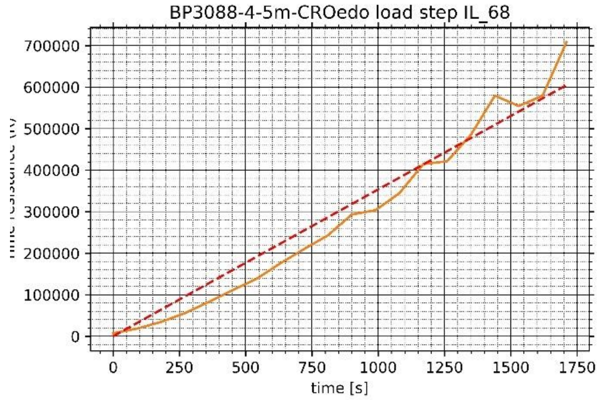


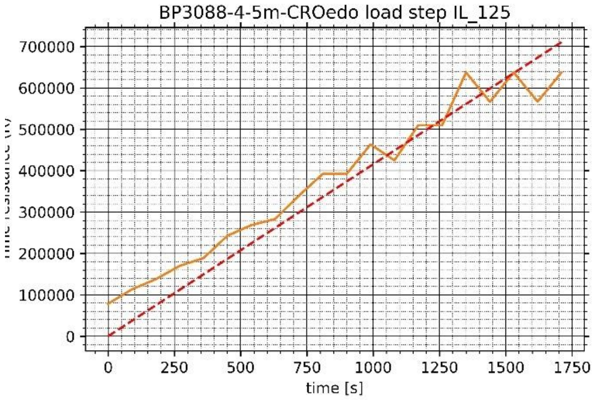


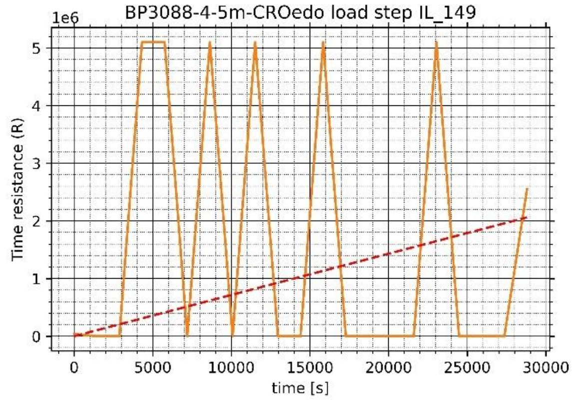


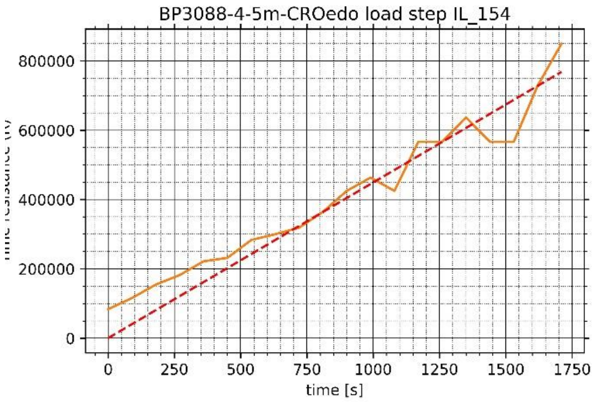
Klettelva cylinder samples-12BP3088-4-5m-CROedo

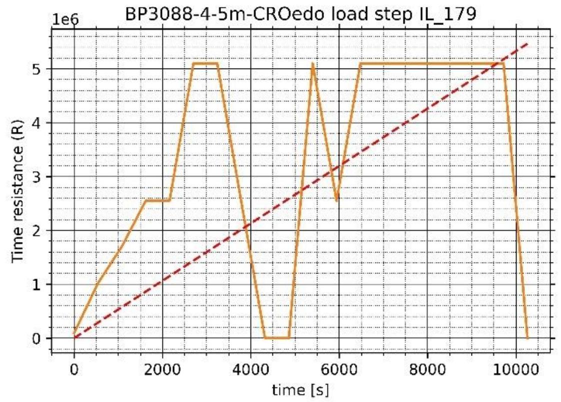


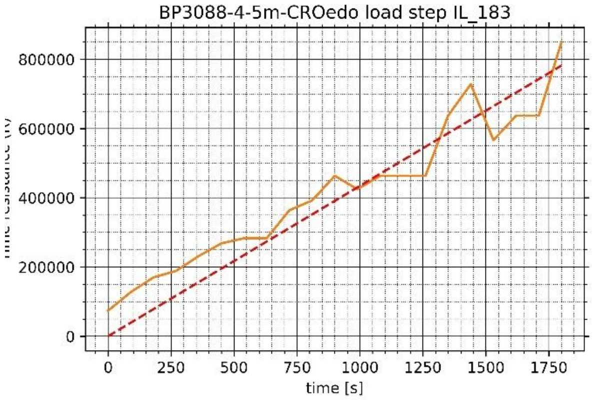


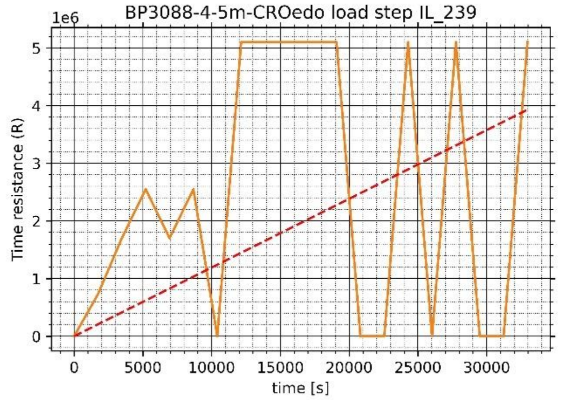




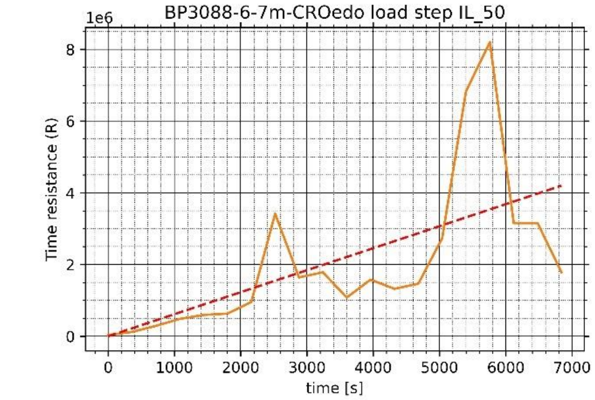


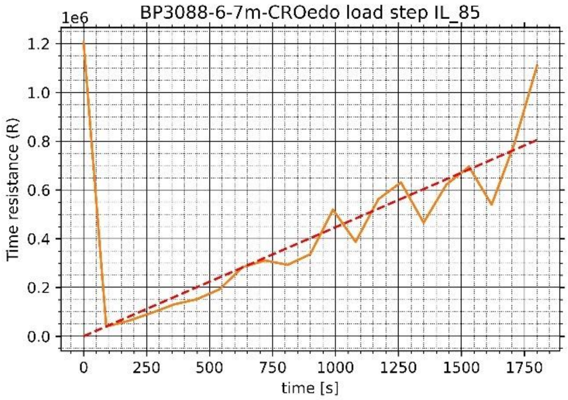


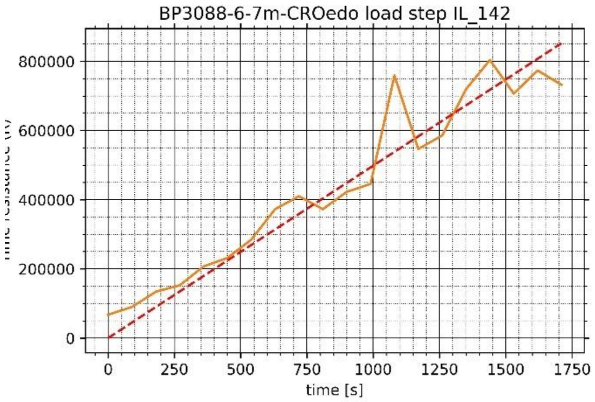


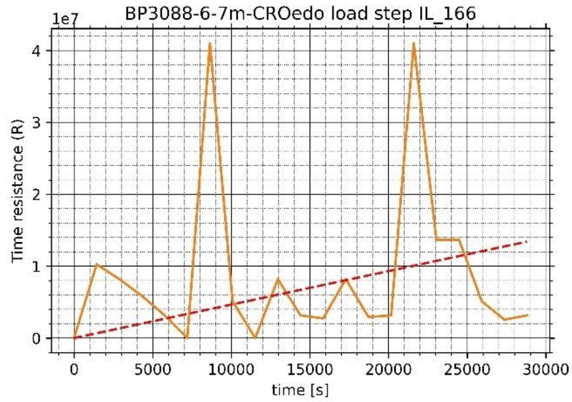


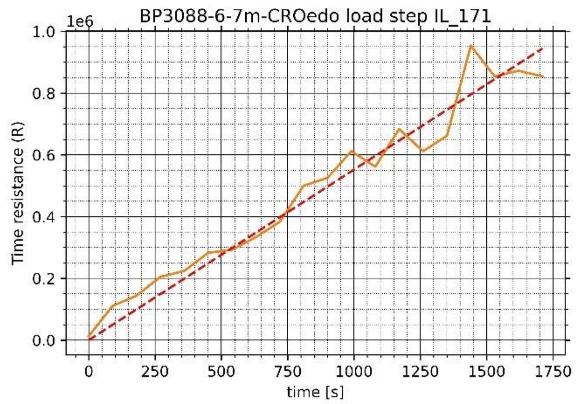
Klettelva cylinder samples-4BP3088-6-7m-CROedo

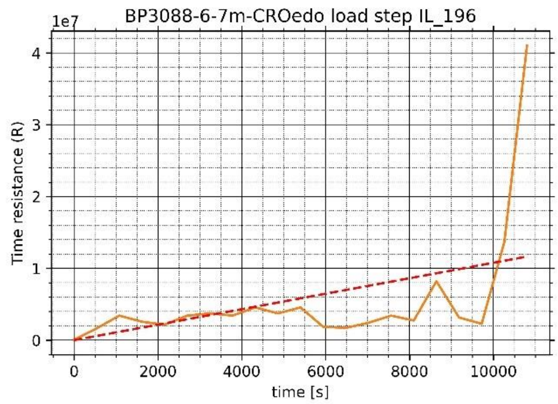


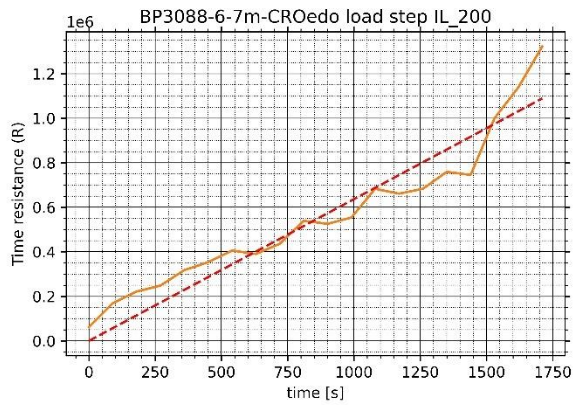


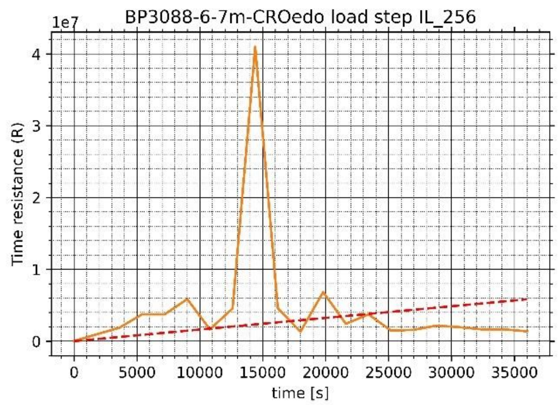




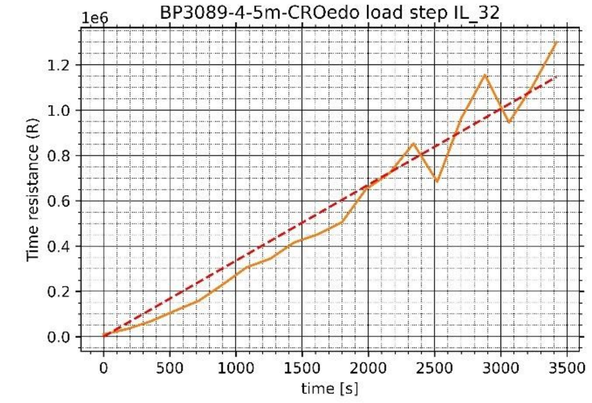


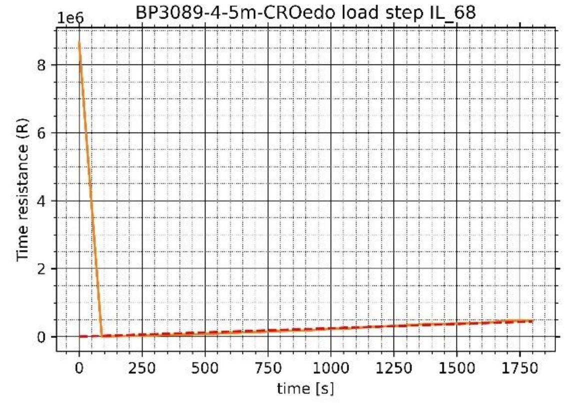


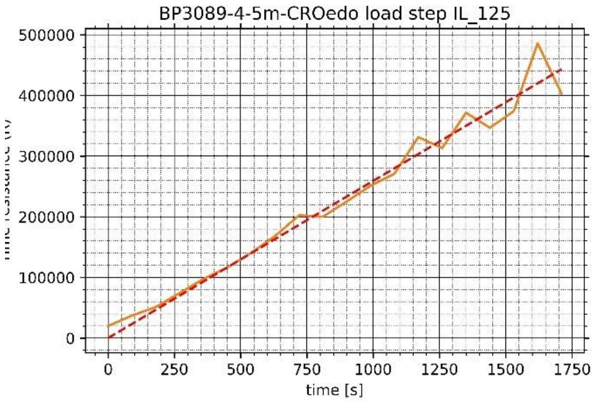


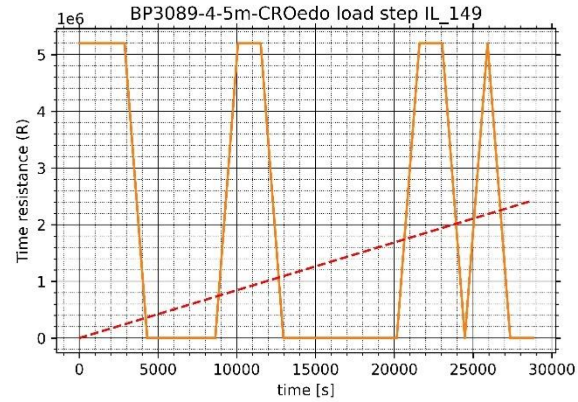


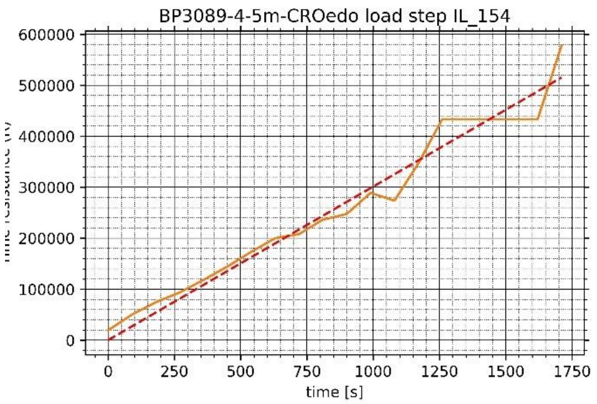
Klettelva cylinder samples-7BP308-4-5m-CROedo

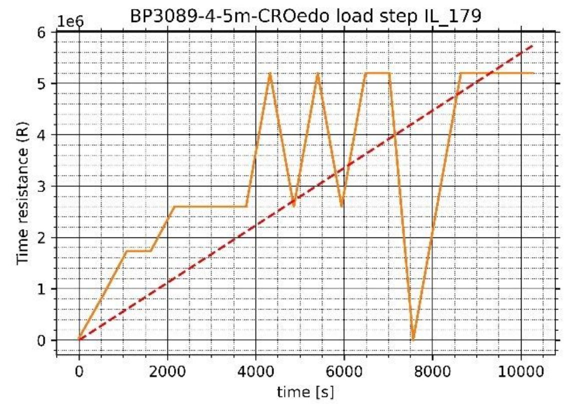


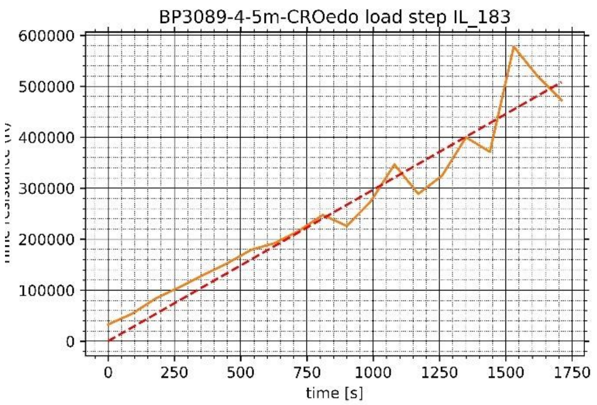


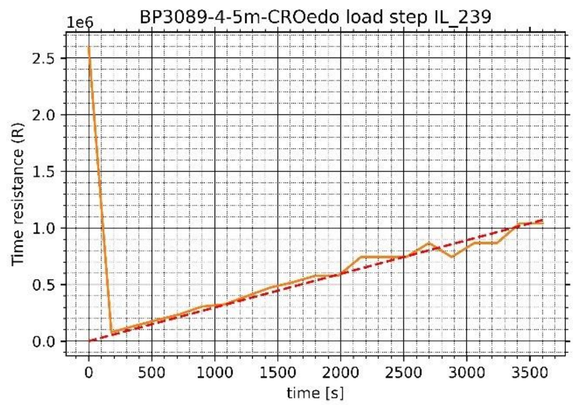




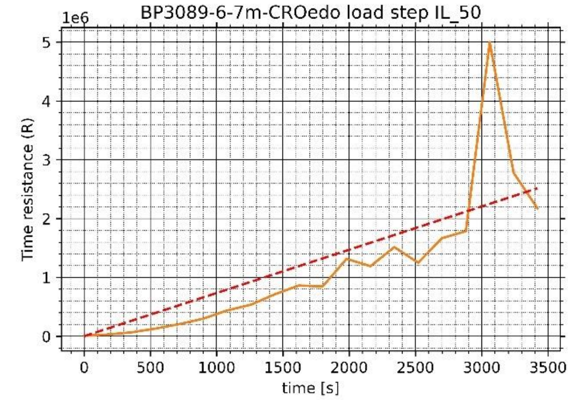


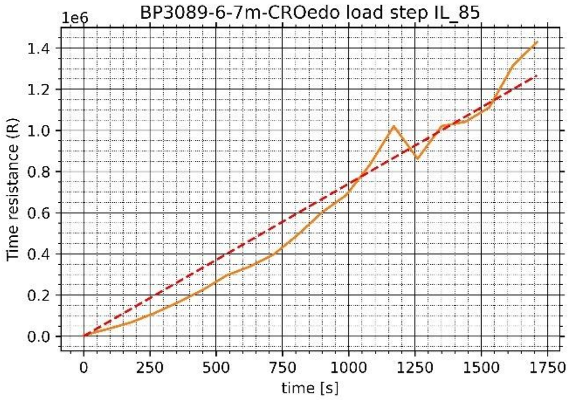


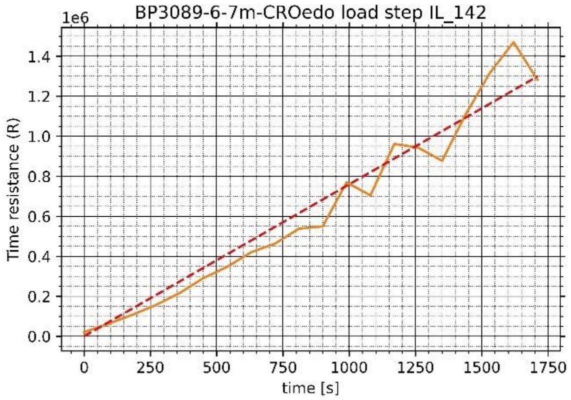


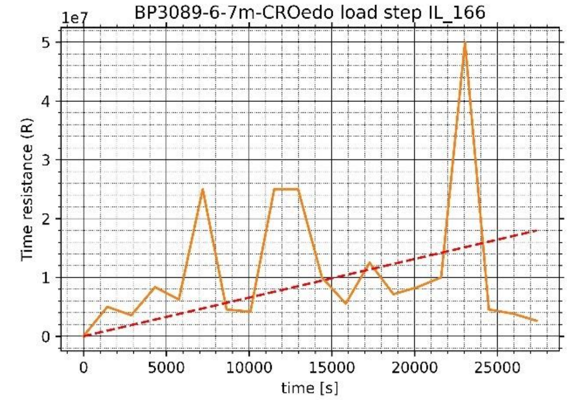


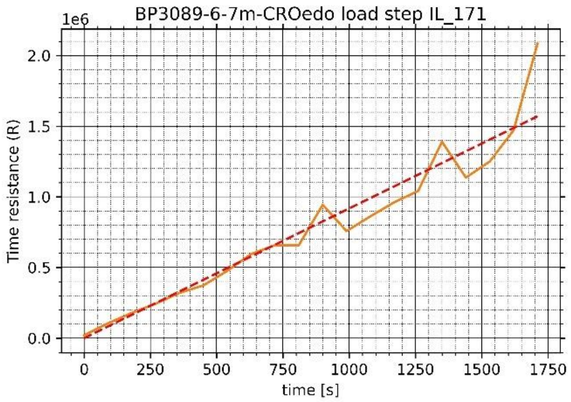
Klettelva cylinder samples-1BP3089-6-7m-CROedo

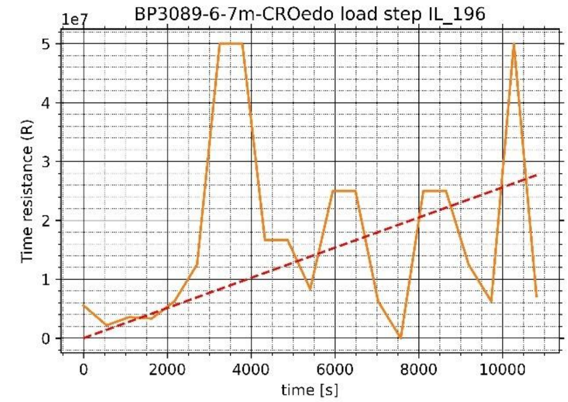


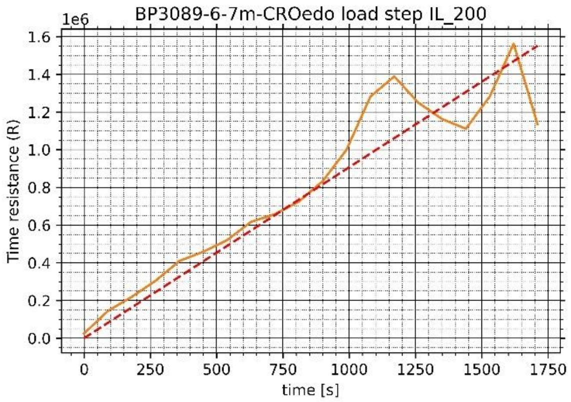


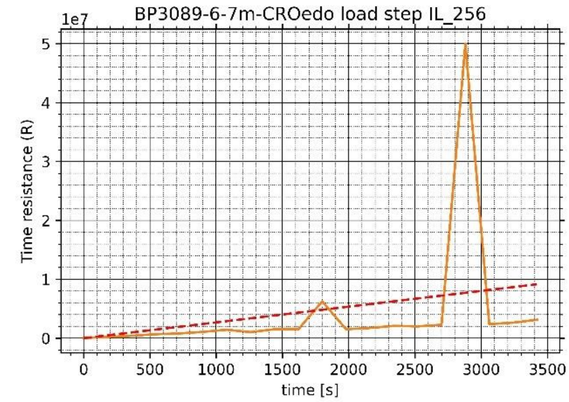




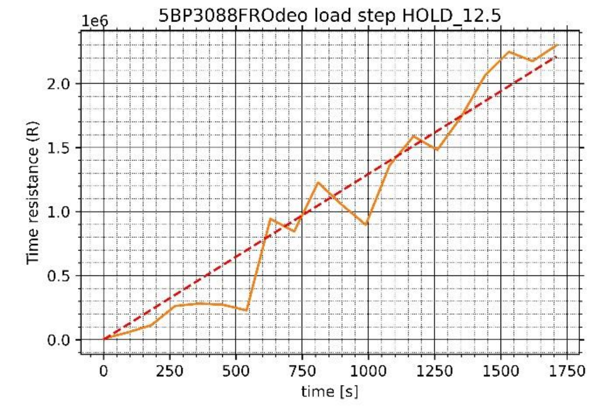


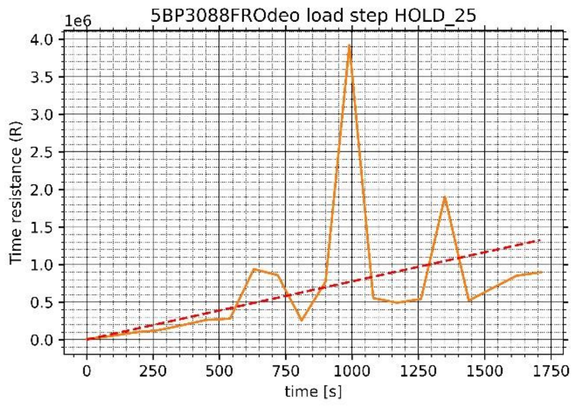


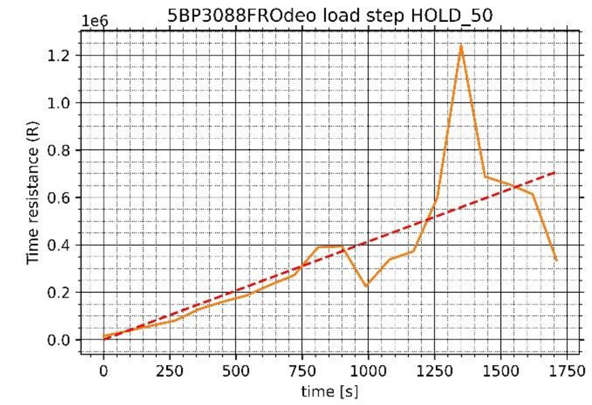


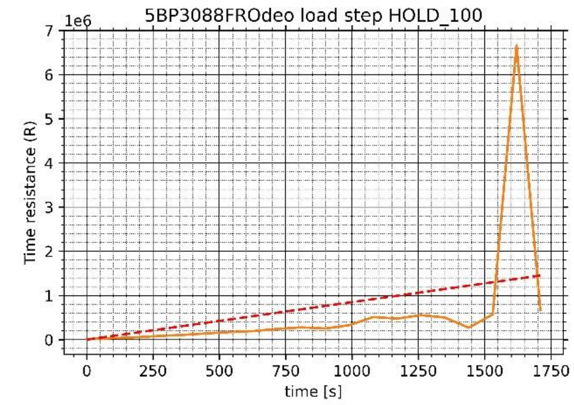


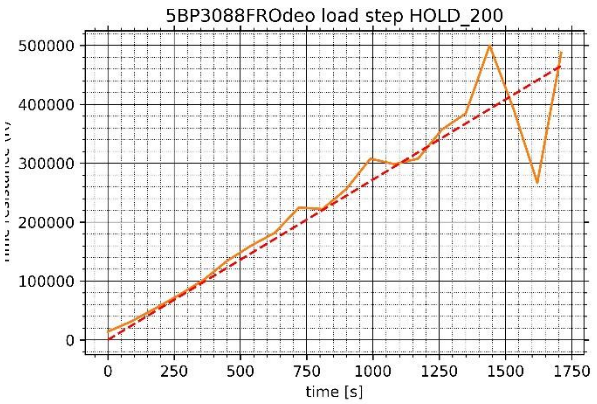
Klettelva bag samples-5BP3088FROedo

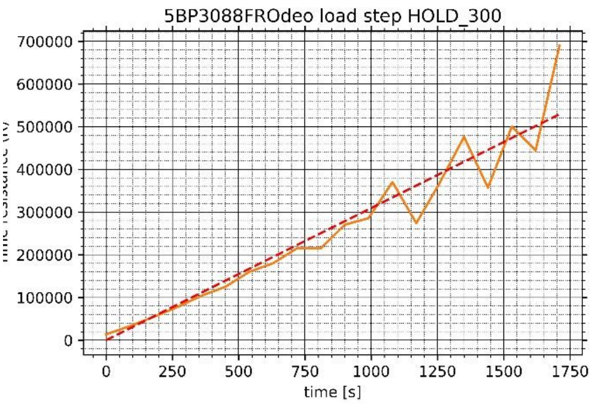


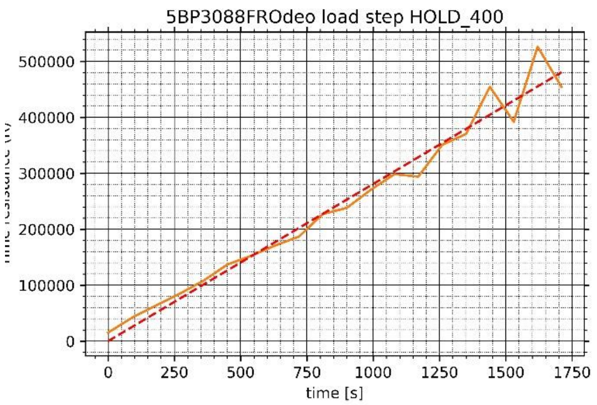


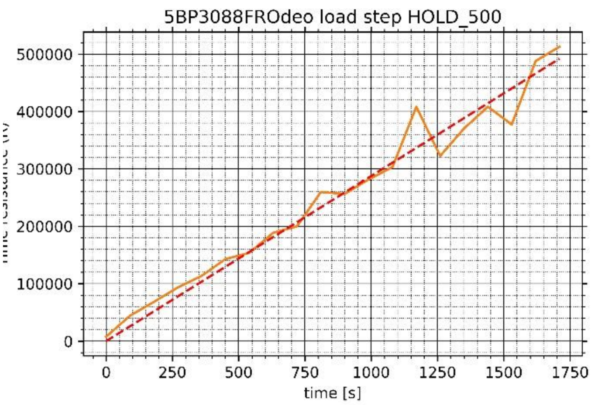




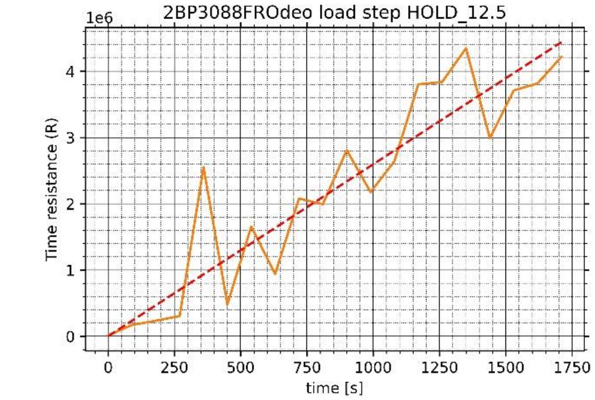


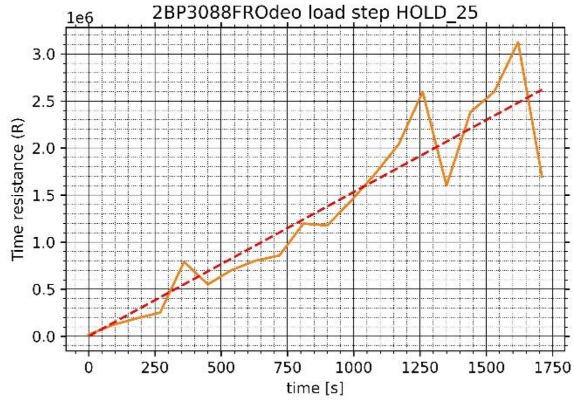


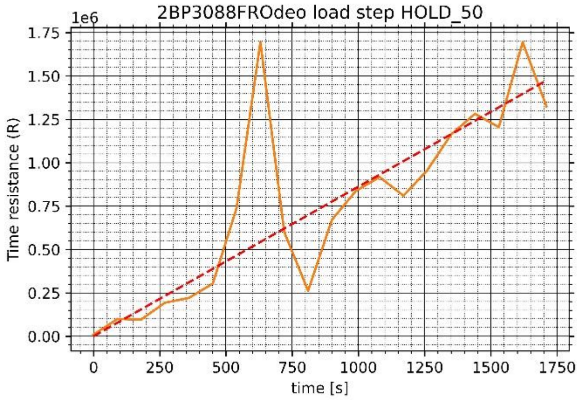


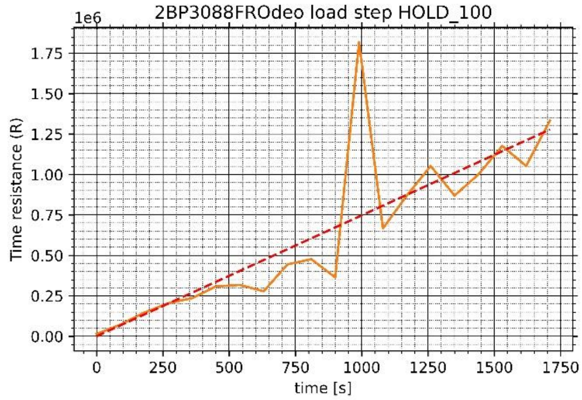


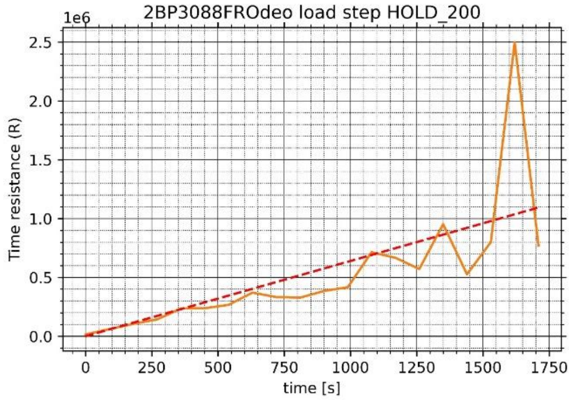
Klettelva bag samples-2BP3088FROdeo

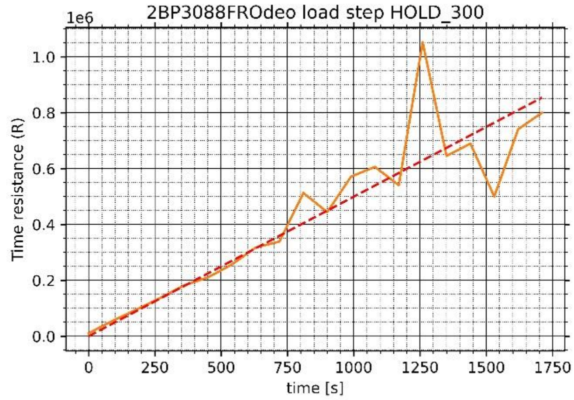


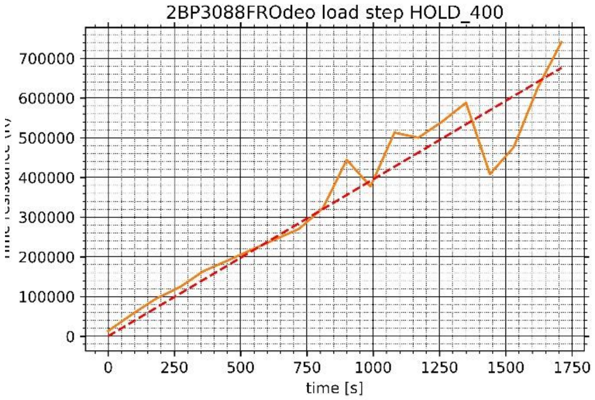


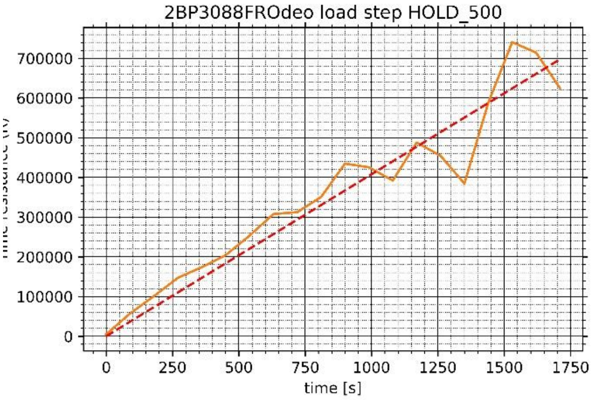




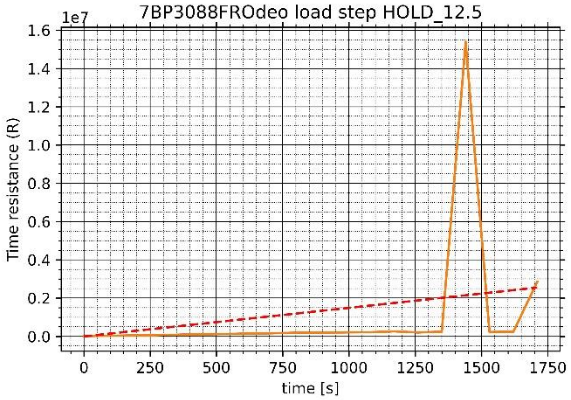


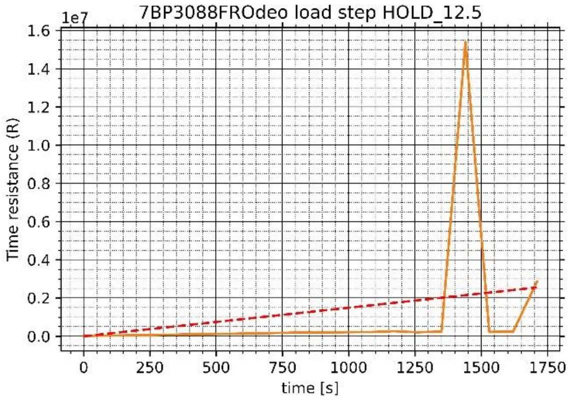


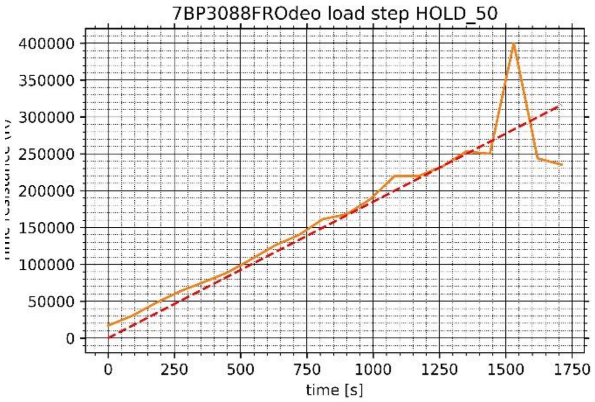


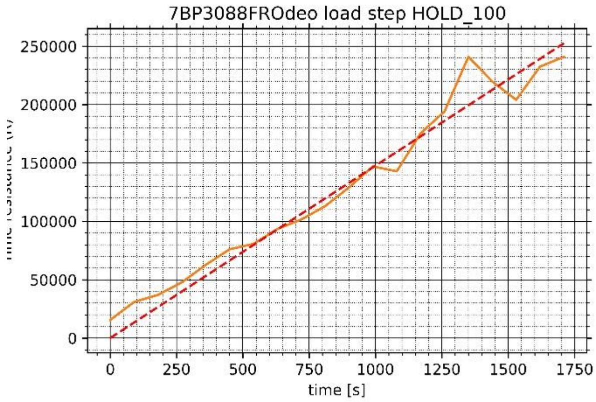


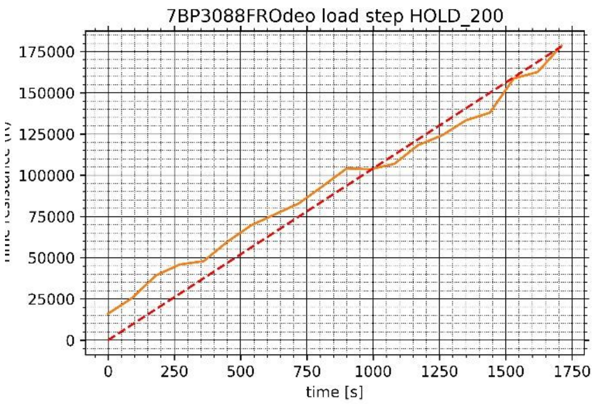
Klettelva bag samples-7BP3088FROdeo

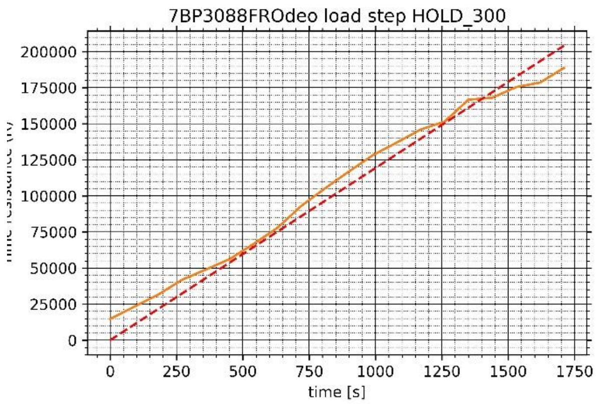


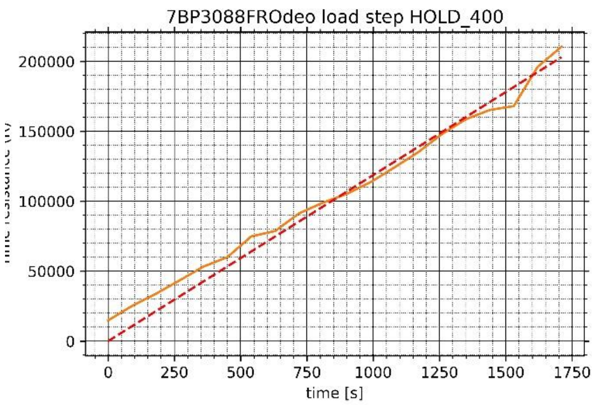


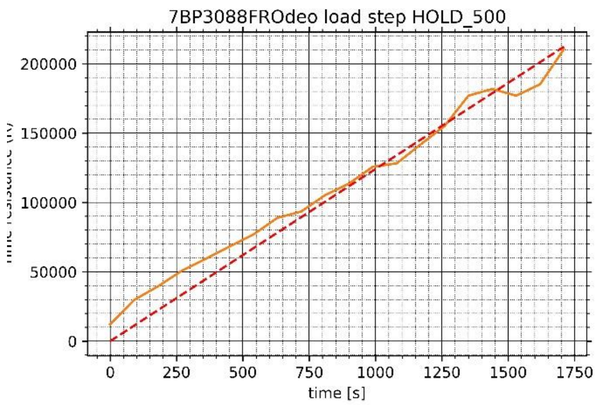




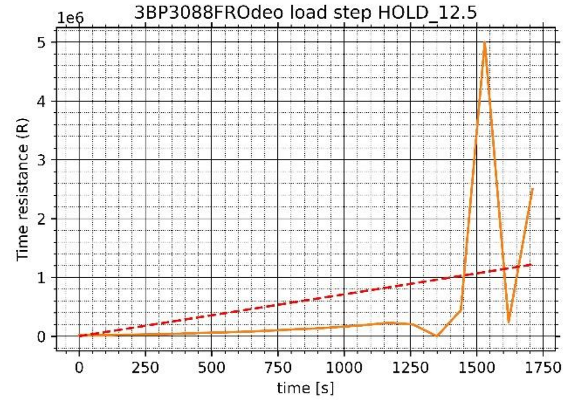


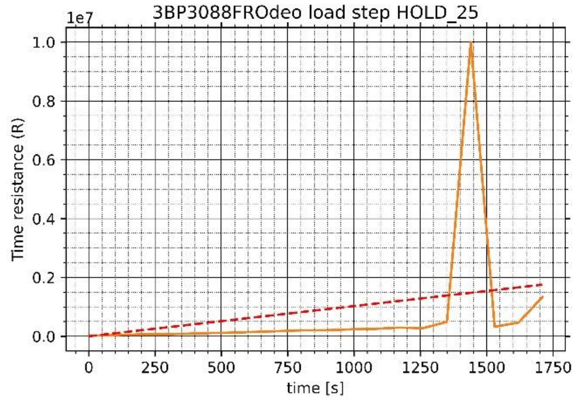


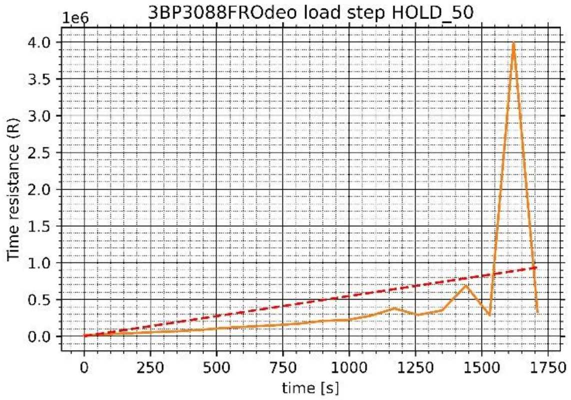


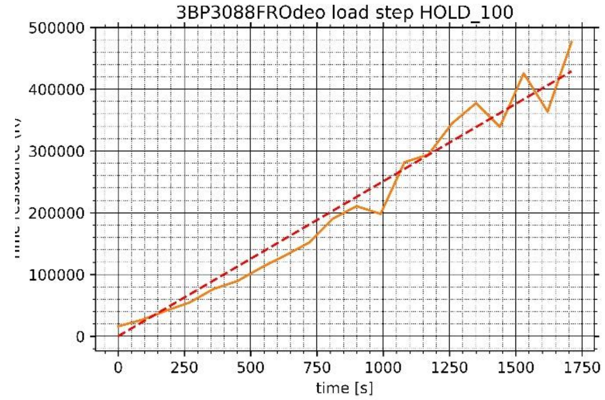


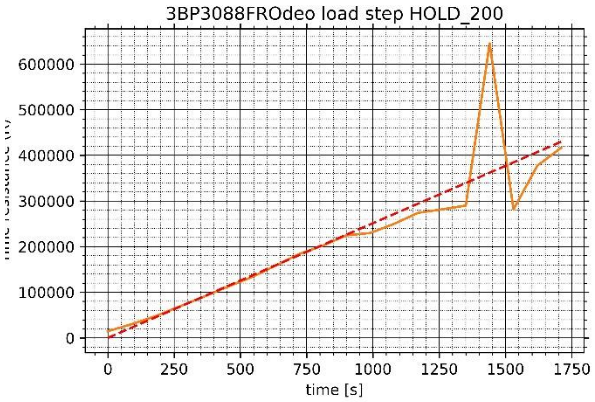
Klettelva bag samples-3BP3088FROdeo

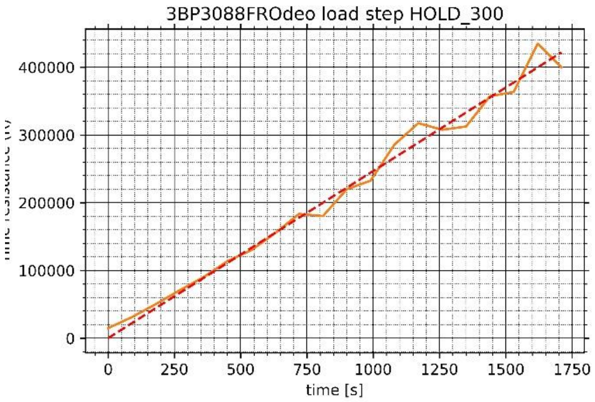


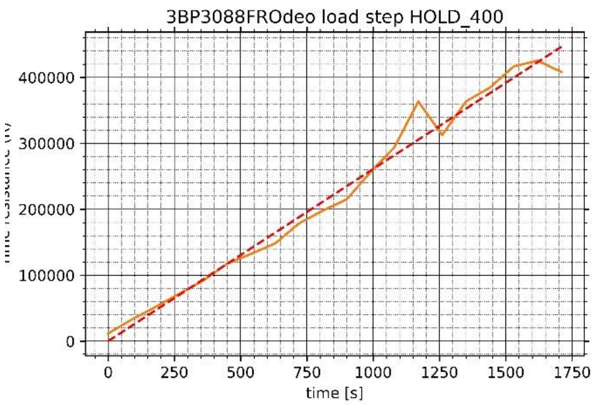


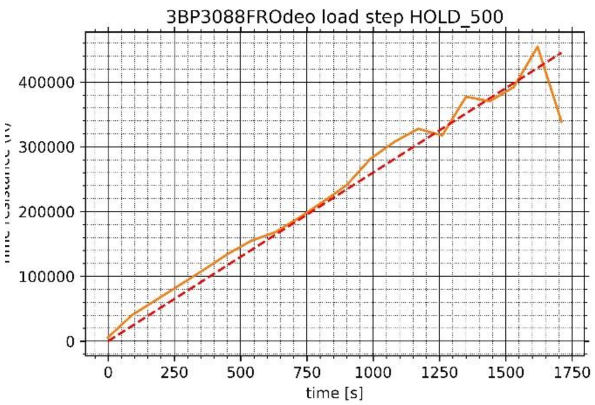




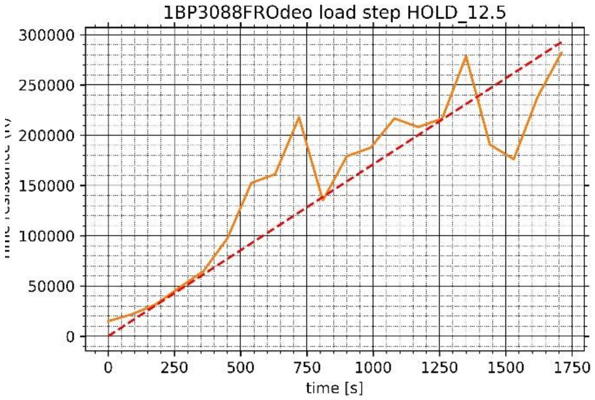


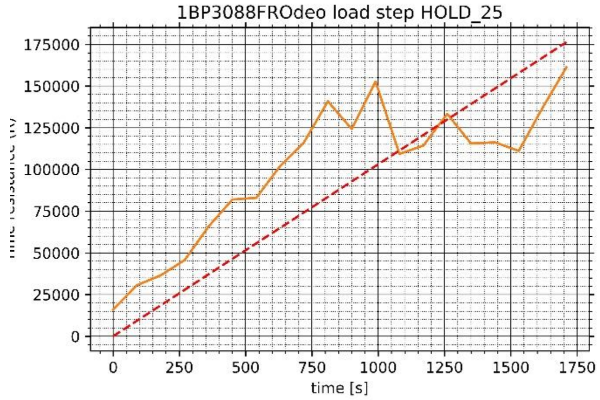


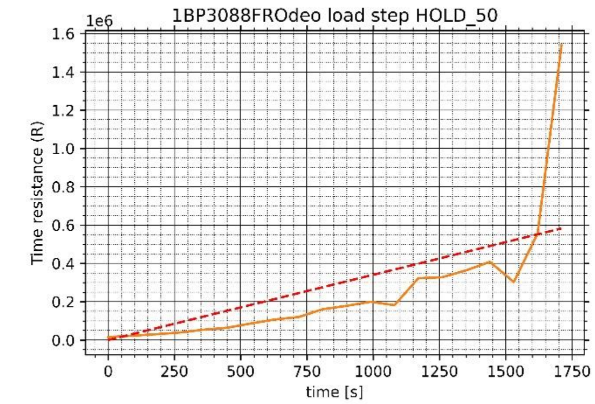


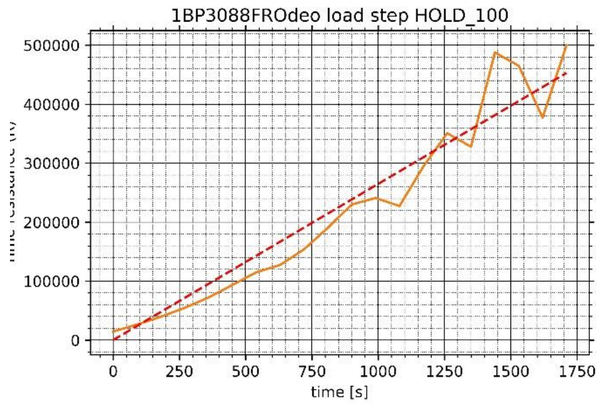


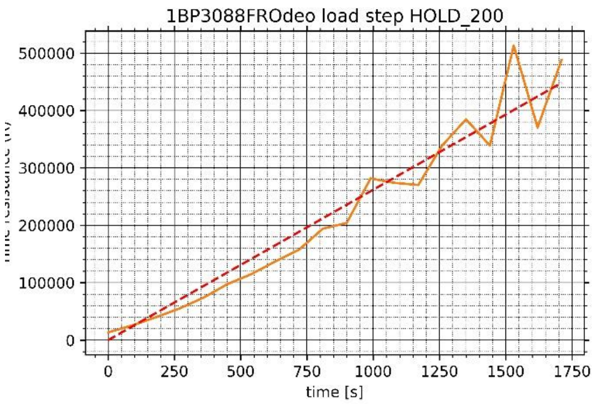
Klettelva bag samples-1BP3088FROdeo

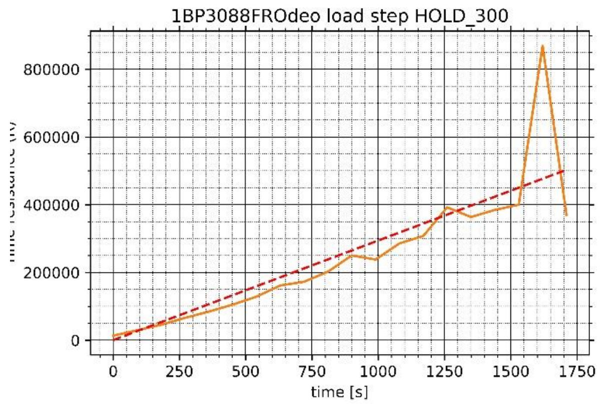


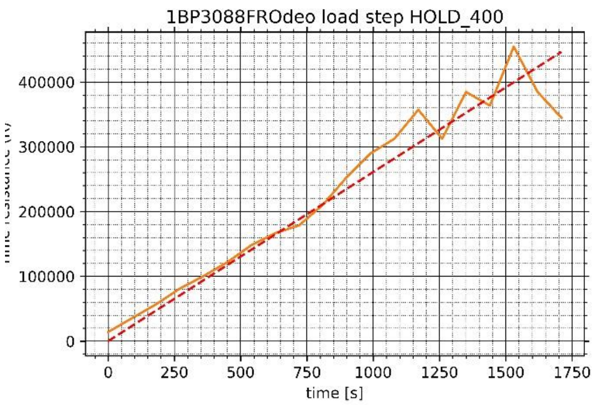


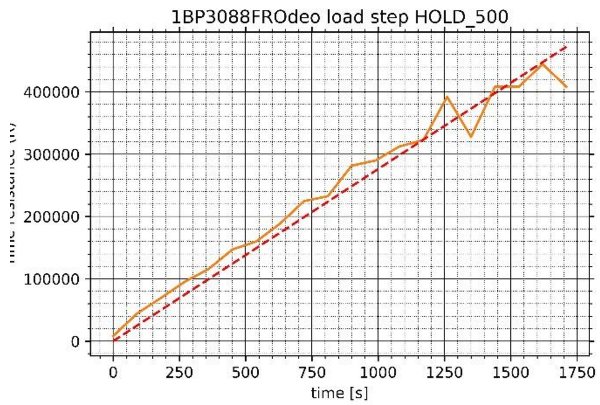




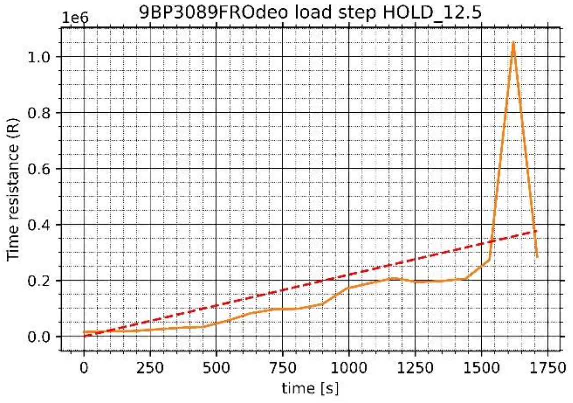


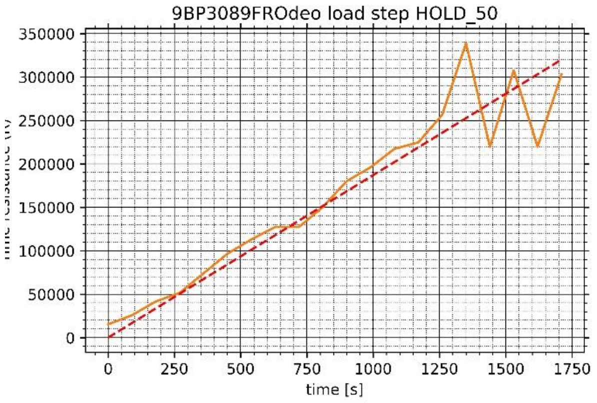


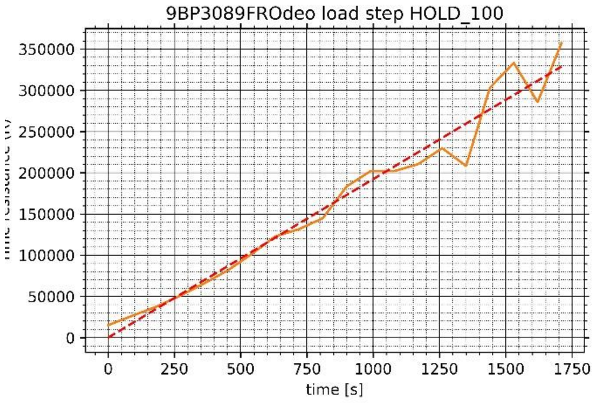


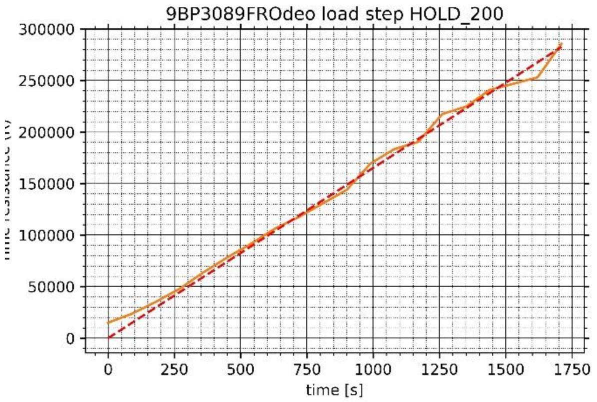


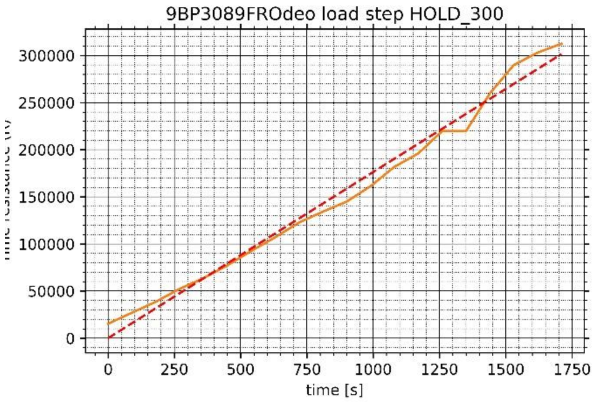
Klettelva bag samples-9BP3089FROdeo

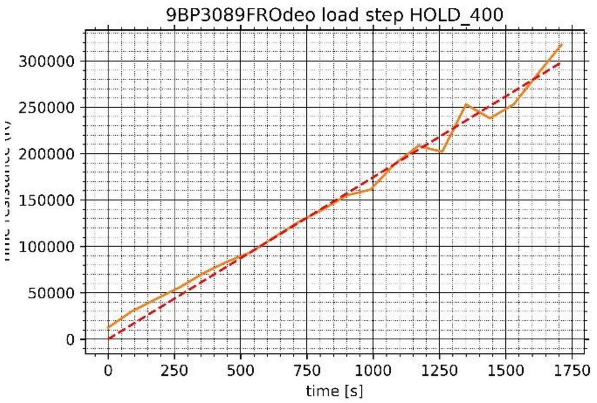


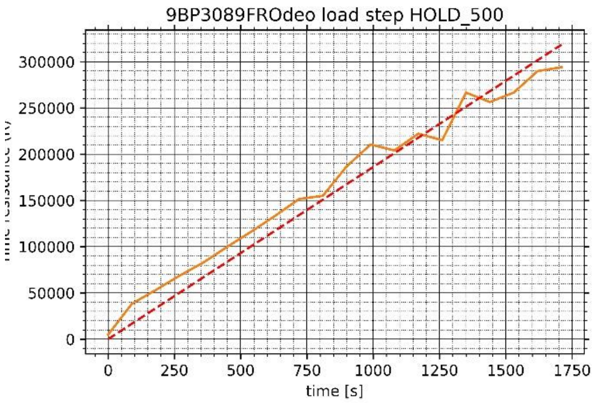




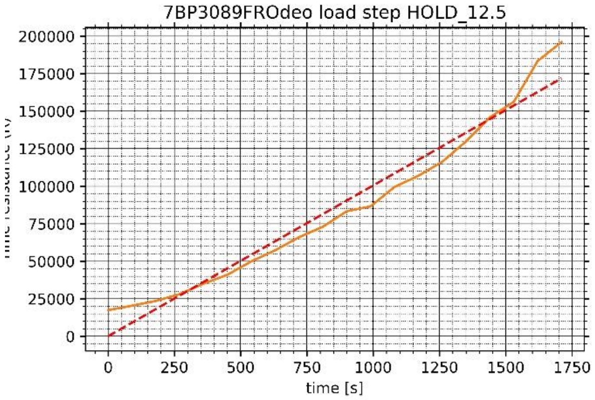


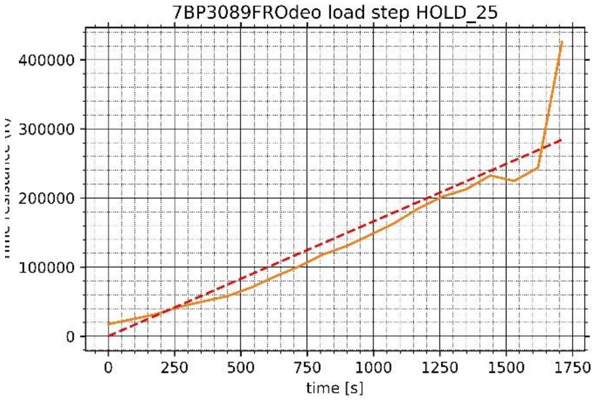


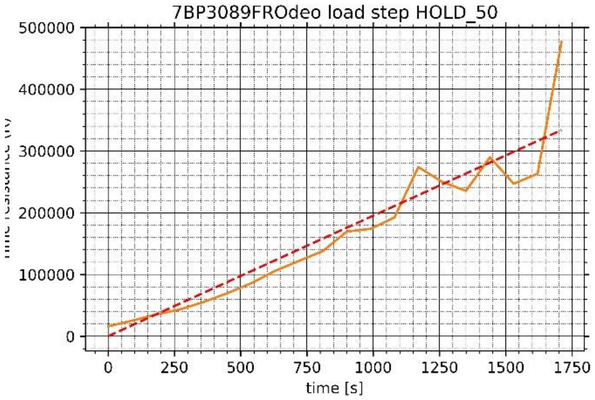


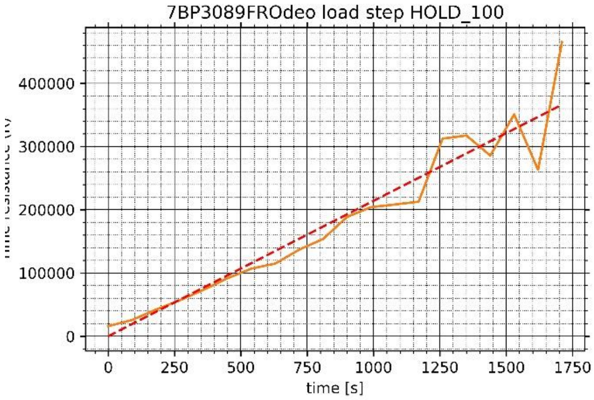


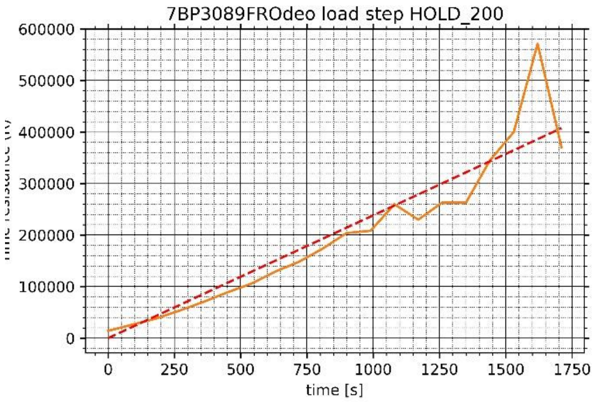
Klettelva bag samples-7BP3089FROdeo

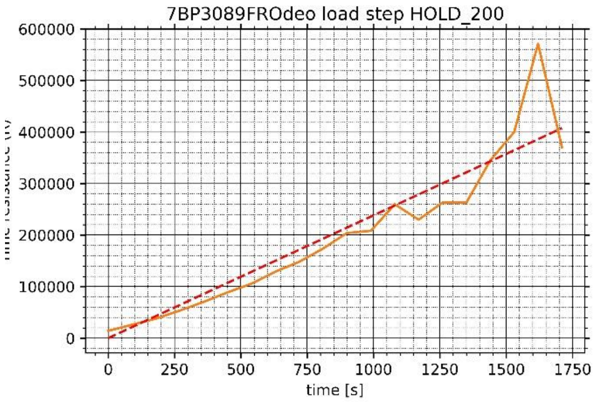


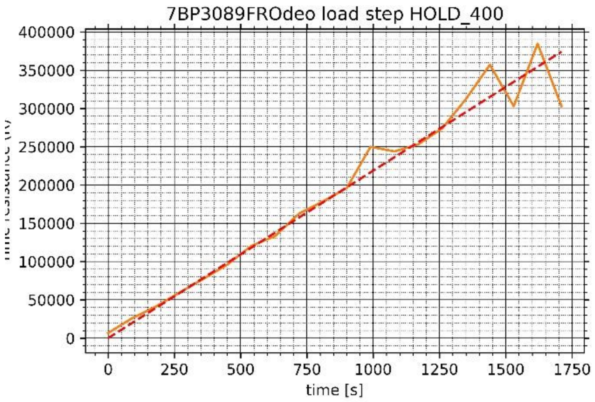


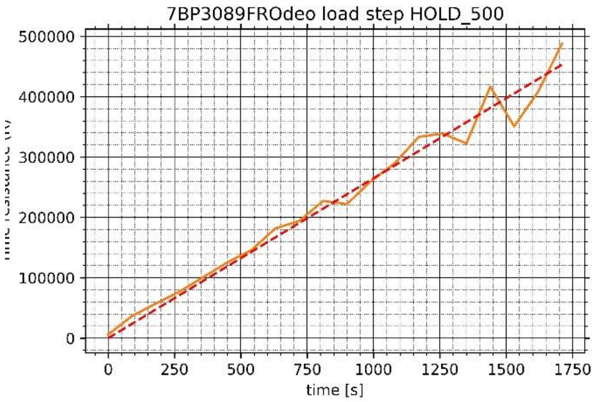




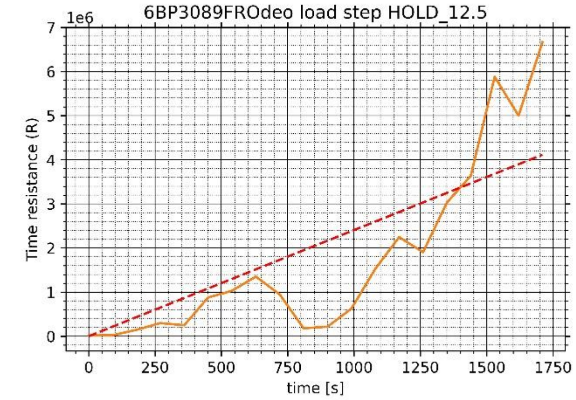


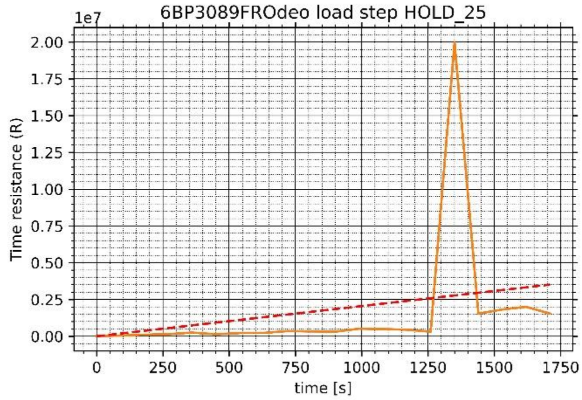


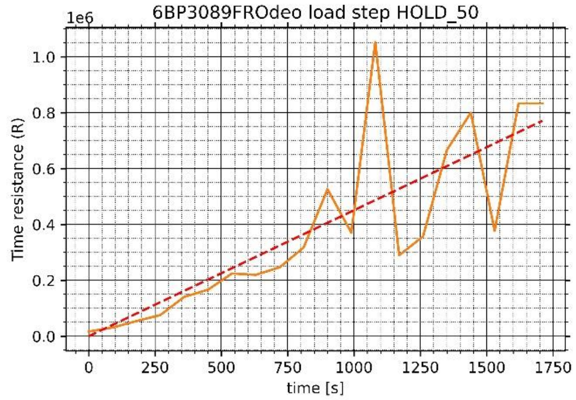


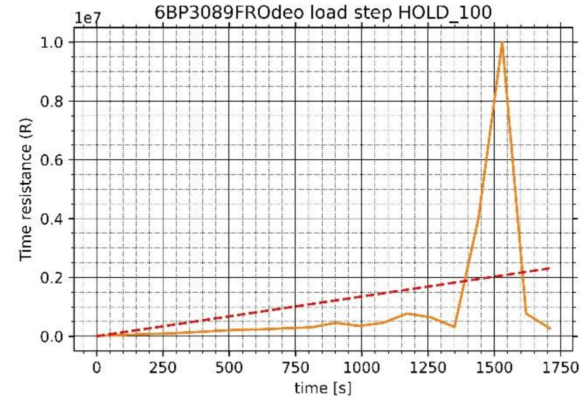


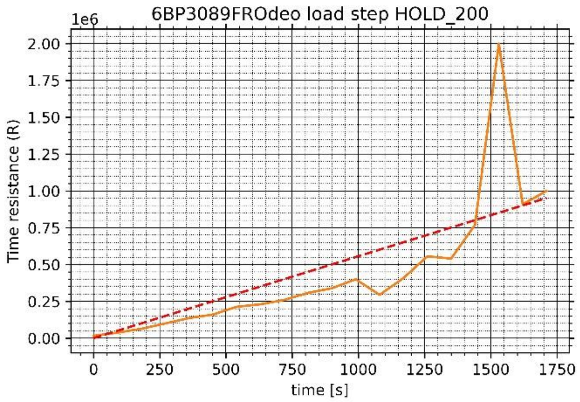
Klettelva bag samples-6BP3089FROdeo

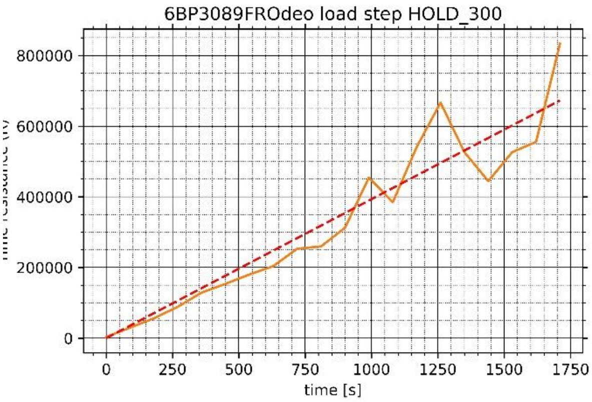


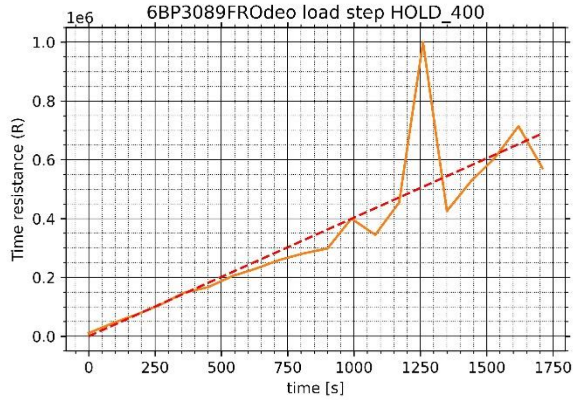


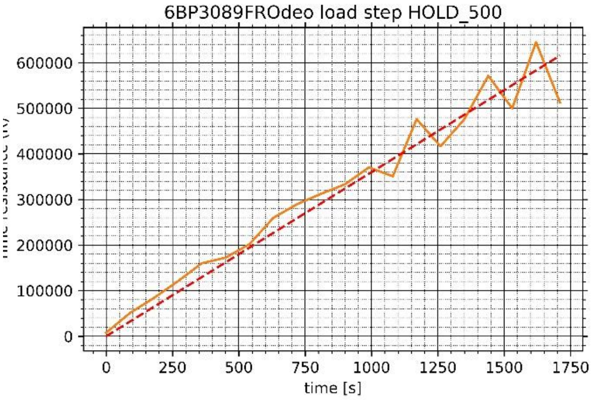




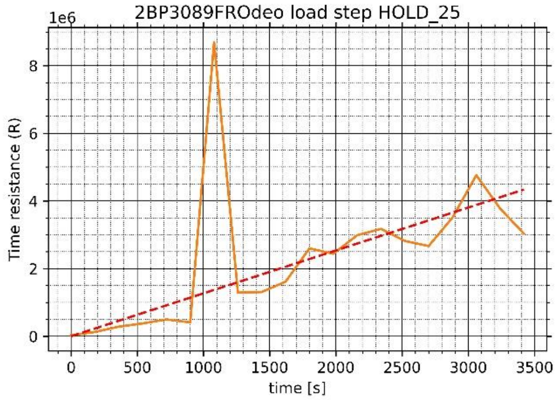


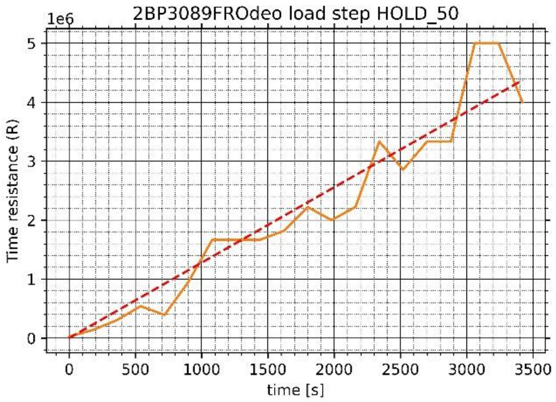


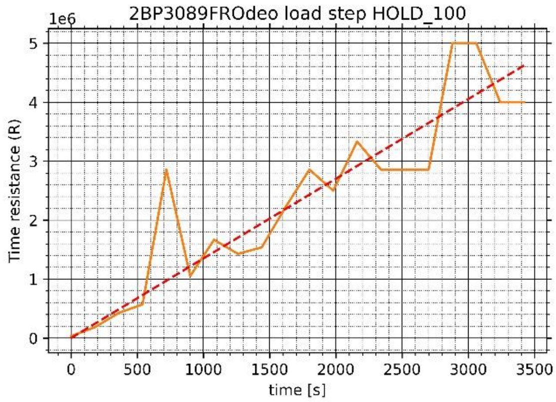


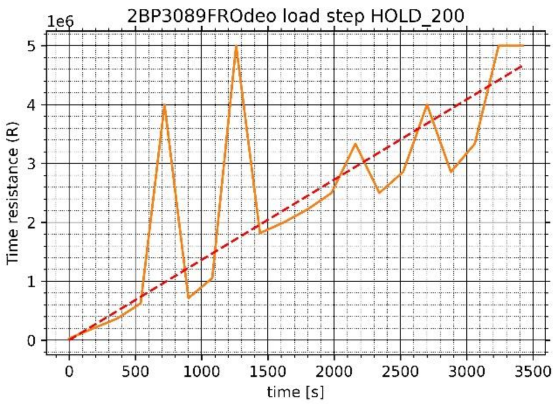


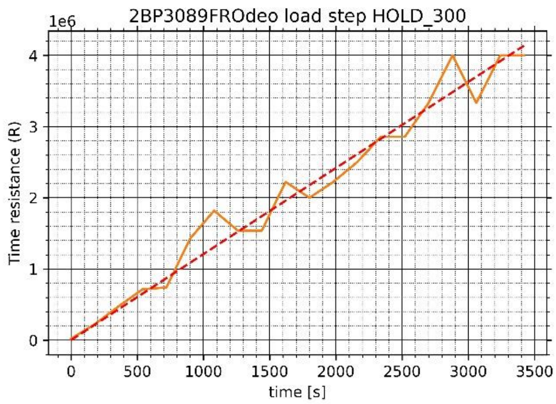
Klettelva bag samples-2BP3089FROdeo

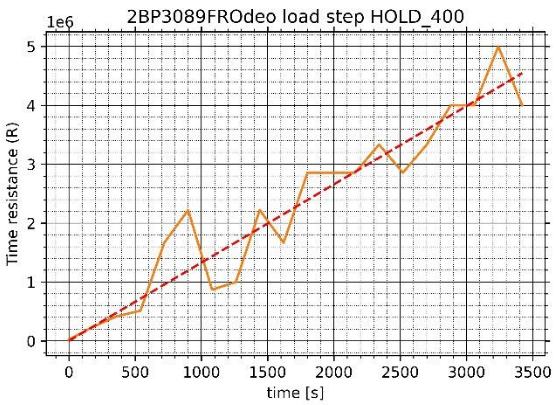


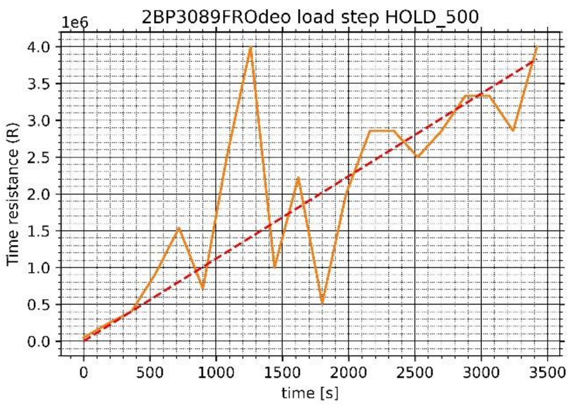




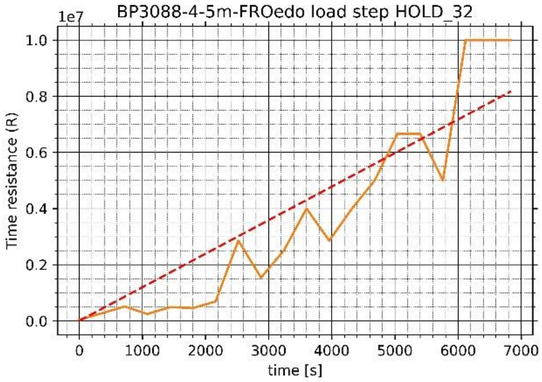


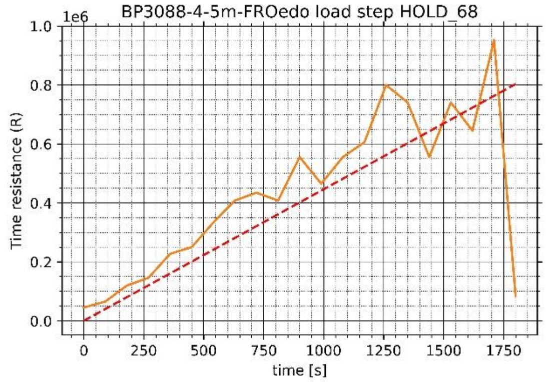


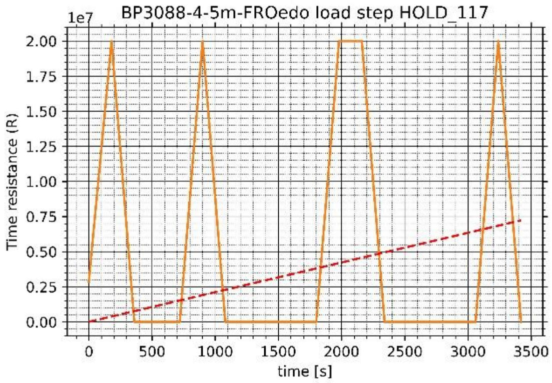


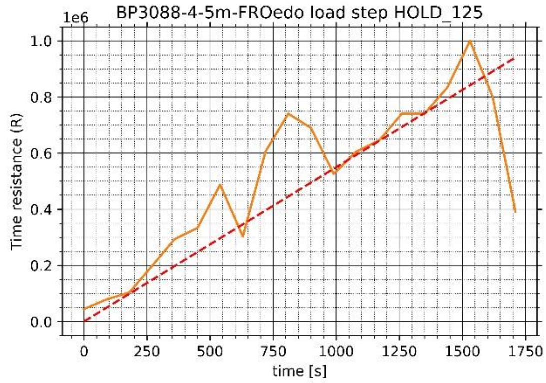


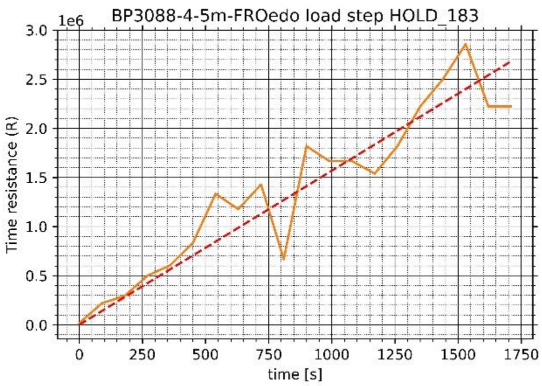
Klettelva cylider samples-12BP3088-4-5m-FROedo

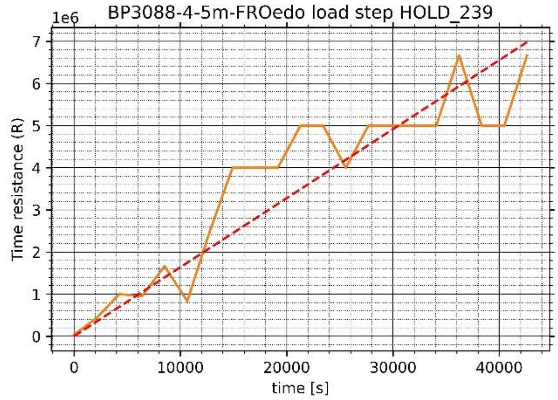




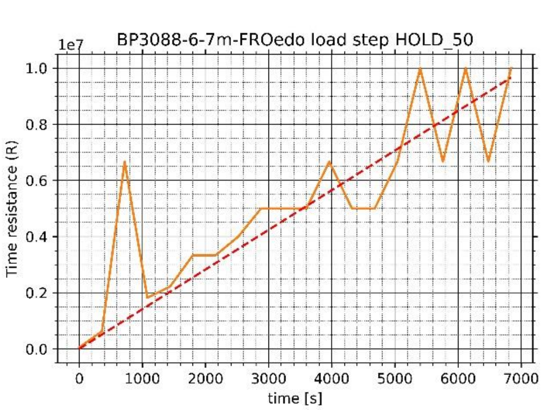


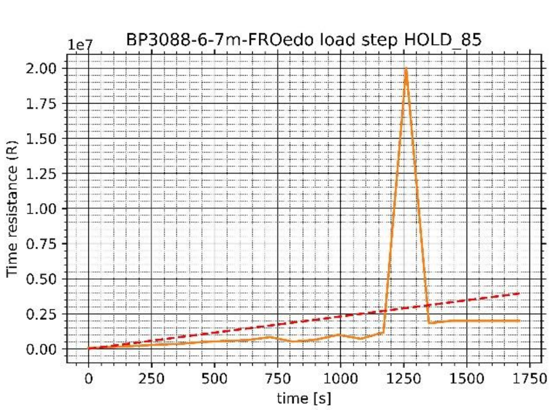


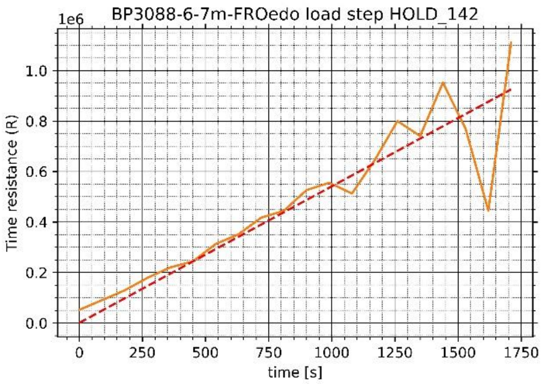


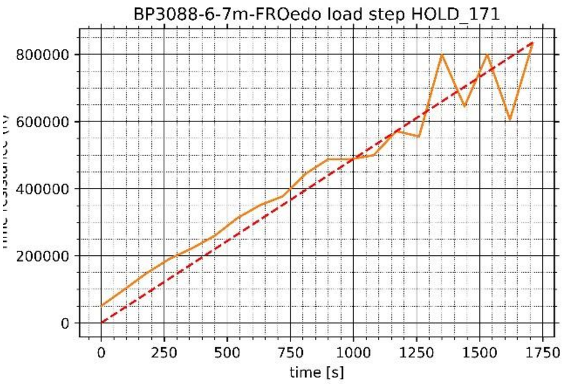


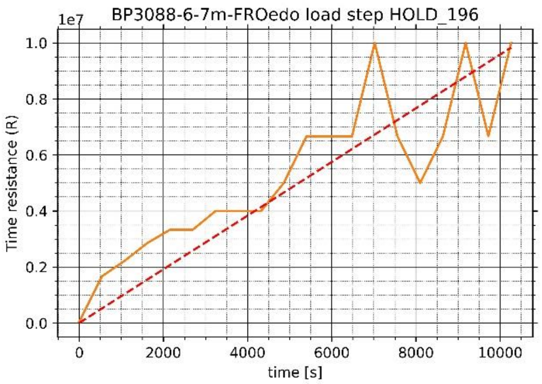
Klettelva cylider samples-4BP3088-6-7m-FROedo

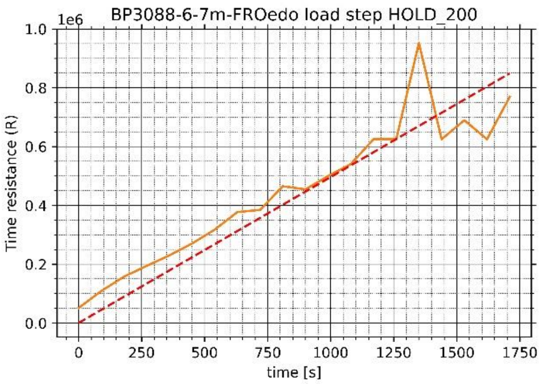




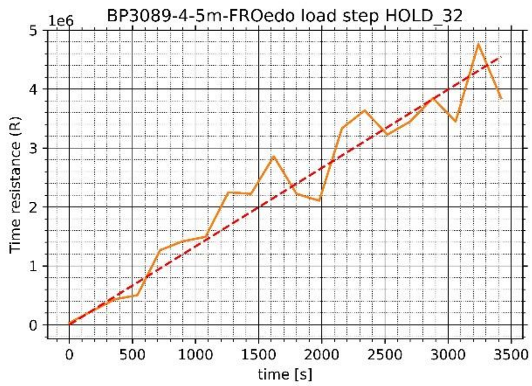


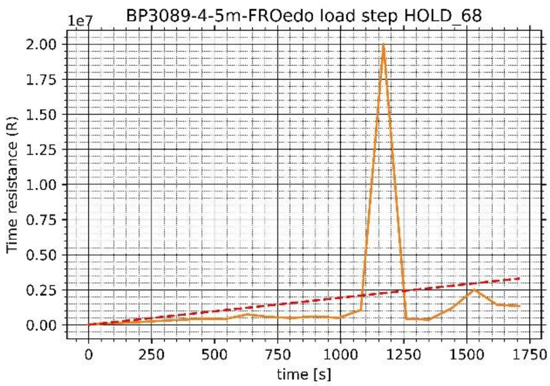


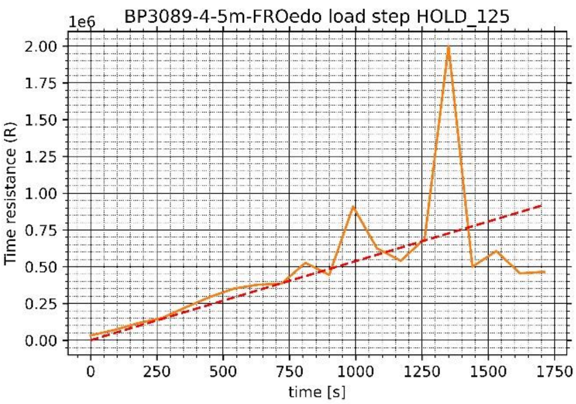


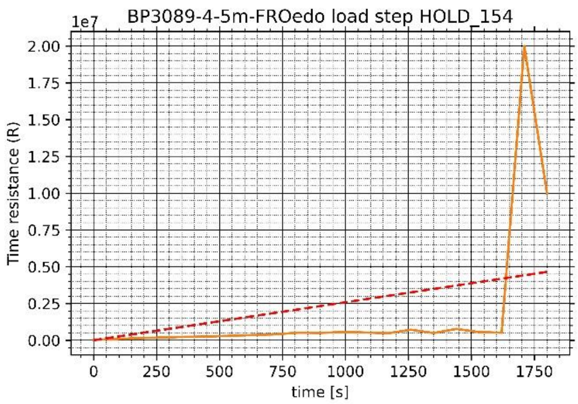


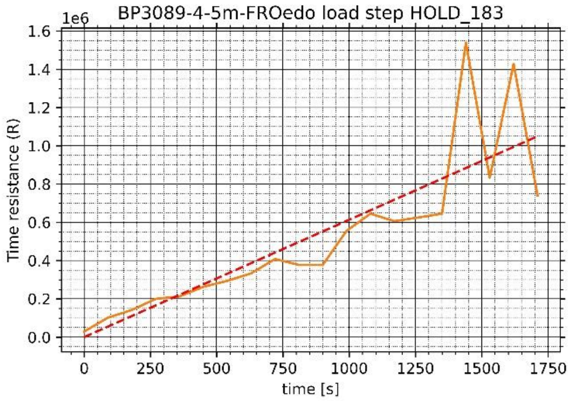
Klettelva cylider samples-7BP3089-4-5m-FROedo

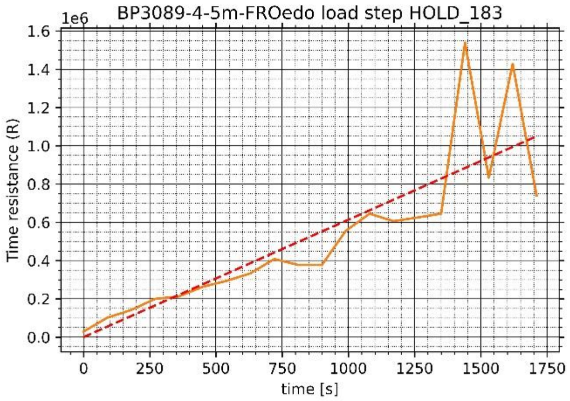












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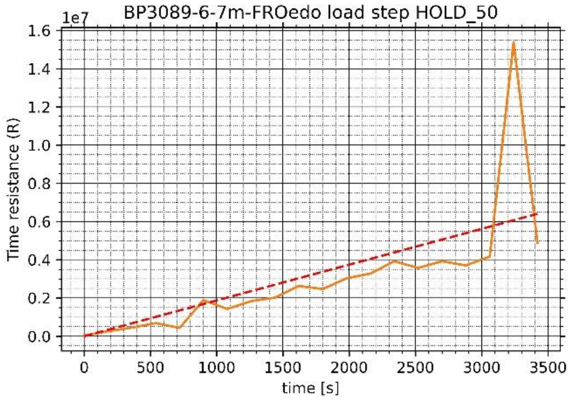

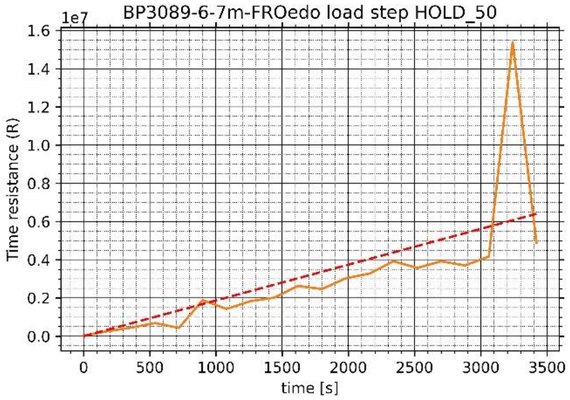

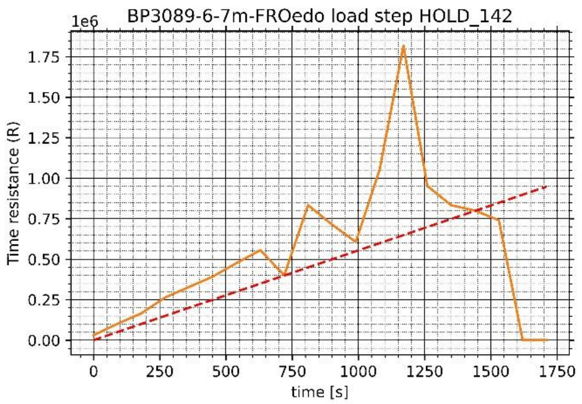

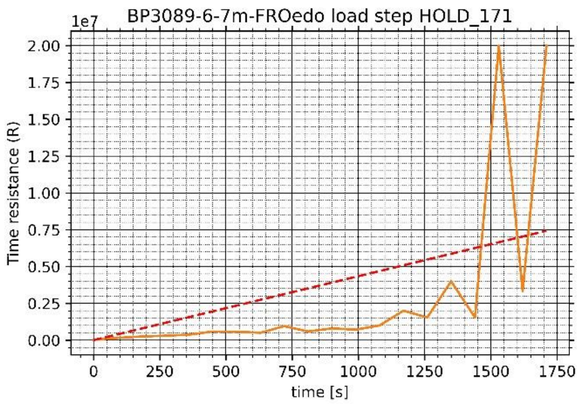

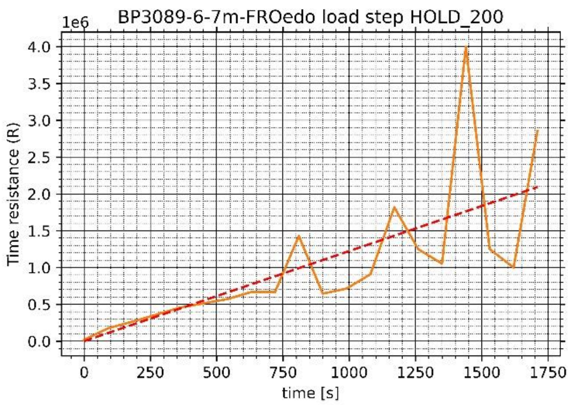

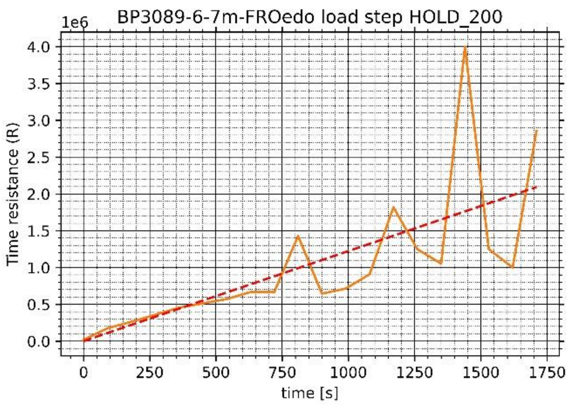

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