CHALMERS
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Settlement analysis of Göingegården test-embankment

In PLAXIS 2D and GeoSuite Settlement

Master's Thesis in the Master's Program Infrastructure and Environmental Engineering

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Göingegården test embankment by Måns Dahlström
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ABSTRACT

The aim of this report is to perform a settlement analysis for the Göingegården test embankment and assess what parameters the soil profile could have. The embankment is located close to Varberg and is standing on a 25 meter thick soil layer where clay is dominant through the profile, the monitored process have been carried out using ground peglar to measure the settlement over time. A literature study was performed to study the parameters for settlement with a special focus on secondary consolidation/creep. The embankment was then studied in GeoSuite and Plaxis 2D where the indata comes from performed field measurements, laboratory test and empiricism.

The results from the calculations software is varied and it is found it is hard to predict accurate predictions from the data sets. In the GeoSuite with Chalmers Creep model the rate of settlement is rather similar after 2 years, however the settlements is too low for the first 2 years. In the Plaxis software the total settlements has the closest value when using the Soft soil model. And with the Soft soil Creep model the settlement is about 4 times bigger than calculated with Asaokas method.

There are many uncertainties for this, especially the permeability and density of the embankment is uncertain, it can be believed that the mean permeability shall be higher through the soil profile since it is possibly a sand layer at 13 meter that shorten the drainage ways and/or the density of the embankment shall be lower.

Keywords: Settlement, PLAXIS 2D, GS settlement, Soft Soil Creep, Chalmers Creep soil model, Göingegården, Consolidation, Creep

Sättningsanalys för Göingegården provbank
I PLAXIS 2D och GeoSuite Settlment

Examensarbete inom masterprogrammet the Master's Program Infrastructure and Environmental Engineering

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SAMMANFATTNING

Syftet för denna rapport är att göra en sättningsanalys för Göingegården samt bedöma vad för parametrar jorden har i området. Göingegården provbank ligger mellan Varberg och Trönninge och står på ett 25 meter tjock jordlager där lera är dominerade genom profilen, under 4 år så har sättningarna blivit uppmäta med hjälp av mark peglar. En litteraturstudie har blivit utförd med vikt på konsolidering och krypning. Gamla samt utförda fältundersökningar och labratorie

Resultaten från sättningsberäkningar är varierade och det visar sig att det är svårt att beräkna totala sättningarna i beräkningsprogrammen, handberäkningarna verkar överensstämma väl mot uppmätta värden. Hastigheten på sättningarna verkar dock stämma väl överens i Chalmers Creep soil model, men den totala sättningen är för låg. I PLAXIS så är sättningen som uppstår efter 4 år för låg medan total sättningen efter 100 år är cirka 4 gånger större än uträknad sättning i Asaokas metod. Även hastigheten på sättningen är större än uppmätt.

På grund av att det är så stora oklarheter en känslighetsanalys blivit utförd där det framkom att permeabiliteten och vikten på provbanken har stor inverkan på resultaten. Och då undersökningar ger tecken på att det finns ett sandlager 13 meter under jordytan.

Nyckelord: Sättningar, PLAXIS 2D, GS settlement, Soft Soil Creep, Chalmers Creep Soil Model, Göingegården, Konsolidering, Krypning

Contents

Preface

This master thesis was conducted at the geotechnical department WSP Sverige in Gothenburg and Varberg between February to June in 2016.

Thanks to our supervisors Mats Karlsson, at Chalmers and the department of geotechnical and environmental engineering, and Madelene Markusson, at WSP, for there mentoring and guidance during the thesis.

Notations

In the notation table, if included, all variables occurring in the report (text, equations, figures or tables) are listed alphabetically. The variables should appear in the same format as later on in the report. Therefore, it may be wise to use the equation editor to write them, see also Section 1.5. In case of many variables, it is preferably to separate the table in “Roman upper case letters”, “Roman lower case letters”, “Greek upper case letters”, etc. Use the style “Notations” for lines with the explanations of the variables, but the style “Normal” for the table headings (and for one blank line before a new heading).

Greek lower case letters

α_s	Coefficient of secondary compression
β_k	Permeability reduction coefficient
γ	Unit weight
γ_w	Unit weight of water
ε	Strain
ε_{cr}	Creep strain
ε_v	Vertical strain
ε_{vol}	Volumetric strain
ε_z	Vertical strain
ε_z^{cr}	Vertical creep strain
ε_z^{ep}	Vertical elastic and plastic strain
$\dot{\varepsilon}$	Strain rate
$\dot{\varepsilon_{cr}}$	Creep strain rate
λ^*	Modified compression index
μ^*	Modified creep index
κ^*	Modified swelling index
σ'	Effective stress
σ'_0	Effective In-situ stress
σ'_L	Effective stress where the compression modulus start to increase
σ'_c	Preconsolidation pressure
σ'_v	Pressure vertical
$\Delta\sigma$	Pressure difference
φ	Friction angle
ψ	Dilatancy angle

Roman upper case letters

M	Compression modulus, oedometer modulus
M_0	Compression modulus when $\sigma' < \sigma'_c$
M_L	Compression modulus when $\sigma'_L > \sigma' > \sigma'_c$
M'	Modulus number
R	Time resistance
T_v	Time factor
U_v	Degree of consolidation
Q	Load

Roman lower case letters

α_0	Factor at which the improved modulus curve start to decrease linearly
α_1	Factor at which the improved modulus curve stops to decrease linearly
b_0	Factor for time resistance number r_0
b_1	Factor for time resistance number r_1
c_v	Coefficient of consolidation
c_u	Undrained shear strength
c_{uk}	Undrained shear strength (unreduced)
c_v	Coefficient of consolidation
c_{ref}	Effective cohesion
d	Drainage distance
k	Permeability
k_{int}	Initial permeability
k_z	Vertical permeability
r_0	Initial time resistance number for $\sigma' \leq b_0\sigma'_c$
r_1	Time resistance number for $\sigma' \geq b_1\sigma'_c$
r_s	Time resistance number
r	Time resistance number
s_c	Consolidation settlements
s_i	Immediate settlements
s_s	Secondary consolidation or creep settlements
s_t	Total settlements
t	Time
t_r	Reference time
t_0	Time when the R-t curve start to be linear
u	Pore pressure
u_0	In-situ pore pressure
Δu	Excess pore water pressure
z	Depth

Abbreviations

CRS	Constant Rate of Strain oedometer
CPT	Cone penetration test
FEM	Finite Element Method
GS	GeoSuite
SGI	Swedish Geotechnical Institute
SS	Soft Soil Model
SSC	Soft Soil Creep Model
IL	Increased Loading oedometer
OCR	Overconsolidation ratio

1. Introduction

To develop a good understanding of how specific a soil behaves when a load is applied, is one of the most common questions a geo engineer has to answer. Doing that needs a good understanding how a soil layer is behaving and performing, various tests can be done both in laboratory and in field. However when in really complex soil conditions, a test embankment can be built, where different parameters such as settlement and/or pore pressure is monitored to give an increased knowledge of the properties of the soil.

1.1 Background

Test embankments have been used in Sweden since at least 1945 when SGI, Swedish Geotechnical Institute, constructed two test embankments at Lilla Mellösa as a part of the investigation of where the new international airport in Stockholm should be located, this site is still a place of study for many years for geo engineers and the most recent major investigations were carried out in 2007 (Gündüz, 2008).

Göingegården is one of the future neighbourhoods of Varberg that started to get exploited in 2004. In 2012 Madelene Markusson, Geotechnical engineer at WSP, decided to construct a test embankment on the soft clay layer that exist in the area to increase the knowledge of the soil characteristics in the area.

1.2 Aim and objectives

The main aim of this project is to calculate and do a settlement analysis for the test embankment Göingegården and compare with the measured settlements. Another aim is to compare how well the settlement can be predicted by GeoSuite Settlements and PLAXIS 2D.

To achieve these goals different objectives is set to

- Perform new tests on the embankment for new and more in-data
- Produce a model in GeoSuite Settlement and PLAXIS to validate and compare results from models with the actual settlements
- Do a sensitivity analysis of the model data and examine the results.

1.3 Method

The report will contain a literature study on soil behaviour during loading, some common test methods and the ways the two scrutinised FEM programs works, and a case study on the Göingegården where during the project a field test have been done to get new samples and data by in-situ tests such as CPT and vane.

The CPT test and the rest of the field test where performed between 2016-03-14 and 2016-03-15, samples for lab tests were also collected during this time. Two different boreholes were drilled, the first one called 16W1 under the centre of the embankment and the other one 16W2, 20 meters from the foot of the embankment, see Figure 21. The CPT tests was the analysis with the CONRAD software.

At 16W1 a CPT where preformed and samples were collected, at 16W2 CPT and vane test were preformed and more samples were collected.

During the project laboratory work have been carried out, both at WSP Gothenburg geotechnical laboratory with qualified lab technicians and by the writers themselves at Chalmers geotechnical laboratory. At the WSP all the standard tests and CRS where carried out, with a strain rate of 0,7% per hour were done.

The writers have been responsible for the increased loading (IL) oedometer tests done to validate the creep parameters.

The monitored settlement data from the test embankment is carried out mostly before the project started and is included in Appendix 1.

Input data from the lab and field test have then been performed by the writers in GS and PLAXIS and thereafter sensitivity analysis have been carried out to see how different parameters change the results to see how a set percentage of change that can come from data errors and misinterpreted values can influence the results.

1.4 Limitations

The soil layer is not influenced by other loads than the test embankment.

In both PLAXIS and GeoSuite Settlement, only the vertical displacement is examined.

In PLAXIS only the Soft soil and Soft soil creep have been used when calculating the settlements.

In GeoSuite only Chalmers with and without creep have been used.

The embankment density is unknown therefore the load is uncertain.

2 Literature Study

This chapter is trying to give an overview of how settlements are calculated and the theories behind it.

2.1 History of creep research

There are several different names regarding the same phenomenon such as creep, secondary compression, time resistance and rate effects. They are all describing the time-dependent relation between effective stress and compression. Creep is defined as the decrease in volume during a constant effective stress (Holtz & Kovacs, 1981).There are different methods and theories that try to take this effect in to account in settlement calculations. One of the simplest approaches is classical consolidation theory, presented by Taylor in 1940, that can be seen as valid until all excess pore pressures have dissipated, after that creep begins. Creep is modelled as a linear function versus the logarithm of time with a continuously decreasing rate. Classical consolidations theory has been shown to be valid for results from small oedometer test but not so well with long term measurements in field (Larsson, Bengtsson, & Eriksson, 1997).

In 1954 another model was presented that didn't separate the change in pore pressure in to different stages, more than taking hydrodynamic delay for the first rapid phases, this model was presented by Suklje. It describes the relation between effective stress and compression changes in comparison to the rate of deformation (Larsson, Bengtsson, & Eriksson, 1997).

The model also describes what is called "secondary consolidation" but not as a separate phenomenon that comes after the "primary consolidation" it describes it as one process that is stress-deformation which is time dependent. This process is time dependent and controlled by the permeability of the soil and drainage paths, as water from the soil pores has to flow out from the soil for settlements to occur (Larsson, Bengtsson, & Eriksson, 1997).

In 1967 and 1972 Bjerrum presented another geotechnical model illustrating the time effect on settlements over long term including that creep and primary consolidation occurs simultaneously (Claesson, 2003).The model explains effects from overconsolidation in natural soils because of geological "ageing" it also explain why settlements can occur even when preconsolidation pressure isn't surpassed as seen in Figure 8. The model illustrates the processes in the soil that continue for a very long time and the influence of the hydrodynamic delay in the settlement process. Bjerrum pointed out that permeability and drainage paths has to be taken into account in normal predictions of settlements. Bjerrum also describes time-dependent increase in shear strength, in relation to undrained shear strength. His models have been shown to correlate well with data collected both in the field and in laboratory (Larsson, Bengtsson, & Eriksson, 1997).

In modern geotechnical engineering more and more advanced models, based on finite element method (FEM), such as Soft Soil Creep (SCC) and Chalmers with creep are used. (Olsson, 2010).

2.2 Empirical knowledge

In order to make accurate settlement calculations a representative model is required that describes the processes that occurs in the soil during consolidation apart from the vertical compression. Field and laboratory tests are only carried out on a fraction of the total soil profile; empirical knowledge is used to correlate different parameters in order to estimate more parameters than tested. In order to make more accurate predictions and gain more understanding of the soil processes a lot of different field studies have been done studying for example, pore pressure build-up during loading and dissipation afterwards, elastic deformations du to loading and horizontal movements, how settlements change with depth and shear strength increase has been studied, this is just some of the processes that have been studied and closely monitored in field test. Empirical knowledge is also gathered in laboratory tests where compression and swelling and the time dependence of these characteristics, the relationship between compression and permeability are some of the characteristics that are being studied. Combining observations made in the field with lab results makes it possible to create methods for translating laboratory tests to the conditions that apply in the field. As more data is gathered new test methods and equipment constantly developed (Larsson, Bengtsson, & Eriksson, 1997).

2.3 Knowledge of the soil profile

When making settlement predictions detailed knowledge of the soil profile, its characteristics, different layers, in situ stresses and pore pressures all containing natural variations, is important to be able to make accurate calculations and predictions. These parameters can be evaluated for soft soils, such as clays and other low permeability soils, using an oedometer. According to (Larsson, Bengtsson, & Eriksson, 1997) oedometer test are not practical to use since the consolidation process is slow in this type of soils, especially IL oedometer therefore CRS oedometer test are preferred (Larsson, Bengtsson, & Eriksson, 1997).

2.3.1 Field tests

There are many different field tests that can be carried out but this chapter will focus on the commonly used in Sweden. One of the simplest ways to test a soil in field is to use a pressure sounding device where the pressure used to drive the head of the rod down in the soil is recorded. (Sällfors, 2009)

Another often used method in Sweden is cone penetration test where the pressure at the tip of the cone is recorded together with the friction along the sleeve of the cone and the pore pressure when it is driven down at a pace of about 2 cm/s. All this together can give continuous information of the point pressure, skin resistance and pore pressure in the soil layer and with this data soil type and some information of strength of soil can be evaluated. (Sällfors, 2009)

In the field also a vane apparatus can be driven down into the soil to calculate the uncorrected undrained shear strength by turning the vane and register the torque needed, this test can be done with a casing or without one along the rod, and for the later one a clutch is used to separate the friction force along the rod with the one from the vane. (Jonsson & Sellin, 2012)

2.3.2 Lab tests

The lab tests are done on soil samples that have been retrieved in field by either a piston sampler that gives undisturbed samples or by an auger sampler that gives a disturbed sample. This part will also address some of the most common ways by Swedish practice to test soils.

One of the most used test is the "rutinundersökning" which is a standardized test procedure where density, together with shear strength, sensitivity and liquid limit is determined by calculating the weight and using cone tests.

Oedometer tests are frequently used. There are two different variants. The most common one is the constant rate of strain test also called CRS; here the sample is enclosed in a small metallic container and an increasing load is applied. Preconsolidation pressure and other stress parameters can be derived. The other way an oedometer test can be set up is the increase loading variant, IL; at a set time step typical a day additional load is applied or removed. From this the creep parameter can be scrutinised. (Sällfors, 2009)

Triaxial tests is becoming more and more common and often needed for more advanced soil models. A quite large sample, height to width ratio of 2, is covered with a rubber membrane and placed in a liquid cell where the pressure can be increased isotropic. When the sample is in equilibrium with the environment the second part of the test can start where a vertical load is applied to the sample to until it fails by shearing. (Sällfors, Geoteknik, 2009)

For more information on methods for investigation in field and in lab the writers recommend Jords Egenskaper by Rolf Larsson.

2.4 Settlements and consolidation theory

In laboratory small specimens of soil are tested and the results then are translated using empirical knowledge in to in-situ condition in the settlement predictions. To be able to transfer these results in-to representative soil models different assumptions needs to be considered.

2.4.1 Classical consolidations theory

One of the first to set up some assumptions of soil behaviour was Terzagi in 1923. He developed a mathematical model to determine of the degree of consolidation for a certain time which is the foundation of all modern theories today (Olsson, 2010). The model is based on the following assumptions:

1. The soil is homogenous.
2. The soil is fully saturated.
3. The solid particles and water are incompressible.
4. Compression and flow are one-dimensional.
5. Strains are small.
6. Darcy's law is valid at all hydraulic gradients.
7. The coefficient of permeability and the coefficient of volume compressibility remain constant throughout the process.

8. There is a unique relationship, independent of time, between void ratio and effective stress.

The process of consolidation can be described with similarities to a water filled cylinder with a piston connected to a spring; this model describes the consolidation process of saturated soil (Holtz & Kovacs, 1981), see Figure 1.

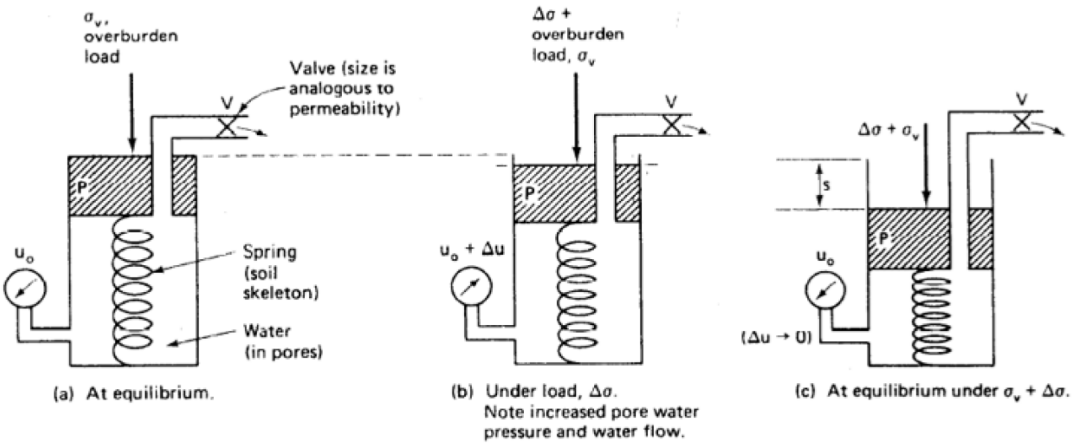

Figure 1. The spring analogy applied to the consolidation process according to (Holtz & Kovacs, 1981)

The soil skeleton is represented by a spring and the pressure applied to it simulates the effective stress in the soil. The open valve on the top simulates the pore size and permeability of the soil that controls the pore water pressure.

When there is no water flowing out from the open valve the system is in a steady state, see Figure 1a.

The pore water pressure increases, and water starts flowing out of the valve, as soon as additional load is applied, see Figure 1b.

When water flows out through the valve the load is transferred from the water to the spring, symbolizing soil settlements. The system will eventually reach steady state and the outflow of water will stop, see Figure 1c,

Long-term measuring have shown that Terzagis first assumptions are not fully accurate and that the actual settlements and pore pressures don't completely follow the classical consolidation theory. Data from measuring and field tests have shown that both the size of settlements and the time required differs but when taking change in modulus into account in relation to increased stress levels and changes in permeability during the consolidation phase calculations have been more accurate according to (Larsson, Bengtsson, & Eriksson, 1997).

2.4.2 Mathematical model

The process of consolidation can be described with equation 1.

$$\frac{\partial u}{\partial t} = \frac{M}{\gamma_w} \cdot \frac{\partial}{\partial z}\left(k \cdot \frac{\partial u}{\partial z}\right) \qquad (1)$$

Where: u= pore pressure [kPa]
t= time [s]
M = oedometer modulus [kPa]
γ_w= unit weight of water [kN/m]
k= permeability [m/s]
z= depth [m]

Since the permeability is assumed to be constant with depth, see 2.4.1, equation 2 can also be written as

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad (2)$$

Where $c_v = \frac{M \cdot k}{\gamma_w}$ [m^2/s]

c_v Is defined as the coefficient of consolidation, which determines the rate of the consolidation process. Since k, M and $\gamma_w$are assumed to be constant c_v is constant during the consolidation process (Knappett & Craig, 2012). Finding solutions to differential equations is complicated. Something called time factor,T_v is therefore often used when calculating consolidation in order to simply the calculations.

$$T_v = \frac{c_v \cdot t}{d^2} \qquad (3)$$

Where: t = time [s]
d= drainage distance [m]

When T_v is known, the average degree of consolidation, U_v, can be evaluated using Figure 2. The graph contains three different lines that represent different relations between U_v and T_v depending on initial variations of excess pore water pressures due to the load that is applied, Figure 2 (Knappett & Craig, 2012).

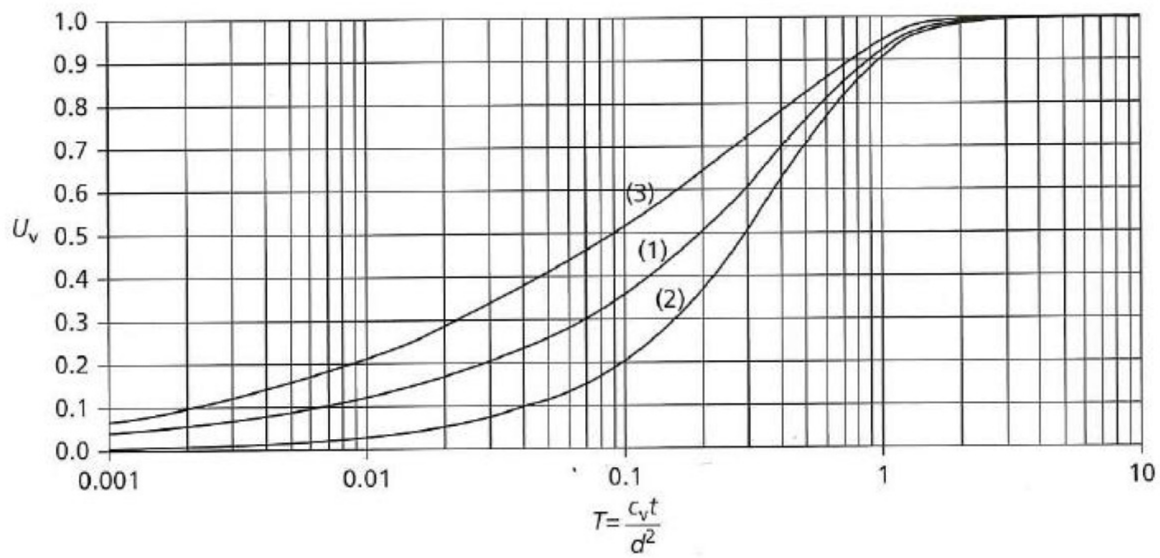

Figure 2 Relationships between average degree of consolidation and time factor for different initial variations of excess pore water pressures. (Larsson, Jords Egenskaper, 2008)

2.4.3 Creep effects

The consolidation process is often divided into two phases, primary and secondary consolidation, see Figure 3 (Olsson, 2010). Primary consolidation phase describes the dissipation of excess pore water pressure, see 2.4.1. The secondary consolidation phase begins when most of the water has dissipated and the load bearing capacity is transferred to the soil structure, see Figure 3. During the secondary consolidation phase compression is mostly controlled by creep strains. The two different phases can be plotted in a graph with log-time plotted against the strain, ε, or void ratio, e, see Figure 3.

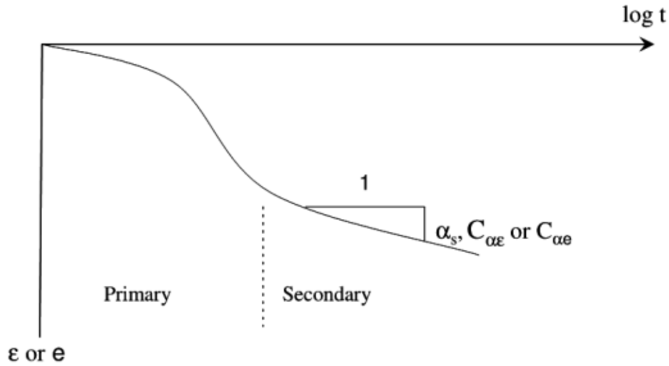

Figure 3. Consolidation curve, showing the primary and secondary consolidation phases (Olsson, 2010).

During the creep process the soil reconstructs due to strains, bonds between particles within the soil making it more compact. Compression due to creep occurs at a much lower rate than primary consolidation that occurs due to consolidation. There are no exact ways of separation between the different phases. Older consolidation models divide primary and secondary consolidation in to two separate phases. The more modern theories and models used today, takes creep effect in to consideration both in primary and secondary consolidation, see 2.1, (Claesson, 2003).

Two different situations is shown in Figure 4. The first is when effective stress and compression is increased over time due to dissipation of excess pore water pressure, and in the second excess pore water pressure is ignored and the effective stresses are transferred to the clay structure. The dashed line in the graph represents the second situation and the solid line represents the first, where the excess pore water pressure dissipates. The graph shows that even if there is no excess pore water pressure that dissipates in the soil settlements can still occur in form of creep (Claesson, 2003)

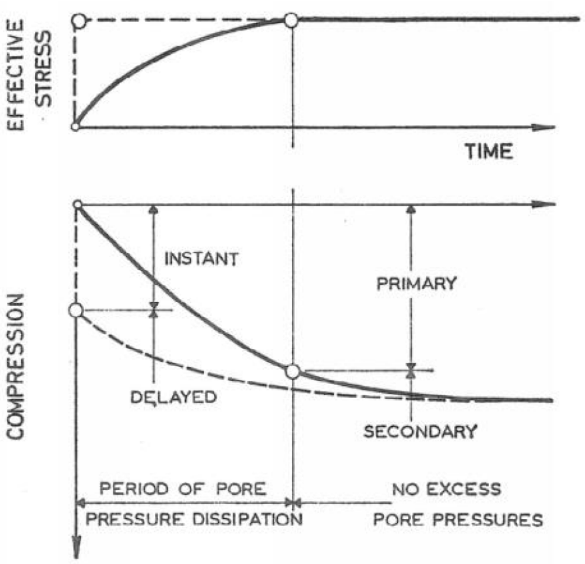

Figure 4. Settlement or compression illustrated by the Bjerrum model describing instant and delayed compression. The dashed line represents compression and effective stress if the pore water pressure could be disregarded. The solid line represents the effective stress and compression when excess pore water pressure dissipates over time (Claesson, 2003).

2.5 Soil parameters important to long-term settlements

The consolidation process of soil is complex and is controlled by several soil parameters that affect the settlement and consolidation process. The following section will explain some of the major processes and parameters.

2.5.1 Soil stresses

The stress history is important to determine if the soil is over- or normal consolidated. The maximum stress level that a soil has been exposed for is called the preconsolidation pressure, σ_c' (Larsson, 2008) and can be determine by using an oedometer test with help of a method presented by Sällfors in 1975 and described in *Figure 5*.

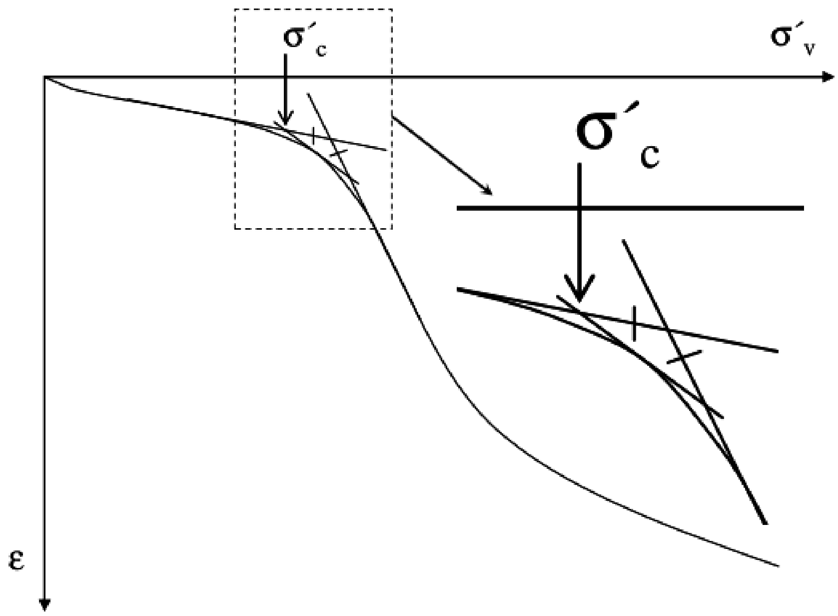

Figure 5 Determination of preconsolidation pressure from oedometer tests (Olsson, 2010).

The compression modulus, M, is the derivative of the curve that is evaluated from oedometer tests. *Figure 5* shows the typical stress-strain relationship for clay. The modulus changes depending on stress level (Larsson, 2008). Compression modulus is determined using equation 4 (Sällfors, 2009).

$$M = \frac{\partial \sigma'_v}{\partial \varepsilon_v} \qquad (4)$$

The modulus is considered to be constant when the effective stress is below preconsolidation pressure, and is written as M_0. The modulus is also constant between the preconsolidation pressure σ'_C and the point where the compression modulus starts to increase is called σ'_L. In this interval, the modulus is written as M_L. For effective stresses greater than σ'_L the modulus is increasing and can be calculated with equation 5.

$$M = M_L + M'(\sigma` - \sigma`_L) \quad (5)$$

Where M' is the modulus number, see Figure 6.

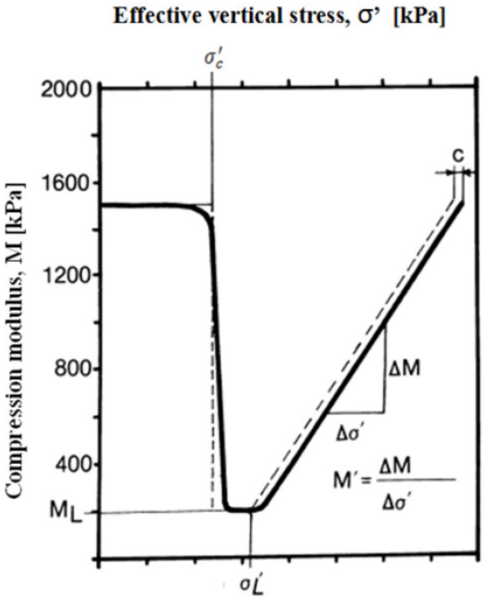

Figure 6 Illustration of the compression modulus at different stress levels (Larsson, 2008).

When plotting the compression modulus for different effective stress levels is can be evaluated, se Figure 6. The compression modulus values obtained from CRS test are generally low and empirical values are used in order to mimic the conditions and soil behaviour. One of these empirical values used are equation 6, where k can vary from 150 to 1000, for Gothenburg clays the value 250 is often used (Larsson, Jords Egenskaper, 2008).

$$M_0 = k \cdot C_u \quad (6)$$

When newly deposited clay is exposed to a stress increase, it consolidates as a normal consolidated soil that follows the Virgin compression line (A-B-C-E-F in Figure 7), this compression is often called the plastic deformation of the soil. The section between the C-D-E points shows the reaction when unloading and reloading the soil sample and in particular the part D-E is elastic behaviour of the clay sample. At E it reaches the preconsolidation pressure again and start deforming plastic again where the strain is higher. (Sällfors, 2009) Therefore it really important to understand where a soil preconsolidation pressure is.

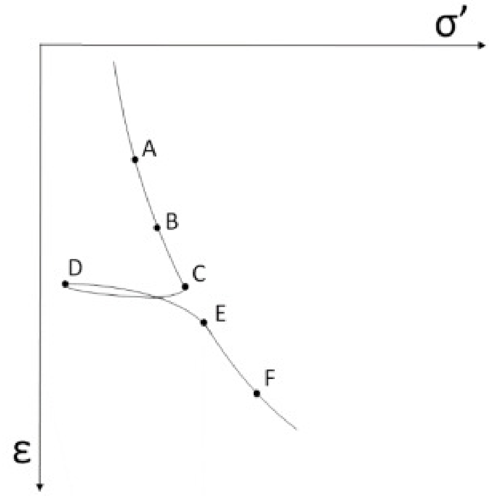

Figure 7. How stress and strain is related to each other under loading of a NC clay sample (A-C), unloading (C-D) and reloading (D-F) (Bergström & Modin, 2015)

The relation between preconsolidation pressure and the in situ stress level σ_0' of the soil is called overconsolidation ratio, OCR and is calculated by equation 7 (Olsson, 2010) and the different grades of OCR is presented in Table 1.

$$OCR = \frac{\sigma_c'}{\sigma_0'} \qquad (7)$$

Table 1. Different OCR ratios (Larsson, 2008).

Grade	OCR
Normalconsoilidated to lightly overconsolidated soils	1 – 1,5
Overconsoildated soils	1,5 – 10
Strongly overconsolidated soils	>10

Normally consolidated soils are when the in-situ stress is the same as the preconsolidation pressure so that the OCR equals 1 (Sällfors, 2009). When a soil is first deposited it will undergo some instant compression and later on it undergoes an ageing effect described by (Bjerum, 1967) as seen in Figure 8.

The normal way of computing the σ_c' in Swedish practice as shown in *Figure 5* gives a OCR that is higher than the actual value, for an perfectly normal consolidated clay where the OCR should be 1 the actual value become 1,3. (Olsson, 2010)

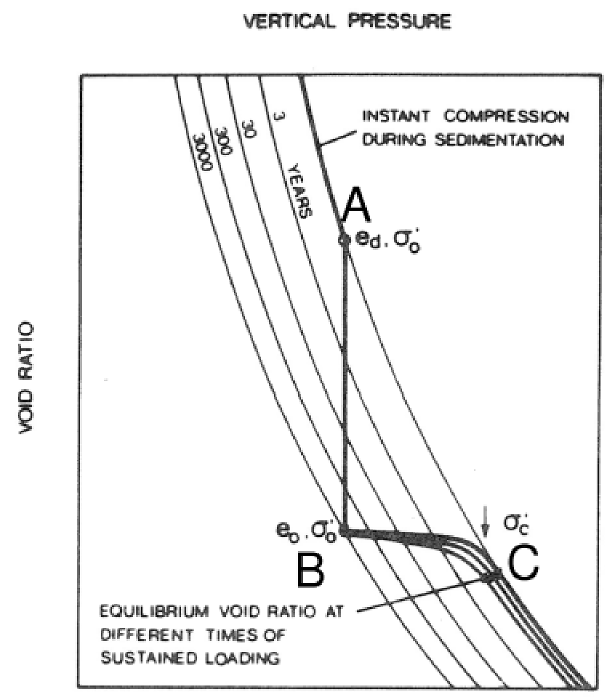

Figure 8. The effect of ageing/secondary compression on preconsolidation pressure and void ratio. (Olsson, 2010).

2.5.2 Permeability

The permeability of a soil describes how easily water can be transported though the soil. The parameter is a function of void ratio and its relative compression, since it is dependent on the pore-size and total pore-volume (Larsson, 2008).The permeability is evaluated from CRS test as seen in Figure 9 where k_i = initial permeability β_k= -Δlogk/$\Delta\varepsilon$ (Larsson, Bengtsson, & Eriksson, 1997).

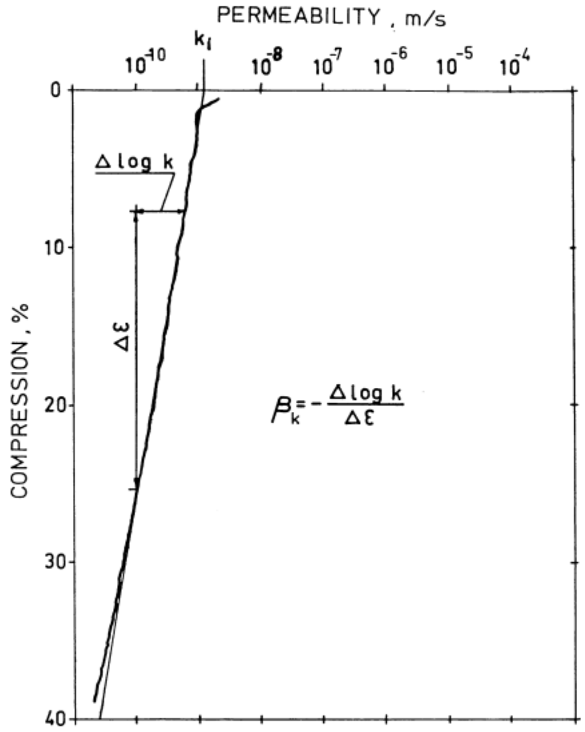

Figure 9 The log-based strain permeability model (Larsson, Bengtsson, & Eriksson, 1997).

This figure is valid for homogeneous soft clays, difference in horizontal and vertical permeability are very small and are considered to be equal. Most clays cannot be considered fully homogenous and needs to be divided in to many different layers with different properties. Clays that have been subjected to very high vertical stresses could have difference in permeability between the horizontal and vertical permeability (Larsson, Bengtsson, & Eriksson, 1997).

2.5.3 Coefficient of secondary compression

The soils creep characteristics is described as the coefficient of secondary compression (Larsson, Bengtsson, & Eriksson, 1997).This parameter is widely used in Sweden as a part of the Chalmers model and is defined as equation 8

$$\alpha_s = \frac{\Delta\varepsilon_{cr}}{\Delta\log(t)} \qquad (8)$$

Where α_s = coefficient of secondary compression [s-1]
$\Delta\varepsilon_{cr}$=creep strain [-]
t= time [s]

Laboratory investigations and empirical knowledge show that the coefficient of secondary compression is very low in clay until around $0{,}8 \cdot \sigma_c'$ and then reaches its maximum when the effective stress is equal to the preconsolidation pressure, after that the coefficient starts to decrease se Figure 10.

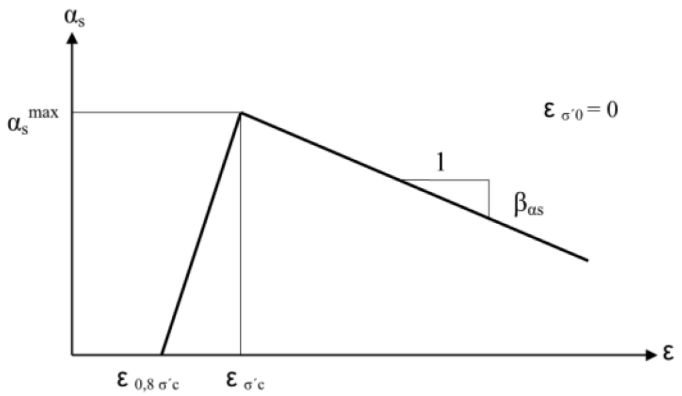

Figure 10 Relationship between the coefficients of secondary compression and strain (Claesson, 2003).

2.5.4 Time resistance

The creep characteristics of the soil can be described using time resistance (Claesson, 2003). The term time was first introduced in 1969 by Janbu that presented the following equation:

$$R = \frac{\partial t}{\partial \varepsilon} \qquad (9)$$

Where R=time resistance [s]

t=time [s]

ε =strain [-]

Laboratory test show that the time resistance increases linear with time and can therefore be rewritten as:

$$r_s = \frac{\partial R}{\partial t} \qquad (10)$$

Where r_s is the time resistance number.

Figure 11 demonstrates that the time resistance is increasing linearly only after a certain time, t_0, which makes it possible to describe using equation 11, It should be noted that is only valid after time t_0.

$$R = r_s \times (t - t_r) \qquad (11)$$

Where t_r is the reference time for the idealised curve shown in Figure 11.

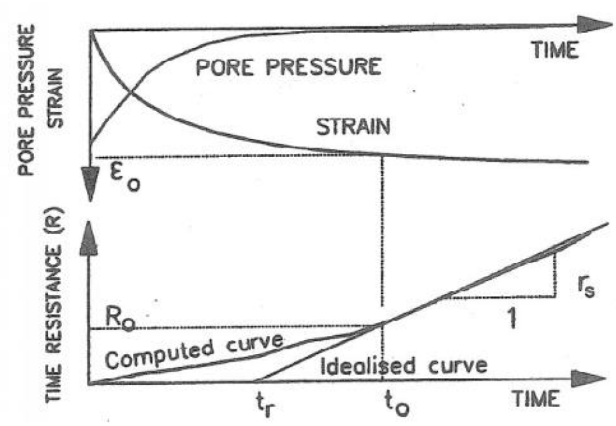

Figure 11 The relationship between time resistance and time (Claesson, 2003).

For a more general formula that can be used to calculate the time resistance at any given time equation 12 can be used:

$$\dot{\varepsilon_{cr}} = \frac{\partial \varepsilon_{cr}}{\partial t} = \frac{1}{R} = \frac{1}{r_s \cdot (t - t_r)} \qquad (12)$$

When integrating equation 12 from t_0 to t, the creep strains can be calculated using equation 13:

$$\Delta\varepsilon_{cr} = \frac{1}{r_s} \int_{t_0}^{t} \frac{\partial t}{(t - t_r)} = \frac{1}{r_s} ln \frac{(t - t_r)}{(t_0 - t_r)} \qquad (13)$$

Previous equation can be rewritten as:

$$\frac{1}{r_s} = \frac{\partial \varepsilon_{cr}}{\partial ln(t)}$$

Secondary compression can be calculated using time resistance number when combining equation 12 and 13, see equation 14.

$$\propto_s = \frac{ln10}{r_s} \qquad (14)$$

3 Settlement calculations

There are several different methods on how to calculate the strain on a soil, in this chapter the ways the GeoSuite Settlement, Plaxis 2D, Ashokas and hyperbolic method to calculate the settlement is presented.

GeoSuite settlement and PLAXIS have been chosen since they are popular programs that is often used in the industry in Sweden to calculate settlements.

3.1 Geosuite description

GeoSuit Toolbox includes different programs for calculations in slope stability, sheet walls, pile groups and settlement calculations which can be presented according to national standards (Vianova GeoSuite AB, 2013).

GeoSuite Settlement (GS Settlement) is used for calculations of time dependent settlements due to loads and boundary conditions that vary over time (Vianova GeoSuite AB, 2013). The program uses GEOnac (Geotechnical nonlinear analysis code) which is a general finite element program that makes one-dimensional and uniaxial calculations for deformations and assumes vertical pore water flow (Vianova Systems AS). The program is based on one dimension calculations and in order to make three dimensional calculations it calculates different points in one dimension and then the settlements are interpolated between them.
GS Settlement is commonly used in engineering practice and makes it possible to take creep effects into account through different settlement models and it is a quite simple program to use. The models included in GS Settlement are Janbu´s model, Krykon and Chalmers model. Creep effects are taken into account when using Krykon or Chalmers model. Permeability models are also included in the program, such as the CV based, the exponential and the log-based (strain and void) models (Vianova Systems AS). In this thesis the log-bases strain permeability is used, se Figure 9.

3.1.1 Chalmers Creep soil model

The Chalmers creep model is a soil model designed for fine grained soils such as clay that includes creep when calculating settlements (Olsson, 2010).The model was presented by Claesson in 2003. The model modifies the creep numbers from an oedometer curve according to Figure 12. This model is used in GS Settlement. In addition to the standard Swedish soil parameters five more parameters need to be defined according to Figure 12, these parameters are a_0, a_1, b_0, b_1 and r_0 (Olsson, 2010)

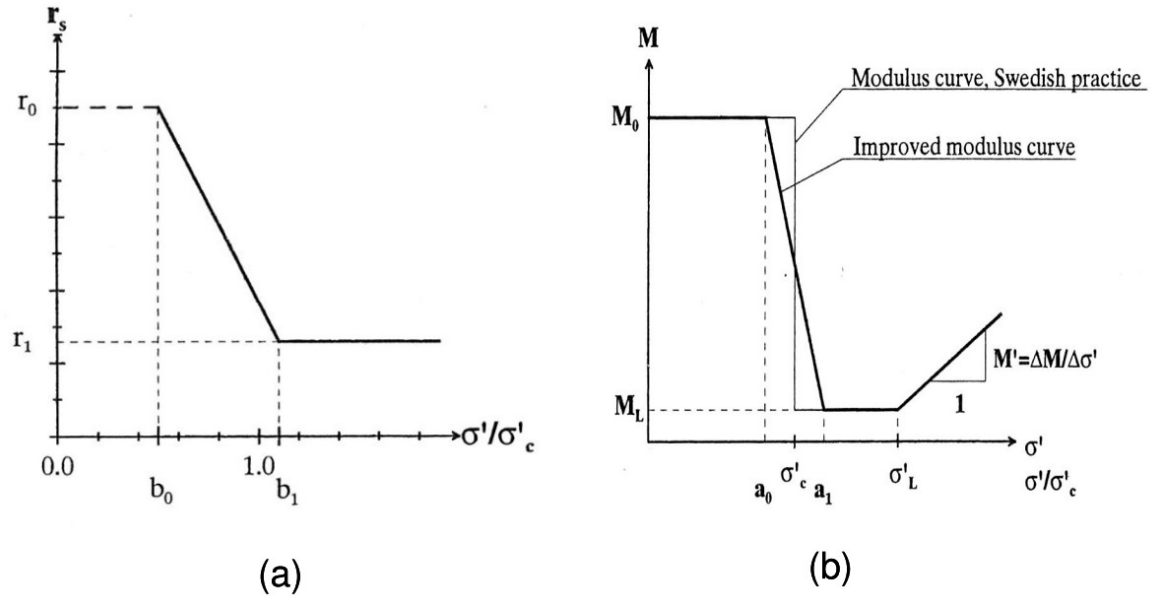

Figure 12 (a) The creep number model, showing b0 and b1 and (b) the oedometer modulus curve, showing a0 and a1 as a function of the normalised effective stress, (Claesson, 2003).

The parameters a0 and a1 are normally set at 0.8 and 1.0 by standard Swedish practise. The method is graphically evaluated from CRS oedometer tests as shown in Figure 13. (Olsson, 2010)

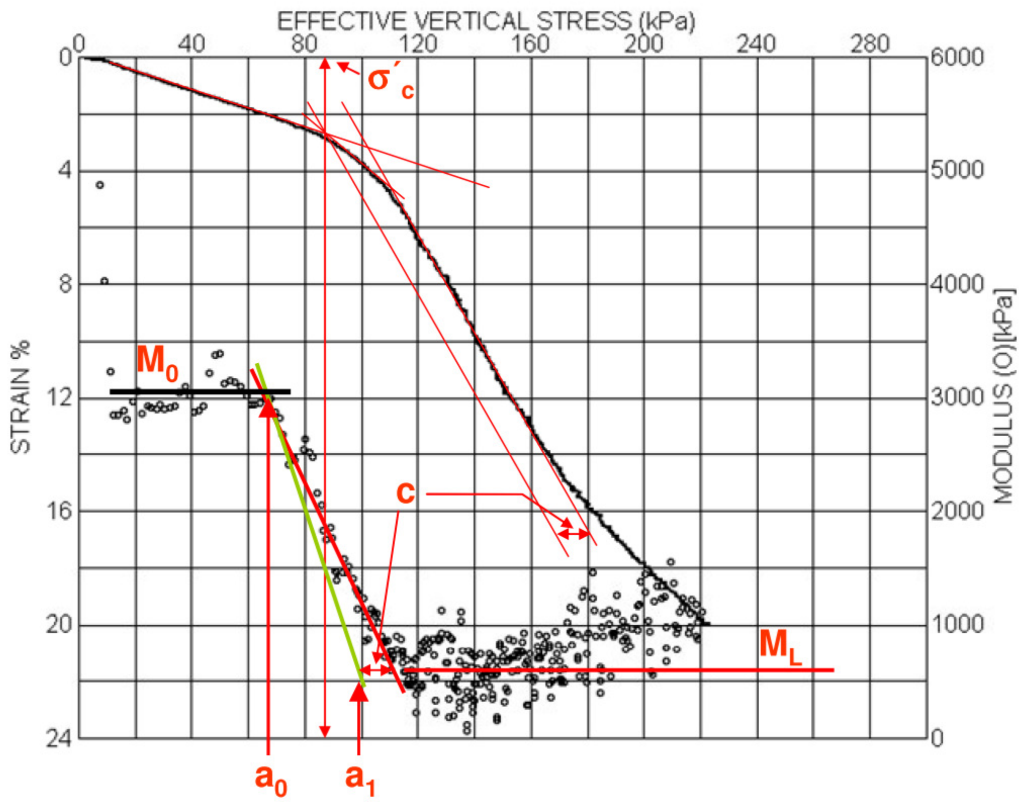

Figure 13. Graphical evaluation of the parameters a_0, a_1, M_0 and M_L from a CRS oedometer test accordingly to Swedish practice (Olsson, 2010).

It is assumed that a0 is not strain rate dependent. Which is supported by studying a series of CRS oedometer curves done with different strain rates by Sällfors (1975) which is shown in Figure 14 The point when the M_0 falls towards M_L stays relatively constant for the different strain rates tested.

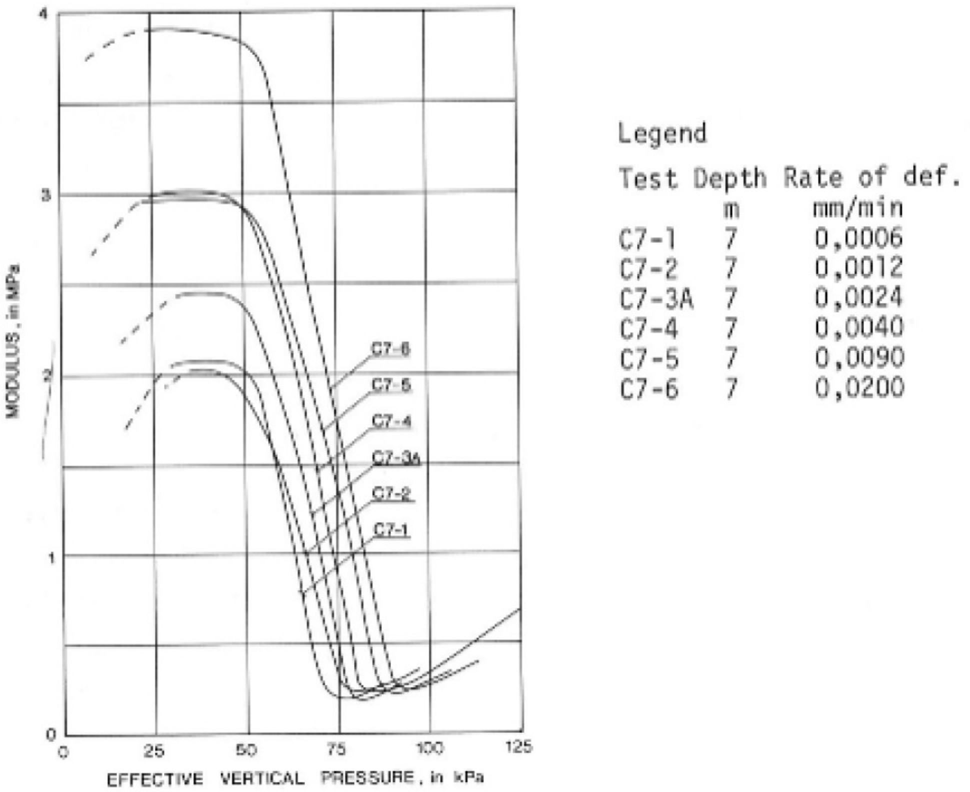

Figure 14 Differences in oedometer modulus for varying strain rates, (Sällfors, 1975).

When evaluating r_0 with corresponding b_0 the value for b_0 is for simplicity set to (σ'_{v0} /σ'_{vc}) to relate to the in-situ effective stress. The method for determining r_0 is based on the idea that the creep number, r, increases towards infinity and is stress dependent witch is described as a hyperbolic equation, see eq 15.

$$r(\sigma') = \frac{\psi}{\sigma'_{vc}}\left(\sigma'_{vc} \cdot b_1 - \sigma_{ref}\right) \cdot \left[\frac{\sigma'_{vc} \cdot b_1 - \sigma'}{\sigma' - \sigma_{ref}}\right] + r_1, \quad \sigma_{ref} < \sigma' < \sigma'_{vc} \cdot b_1 \qquad (15)$$

The ψ factor is evaluated from IL oedometer tests and represents the slope of r between r_0 and r_1 when stress $> b_1 \cdot \sigma'_{vc} \cdot \psi$ can vary from 2000 to 3000 and b_1 is usually set 1.0-1.1, (Claesson, 2003).

Since the Chalmers model is a linear method equation 15 gives the maximum r value that corresponds to the final stress $\sigma'_0 + \Delta\,\sigma$, see Figure 15.
(Olsson, 2010) Suggests that $\sigma_{ref} = \sigma'_c /1.35$.

$$r_0 = \psi \cdot \frac{(\sigma'_{vc} \cdot b_1 - \sigma_{ref}) \cdot (b_1 - b_0)}{\sigma'_{v0} + \Delta\sigma - \sigma_{ref}} + r_1 \qquad \sigma'_{v0} + \Delta\,\sigma \le \sigma'_{vc} \cdot b_1 \qquad (16)$$

The creep number, r, is shown in Figure 15 together with eq 15 and eq 16.

The time resistance number is estimated empirically using equation 17 from the natural water content according to (Olsson, 2010).

$$r_1 = \frac{75}{w_N^{1.5}} \qquad (17)$$

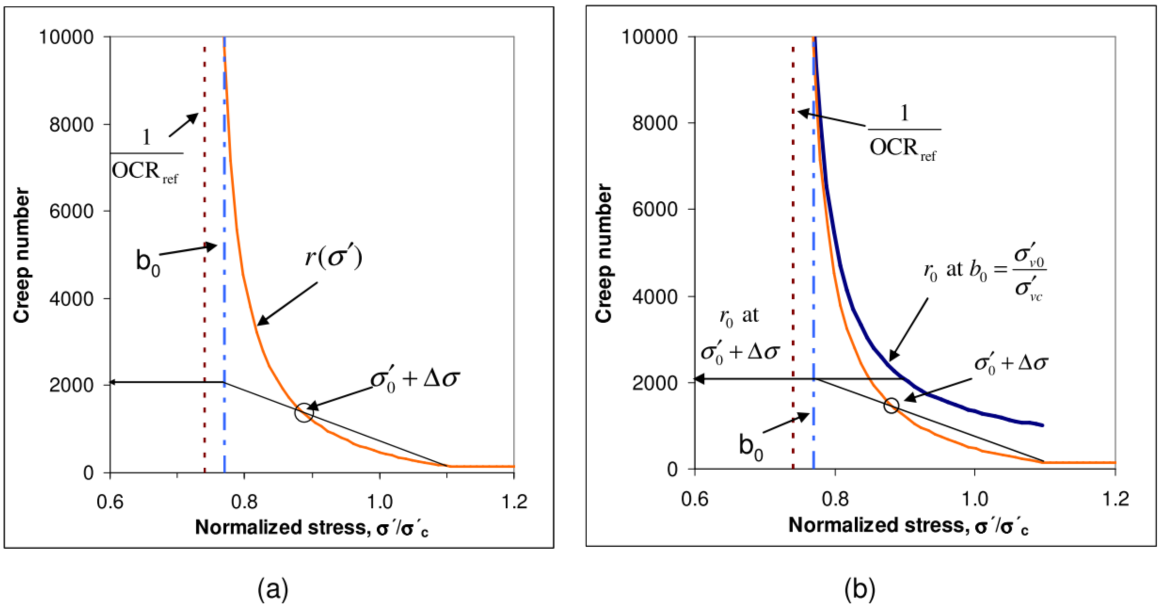

Figure 15. In (a) the creep number, r, and in (b) both r and r_0 as a function of normalised effective stress with r_1 =150, b_0 =0.75, b_1 =1.1 and ψ=2500.

When evacuation r_0 both present effective stress and final effective stress need to be taken in to consideration. Equation 16 and 17 can be used when choosing between different models. If the creep number, r, becomes high the creep would not have very little, or no, influence on the final settlements and there is no need for using a model that involves creep.

3.2 Plaxis 2D description

PLAXIS 2D is a FEM program that calculates in 2-dimensions for analysis of geotechnical problems such as stability and deformation. PLAXIS was produced out of Delft University of Technology by initiative of the Dutch government. It can use several different kinds of soil models such as Soft Soil Model, Mohr Columb and Hardening Soil. However in this project Soft Soil Creep will be used. (PLAXIS, 2015)

PLAXIS 2D have two different calculation types for the geometry, Plane strain and Axisymmetry, in a Plane strain calculation mode the slice you look at is infinite long in the y direction and for Axissymmetry the geometry is circular and you look only at circular segment that rotates around it central axis (see Figure 16).

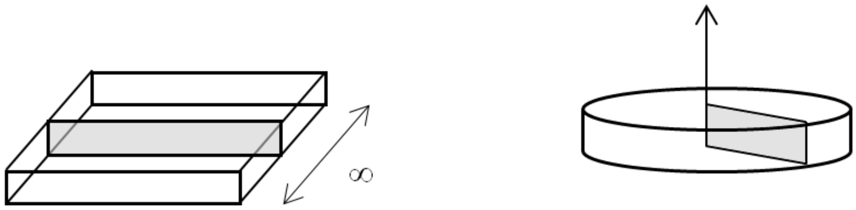

Figure 16. To left is the plane strain geometry and to the right axissymmetry, the grey segment is what modelled in PLAXIS 2D.

For further information about how PLAXIS calculates consult the PLAXIS manual.

3.2.1 Soil model (Soft Soil Creep)

Soft Soil Creep model is an adaption of the elasto-plastic Soft Soil model that also includes creep in the model. PLAXIS recommends it for when the main problems is compaction of soft soils such as clays. The basic concepts of the model is as follow (PLAXIS, 2015)

- Stress-dependent stiffness (logarithmic compression behaviour)
- Distinction between primary loading and unloading-reloading
- Secondary (time-dependent) compression
- Ageing of pre-consolidation stress
- Failure behaviour according to the Mohr-Coulomb criterion, cap is hardening according to MCC

This gives a yield surface as follows (Figure 17), where M_{cs} can be seen as a shape factor for the cap and is based on the resting stress K_0, and M_{mc} is the Mohr-Coulomb failure criterion. On the cap associated flow can occur that means that if the stress point reach the cap it will increase (softening). However if the stress point reach the cone it will lead to failure and no softening is occurring.

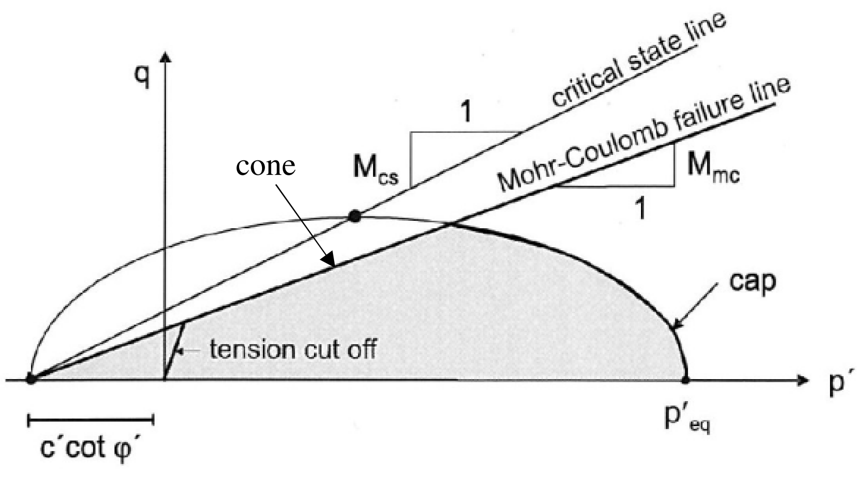

Figure 17. The yield surface of SSC model along the p' axis, the c_{ref} is the value where the mohr-coulumb failure line meets the q axis. (PLAXIS, 2015)

The strain is formulated in SSC as

$$\varepsilon = \varepsilon^e + \varepsilon^c_{dc} + \varepsilon^c_{ac} = A \cdot \ln\left(\frac{\sigma'}{\sigma'_0}\right) + B \cdot \ln\left(\frac{\sigma_{pc}}{\sigma_{p0}}\right) + C \cdot \ln\left(1 + \frac{t'}{\tau_c}\right) \qquad (18)$$

Where ε is the total logarithmic strain from the change of stress from σ'_0 to σ'. The total strain is divided into elastic strain (ε^e) and a creep part (ε^c_{dc} and ε^c_{ac}), there dc stands for during consolidation and ac after consolidation. And t´ stands for the effective creep time and is t´ = t – t_c can be viewed in Figure 18 together with how to derive C from oedometers tests that is the parameter that gives the creep rate.

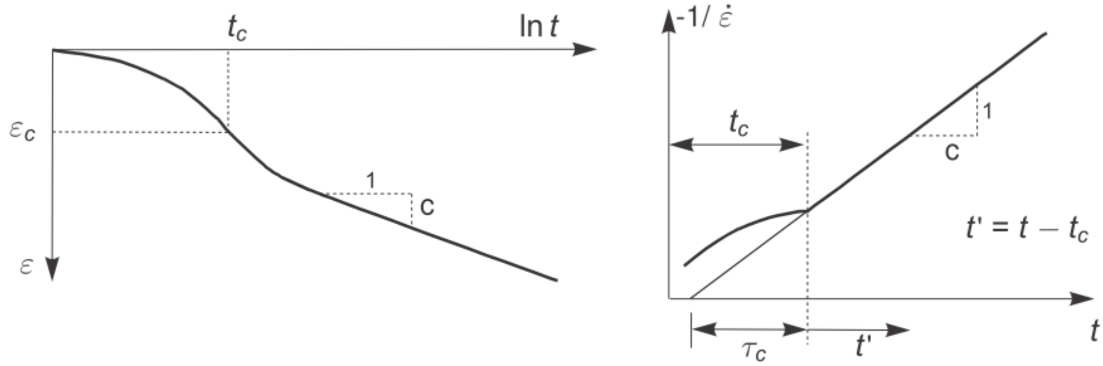

Figure 18. How to determine C from a standard oedometer test (PLAXIS, 2015)

A and B can also be evaluated from oedometer tests if strain is plotted against stress see Figure 19.

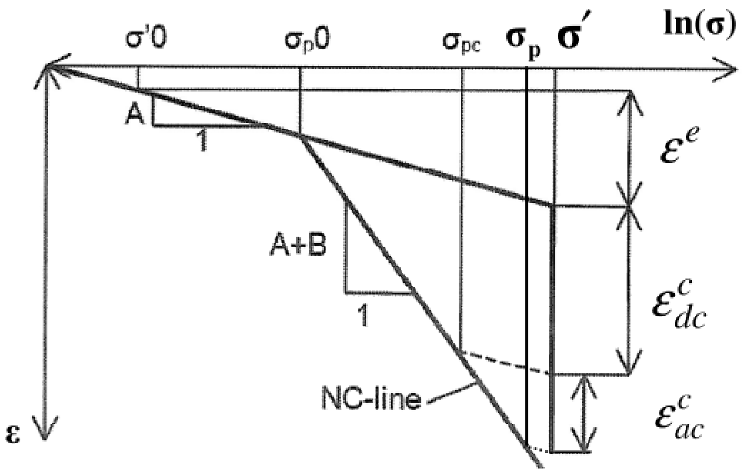

Figure 19. How A and B is derived from a logarithmic stress strain graph. In the figure the different strain increments is shown (Olsson, 2010).

However the parameters used in SSC is transformed from A, B and C to λ*, κ* and μ* respectively where the transformed as follows where v_{ur} is the poisson ratio

$$\kappa^* \approx \frac{3\cdot(1-v_{ur})}{1+v_{ur}} \cdot A, \quad \lambda^* = B + \kappa^*, \quad \mu^* = C \qquad (19)$$

Using this new modified parameters the volumetric strain for creep, $\dot{\varepsilon}^c_v$, is calculated

$$\dot{\varepsilon}_v^c = \frac{\mu^*}{\tau}\left(\frac{p^{eq}}{p_p^{eq}}\right)^{\frac{\lambda^*-\kappa^*}{\mu^*}}, \quad p_p^{eq} = p_{p0}^{eq} \cdot \exp\left(\frac{\Delta\varepsilon_v^c}{\lambda^*-\kappa^*}\right) \qquad (20)$$

If the previous equation gets integrated for a constant stress state the change of the yield surface over a set number of days, Δt, is

$$\Delta\varepsilon_v^c = \mu^* \cdot \ln\left(1 + \frac{\Delta t}{\tau}\left(\frac{p^{eq}}{p_{p0}^{eq}}\right)^{\frac{\lambda^*-\kappa^*}{\mu^*}}\right) \qquad (21)$$

Where τ is one timestep/day in SSC, and this equation defines the time dependent creep strain, and it also shows that the over consolidation ratio $\left(p^{eq}/p_{p0}^{eq}\right)$ have a great impact on the resulting strain.

3.3 Asaokas method

Since the need the way of predicting consolidation and settlements can be difficult from gathered soil data, an observational method can be used such as Asaokas method, where settlement is measured over time to predict the final settlement for one dimensional consolidation. First presented by Asaoka in 1978 it can give a good indication of the final settlement of an embankment.

The method makes use of the fact that the settlement is measured for equal long time steps, where the time is j, the settlement can then be formulated as (Asaoka, 1978)

$$S_j = \beta_0 + \beta_1 S_{j-1} \qquad (22)$$

Where S is the settlement at j and $j-1$ and β_0 and β_1 is unknown that comes from plotting the settlements, where β_0 is the intercept for the best possible line and the y axis when the settlement is plotted S_j against S_{j+1} and β_1 is that line gradient, see Figure 32.

The final settlement can then found by interception of the line x=y and the best fitting line or be calculated by the following calculation.

$$S_f = \frac{\beta_0}{1-\beta_1} \qquad (23)$$

3.4 Hyperbolic method

The hyperbolic method is another way of predicting the final settlement from measured data, the method works by plotting the time against the ratio between time and settlement, from this curve the line that best fit the graph can be found, see Figure 33 (Tan, Inoune, & Lee, 1991). From this line the gradient, *β,* is taken and the final settlement can be calculated by this following equation

$$S_f = 1/\beta \qquad (24)$$

4 Case study Göingegården

The examined test embankment named Göingegården Test-embankment is located about 5 km north east of Varberg and 500 meters to the west of Trönninge By and was raised under supervision by Madelene Markusson, geotechnical engineer at WSP. The building process of the embankment was started the 9 February 2012 and finished the 13 February 2012.

The settlements of the embankment have been monitored by using Ground Peglar that is fixed to the old surface layer as seen in Figure 20, this method will show the total settlements of the whole soil profile.

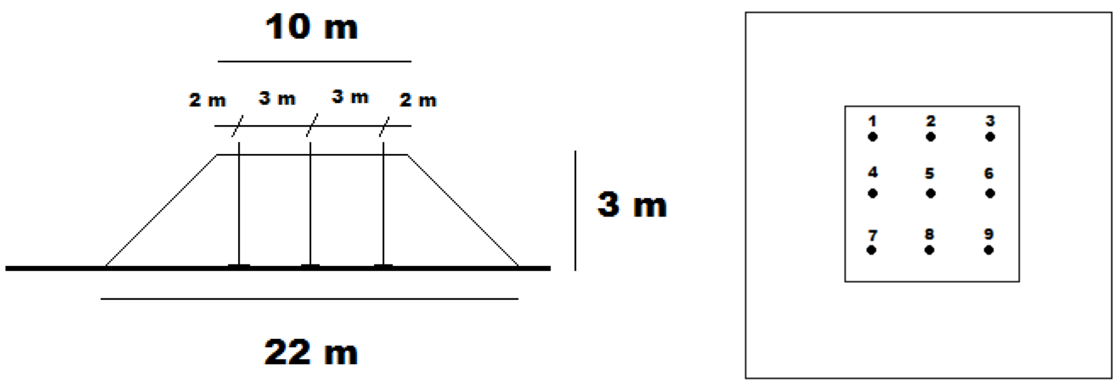

Figure 20. The design for the embankment with the profile view to the left and the top down to the right.

4.1 History of the area

The area where Göingegården test embankment is located has been farming community since at least the 1600 century where a mill have been mentioned at the location. (Riksantikvarieämbetet, 2004)About 200 meters from the test embankment there are two small buildings which are the remains of an 1800 century settlement called Klockartorpet and also a small Wastewater treatment plant. About 40 meters from the base of the test-embankment there is a dumping site for soil and rubble that origins from the construction-site of the new school see Figure 21. This new dumping-site was established about a year ago, and could affect the settlement at the location of the embankment by influencing the earth pressures.

4.2 Geotechnical parameters for the area

The soil layer for the area has around 22- 25 meters to the bedrock, with a 12 meter clay layer on top. There after the soil profile is alternating between clay and more permeable material than clay, such as sand and silt, see Appendix 2 Conrad.

During the project two new borehole have been drilled to examined the soil layer, one under the embankment that is called 16w1, and one 20 meters from the embankment that is called 16w2. There is also an old borehole called BH1 that was drilled before the construction of the embankment, see Figure 21.

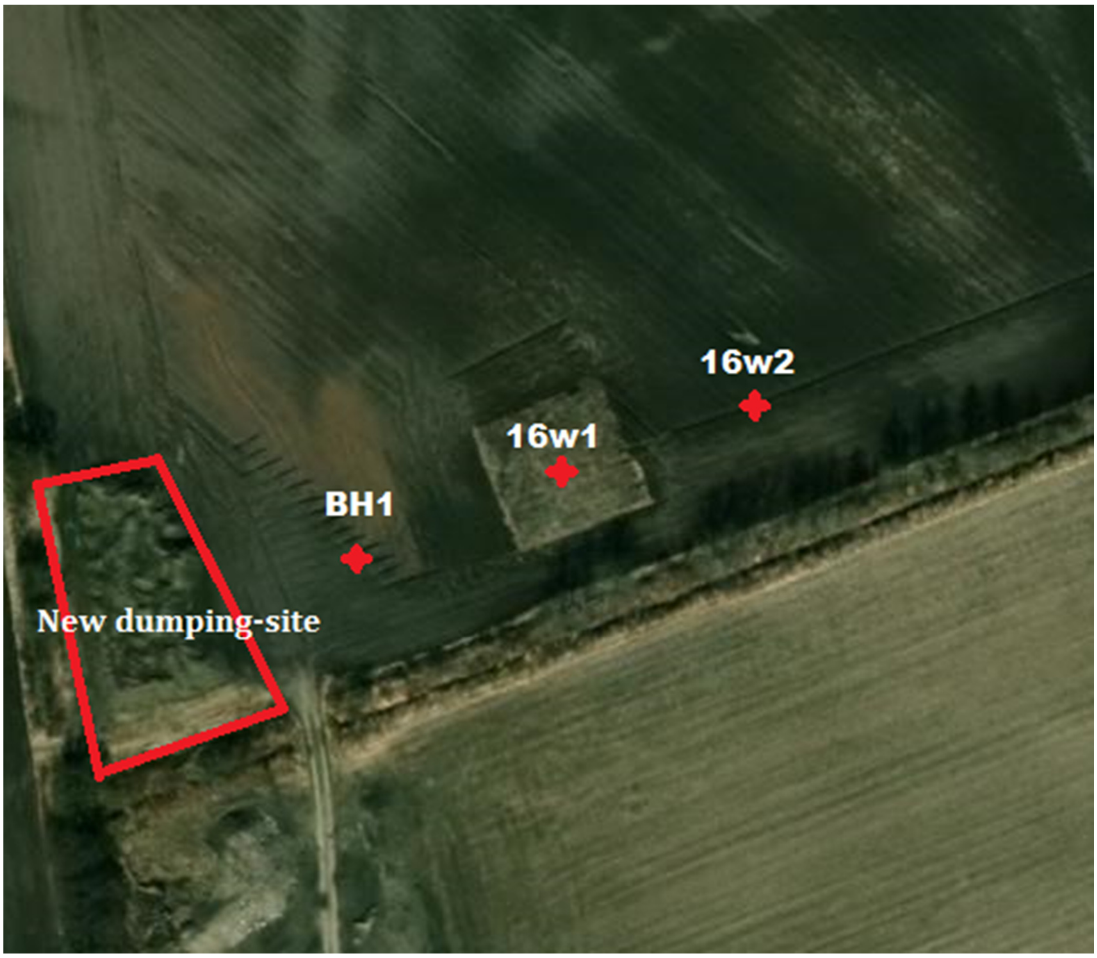

Figure 21. Location of the different boreholes and the new dumping-site. 16w1 is in the middle of the embankment.

4.2.1 Soil parameters

The following sections show the evaluated soil parameters, gathered from undisturbed soil samples for boreholes around the test embankment, see Appendix 3, necessary for settlement calculations.

Unit weight – γ

The density for the top layers above 8 meters is believed to be 16 kN/m^3, below 8 meters the soil weight is believed to be 17 kN/m^3. It is also assumed that the first 0,8 meter of dry crust have a density of 17 kN/m^3, see Figure 22.

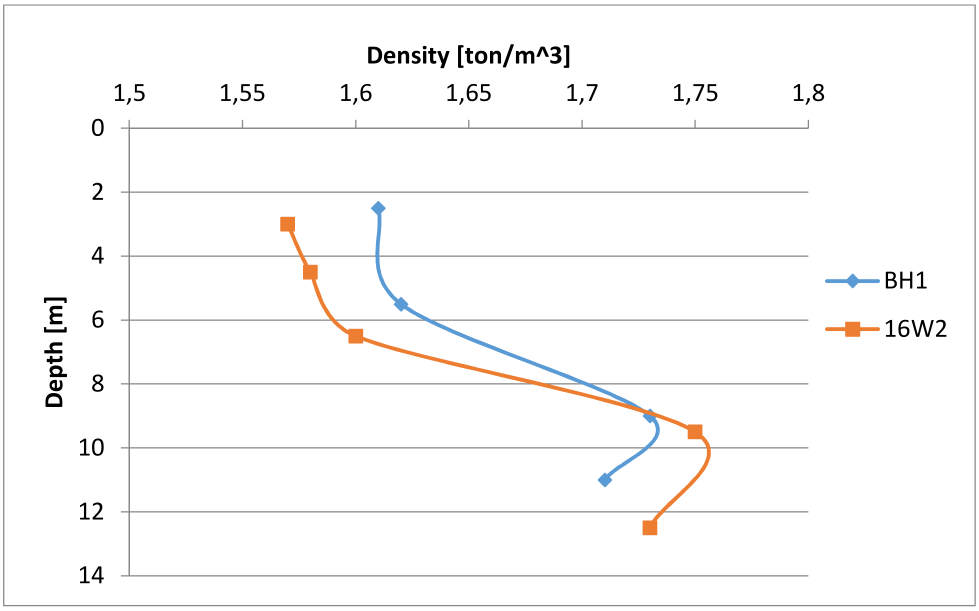

Figure 22. The density over the depth.

Initial pressures

With the density of the layers the in-situ soil pressures can be calculated in the soil profile, so the OCR can be determined by comparing with the σ’c from the CRS tests with the in-situ test. Also an equation for how to calculate the sigma c was found by comparing the test, the equation is presented here

$$\sigma'_c = 44.2 + 7.1z \qquad (25)$$

Where z is the depth is in meters below ground level, this equation is found by matching the values given from the CRS tests that is shown in Figure 23. The results from the IL tests, and the value 16w2 from the 9,5m depth have been omitted since they are believed to be disturbed and or incorrectly carried out. Borehole 16w1 is also not include since this borehole is located under the test-embankment and are therefore not representative for the initial pressures before the embankment were constructed.

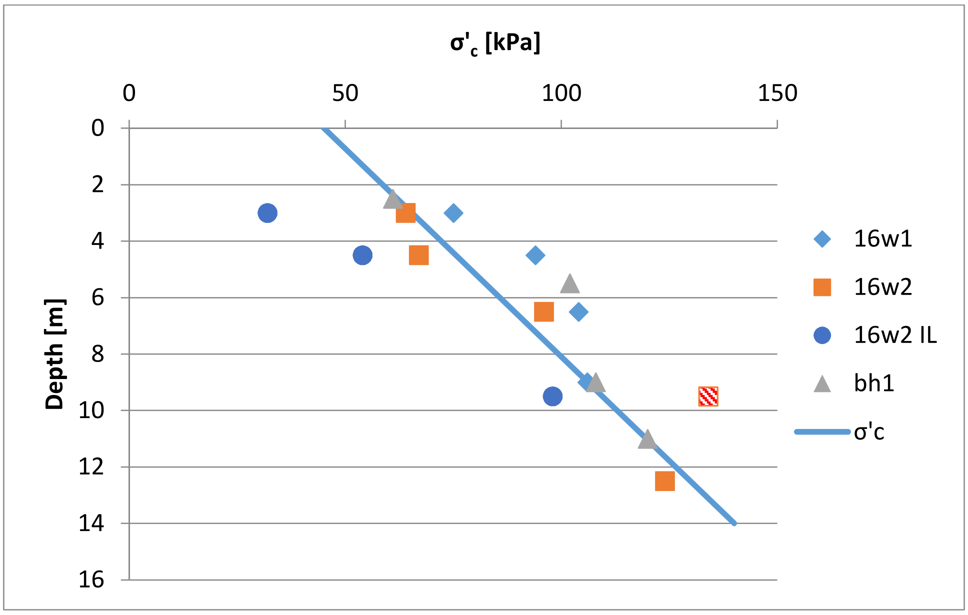

Figure 23. The various σ'$_c$ values plotted at depth values. Notice that σ'$_c$ is higher under the embankment, 16w1.

When the σ'$_c$ is determined the over consolidation ratio can be calculated (OCR), that is used in the PLAXIS model. In GeoSuite the equation for the σ'$_c$ is used. As seen in the diagram the value given from the CPTu test do not follow the calculated from the CRS tests, there for it is deemed faulty.

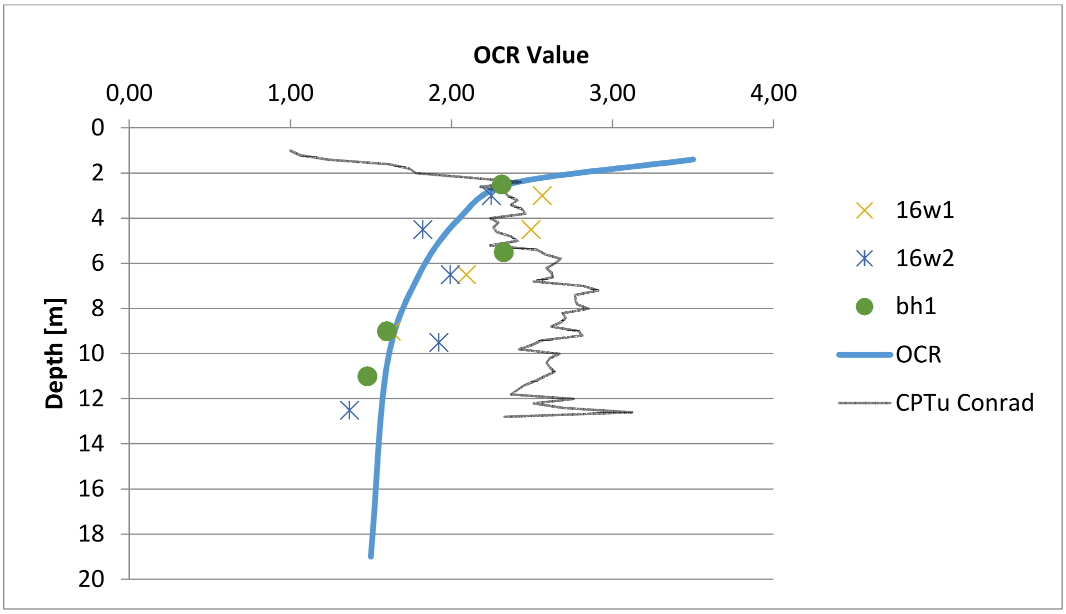

Figure 24. OCR plotted over depth, where thick blue line is the chosen values. The CPTu examination is seen as unreliable.

From this a stress diagram for the soil layer can be created, see Figure 25, where is shown that the top 3 meters in the soil profile is affected of the embankment loads.

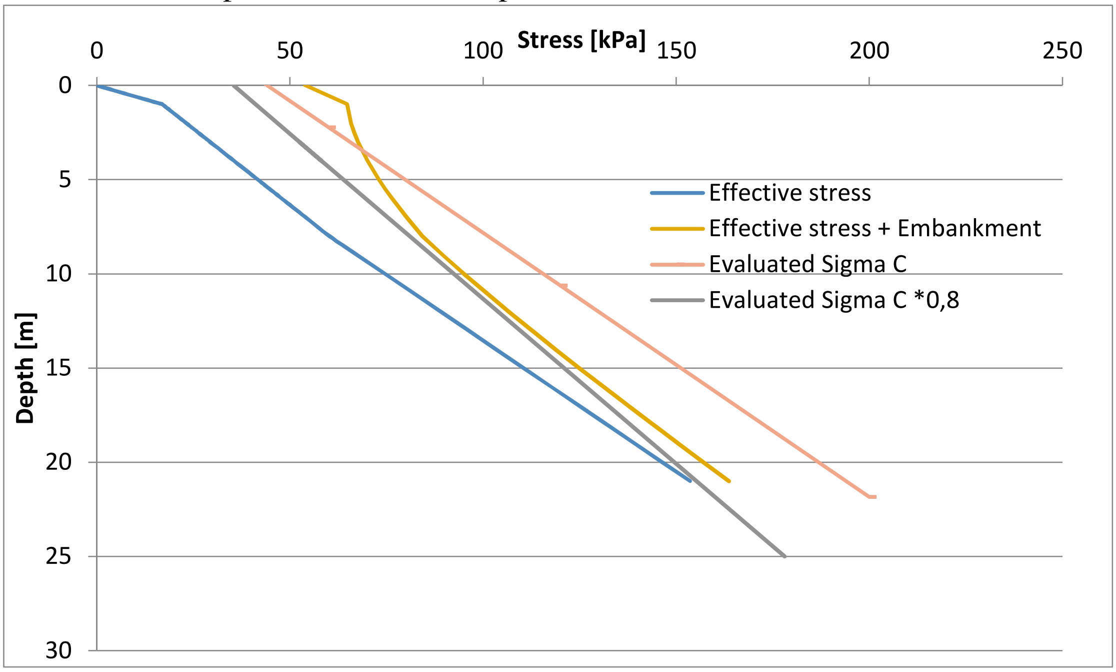

Figure 25. Stress chart for the soil profile when using the linear evaluated σ'c.

Permeability - k_{int}

The permeability is also an important parameter to validate since it determine how fast the increased pore pressures can evaporate over time. In the chosen soil profile it is believed that the permeability is decreasing linear between 0 to 13 meters by equation XX and thereafter is a higher value chosen (0,8 m/y) since it is to believe that more sand and other more permeable material is included in the soil layer that shorten the time for dissipation, however there are lot of clay still left so it can be believed that mean permeability in the layer is matching a fine silt material. (Larsson, Jords Egenskaper, 2008)

$$k = 0.12 - 0.00815z \;\; for\; 0 \leq \; z \; \leq 13 \qquad (26)$$

Where z is the depth under the original ground level.

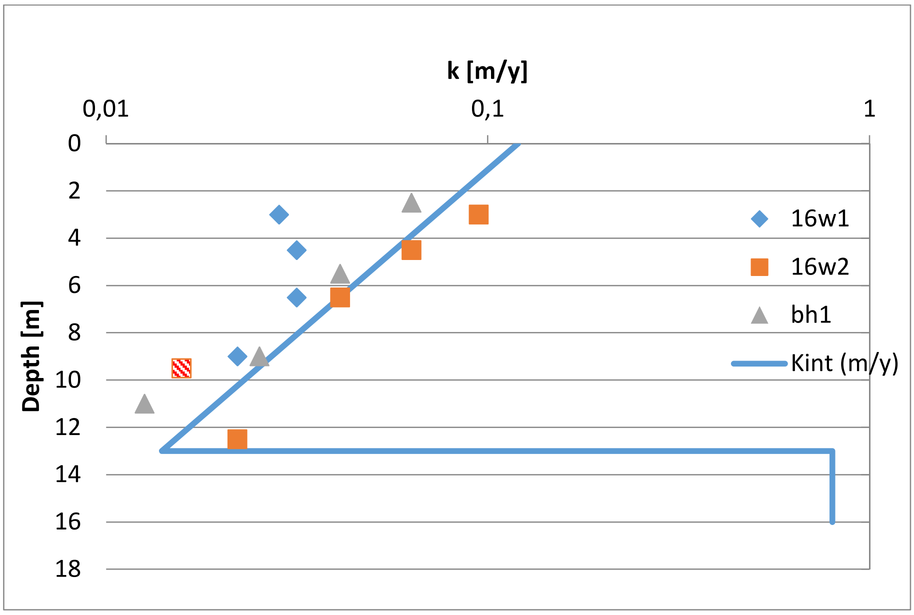

Figure 26. The permeability from different bore holes and what have been chosen

Compression modulus – M_L, M_0, M´

The compression modulus M_L and M´ are evaluated from CRS odometer tests from boreholes close to the embankment, the evaluated results can be seen in Appendix 3. The evaluated values for M_L are plotted against depth together with the evaluated trend line used in calculations in Figure 27.

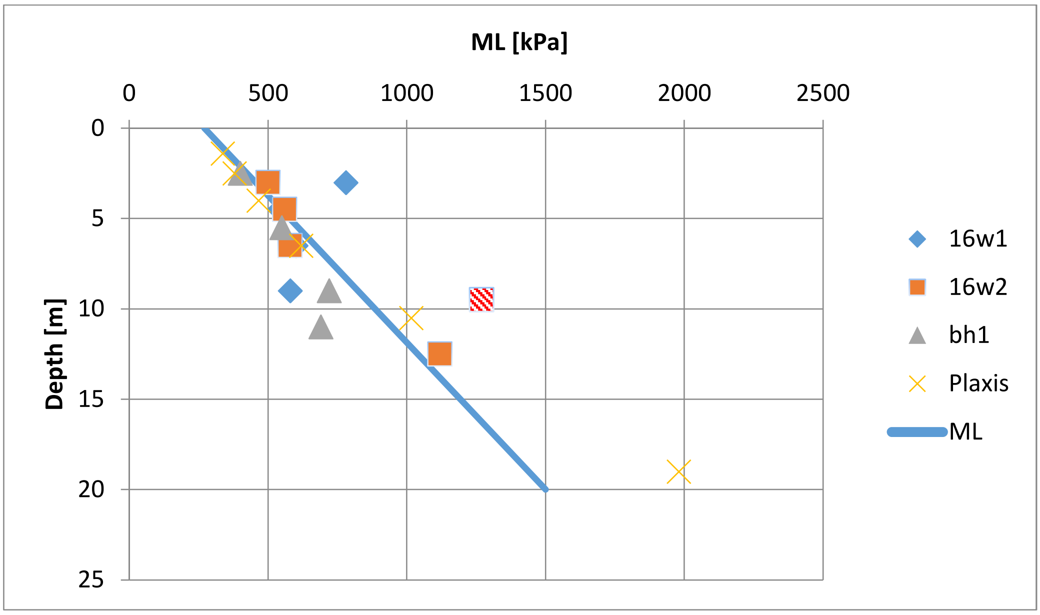

Figure 27 *Compression modulus M_L plotted against depth. The solid line shows the evaluated trend line used for the calculations.*

The compression modulus increases linearly with depth and the value of M_L can be calculated using the following formula:

$$M_L = 270 + 62z \qquad (27)$$

The value at 9.5m from 16w2 are omitted since this value is not following the trend. When evaluating the value of M_0 have been evaluated using two different empirical equations, from c_u (eq 28) and from σ'c (eq 29).

$$M_0 = K \cdot c_u \qquad (28)$$

Where K is an empirical constant evaluated for different clays, set to 350.

$$M_0 = 50 \cdot \sigma'_c \qquad (29)$$

The evaluated M_L values can be seen in Figure 28 together with the evaluated trend line.

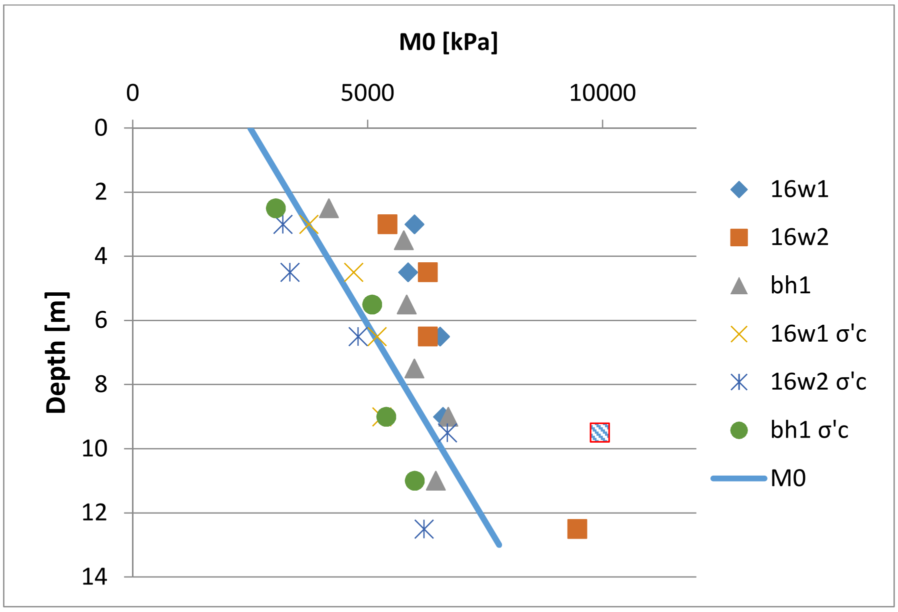

Figure 28 Evaluated M_0 with different empirical correlations plotted against depth. The solid line shows the evaluated trend line used for the calculations

M_0 is considered to be linear with depth and can be calculated using the following formula:

$$M_0 = 2500 + 408z \qquad (30)$$

Factors for improved modulus – a_0, a_1

The parameters a0 and a1 are set to 0.8 and 1.0 according to standard Swedish practise shown in 3.1.1.

Time resistance number – r_0, r_1

The time resistance number r_1 is set using the empirical equation 31 (Olsson, 2010)

$$r_1 = \frac{75}{w_N^{1.5}} \qquad (31)$$

The value of r_1 is constant at 180 from 0 to around 8m depth and from 8m and deeper r_1 is considered to be 250, see Appendix 4

Evaluation of the initial time resistance r_0 where done using equation 15 from 3.1.1. The evaluated r_0 is plotted against depth in Figure 29 r_0 plotted against depth with the condition σ'0 + Δσ > σ'c.

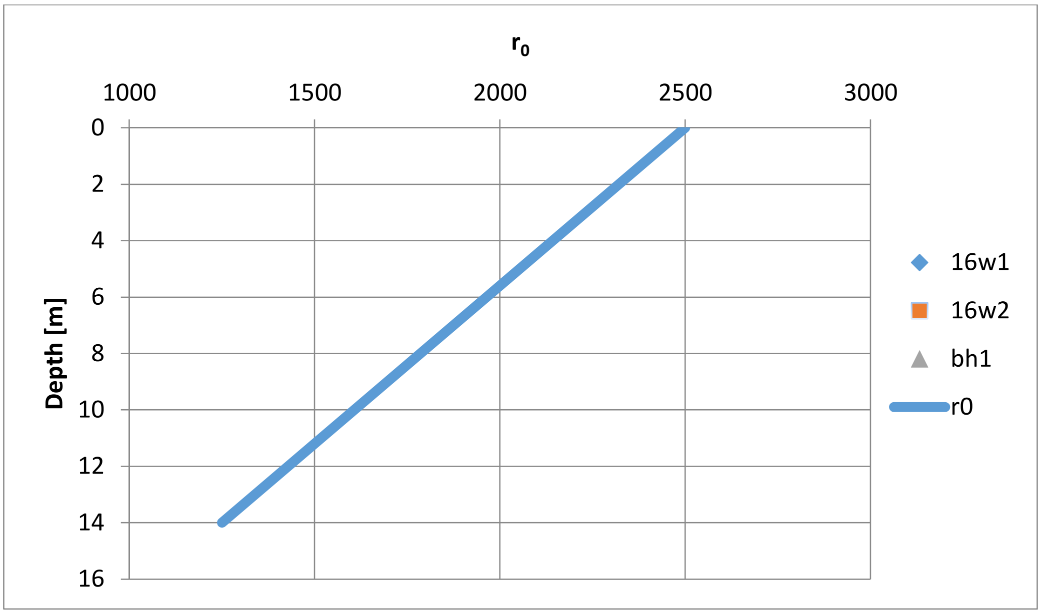

Figure 29 r_0 plotted against depth with the condition σ'0 + Δσ > σ'c.

The equation describing the trend line for r_0:

$$r_0 = 2500 - 90z \qquad (32)$$

Factors for time resistance number – b_0, b_1

The time resistance factors b_0 and b_1 is important when describing the creep behaviour in full-scale. b_0 is set to σ'_0/σ'_c and b_1 is set to 1,1 according to (Claesson, 2003).

Undrained shear strength - c_u

The undrained shear strength is a useful parameter to validate since it is one of the ways the M_0 is determined, it can also be of interest for calculating failure surfaces and piling dimensions. Since it not used directly in any calculation in the report the chart is found in appendix 4 where the shear strength is plotted against the depth.

Water content W_n and liquid limit W_L

The water content and the liquid limit are collected from the different boreholes around the test embankment, see Appendix 3. The values are plotted against depth and is shown in Appendix 4. The water content varies from 60% - 80% in the first meters down to around 30% - 40% at depths of 12m. The variations in liquid limit are about the same as water content, starting at 50% - 60% and decreasing down to 30% - 40% at 12m. When comparing the water content and the liquid limit it shows that the water content in the soil is above or very close to the liquid limit at all depths in the profile.

Pore water pressure – *u*

After evaluating the different boreholes in the area the groundwater level is constant at 0.8m. The measured pore water pressure and the hydrostatic pore pressure are plotted against depth in Appendix 4

Initial permeability - k_{int} , and permeability reduction coefficient – β_k
The initial permeability and the reduction coefficient are evaluated from CRS test. The initial permeability decreases from 0.12 to 0.014 m/year over the first 13m it then estimated to be around 0.8 m/year in more course material. The evaluated values is plotted against depth can be seen in Appendix 4.

Creep parameter - μ^*

μ^* is the same in the whole layer since the few test samples it is to derive the result from. From the three IL test the mean values of the samples have been taken to represent the whole layer. The IL tests can be found in Appendix 5. This value is only used in Soft Soil Creep.

Strength parameters - λ^* and κ^*

The λ^* and κ^* values have been derived from the CRS curves see Figure 19 and performing digital CRS test in PLAXIS Soil test on the chosen values and to try to match the actual CRS curve as close as possible. As seen in Appendix 6 note that the CRS for 9,5 meter is believed to be faulty.

Effective cohesion - c_{ref}

The c_{ref} have chosen when validating the clay layers in Soil test in PLAXIS.

Poisson ratio - v_{ur}

This value is set to 0.15 since that is the standard value in PLAXIS.

4.3 Measured settlements

The test-embankment have been monitored for over 4 years, it should be noted that there is a gap of about 2 years in the data , see Appendix 1, the total amount of settlement at the latest data set, 2016-02-22 is 297 mm. As seen in Figure 31 the primary compression is still ongoing since the rate of settlement hasn't levelled out completely, and is about 4 mm per year.

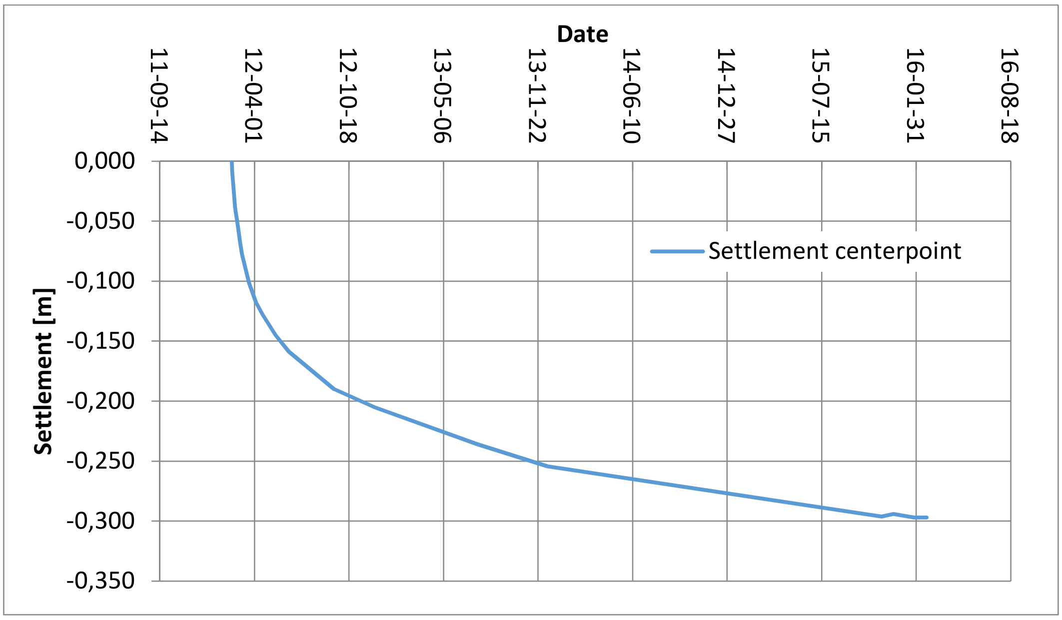

Figure 30. Settlement of the centerpoint under the embankment in meters over time. 200 days between each major tick.

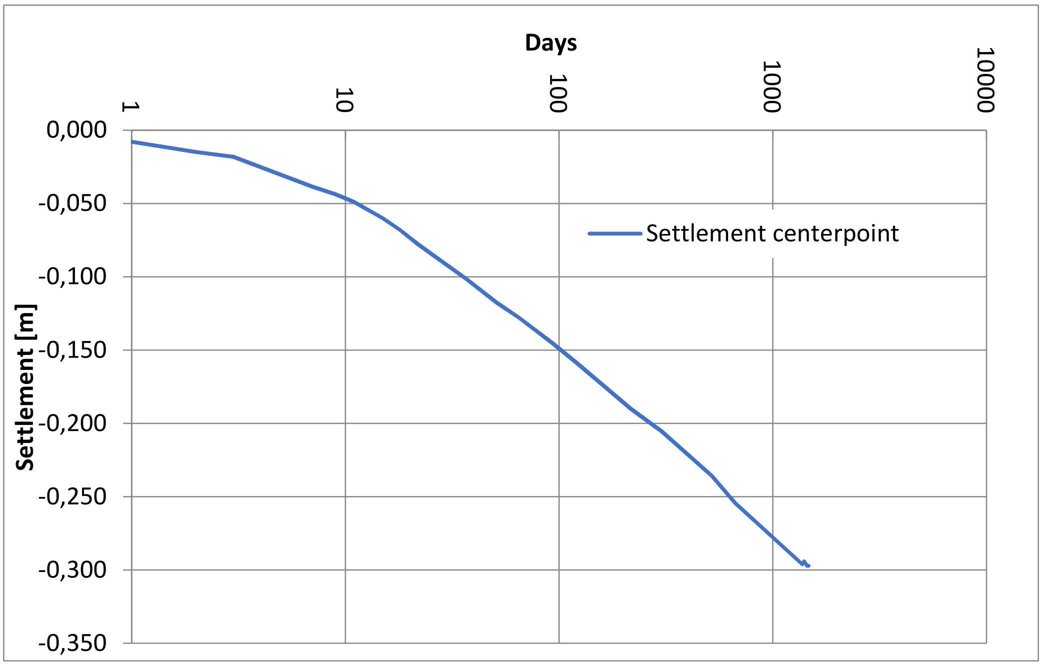

Figure 31. Settlement at the center point of the embankment in meters plotted in logarithmic scale for the days. Compare this to Figure 3 it is seen that secondary settlement haven't started yet

4.3.1 Calculated settlement by Asaokas Method

The total final settlement calculated by Asaokas method for the Göingegården test embankment is 322 mm.

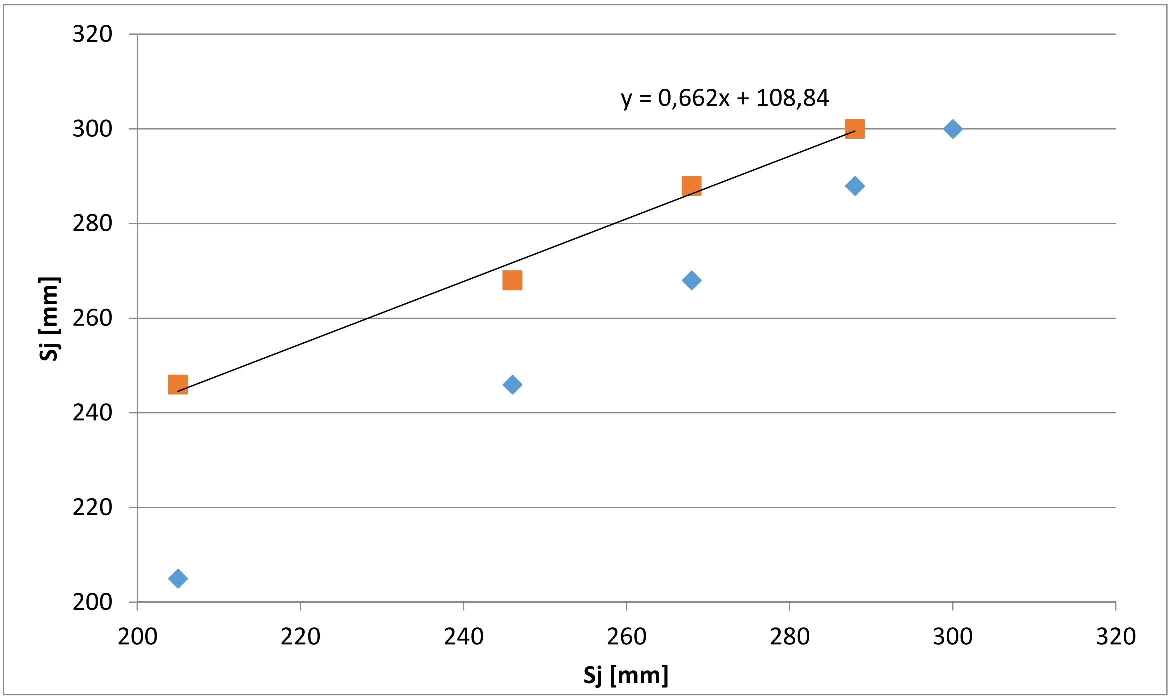

Figure 32. Asaokas method calculation. Note the scale of the axis.

4.3.2 Calculated settlement by hyperbolic method

The total settlement according to the hyperbolic method is 312 mm and the gradient of the curve is 3.201.

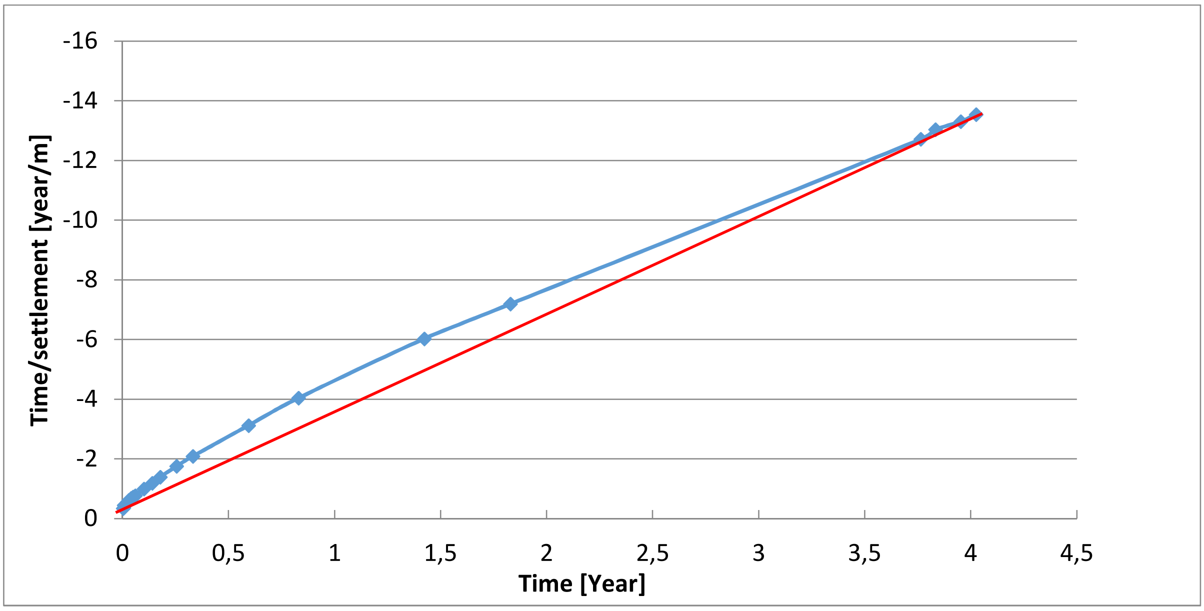

Figure 33. Time in years plotted against the ratio between time and settlement. Red straight line is from where the gradient is taken, the gradient was 3,201.

5 Calculation

5.1 Geosuite

The model used in GeoSuite is Chalmers both with and without creep and a detailed list of all in-data parameters and their values used can be seen in Appendix 5. The settlements are calculated in the centre of the embankment. The load from the embankment is modelled as a block of 16x16x3m with a unit weight of 18kN/m^3.

5.1.1 Soil profile in GeoSuite

The soil have been divided into four different layers with different parameters in order to mimic the actual soil profile, Figure 34. The thickness of the layer have been set to follow changes in different soil parameters found in Appendix 3.

The profile consists of 0,8m dry crust followed by two clay layers with changing soil parameters and at the bottom there is a layer of clay mixed with sand and silt.

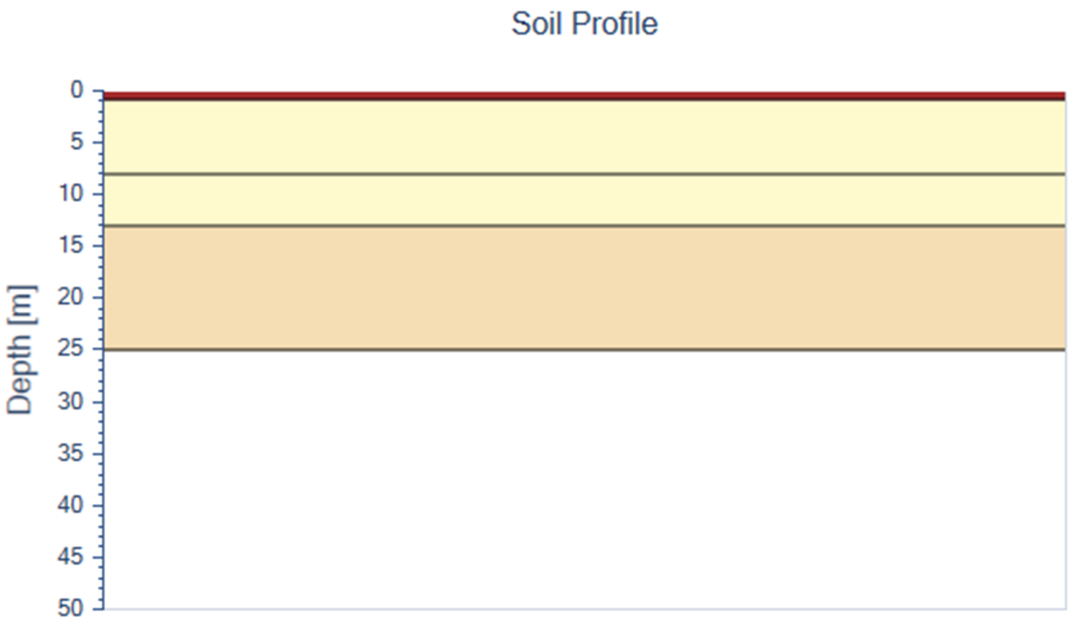

Figure 34 The soil profile in Geosuite.

The top dry crust layer is modelled without creep and are considered incompressible with high soil parameters. Most of the settlements are expected to take place in the first 13m and then slowly dissipate with depth.

5.2 Plaxis 2D

The modelling in PLAXIS have been done primary in a 15 noded plane strain model where the geometry of plane that is looked at is 44 meters wide so the width is 4 times the embankment size and 28 meters high, with origin at the center point of the embankment, see Figure 35.

The different phases used are:

- Initial stage where the in-situ pressures are set up.

- A 4 day construction period of the embankment

- Consolidation phases to 4 and 100 years to measure the settlement right now and see the total settlements.

Both the Soft soil and Soft Soil Creep have been used to see the influences the creep have on the settlement.

5.2.1 Soil profile in PLAXIS

A total of 7 layers have been used when calculating the settlement in the area where 6 layers is believed to be compressible. From top to bottom we have the embankment follow by a 0,8 meter deep dry crust. After that it is 5 clay layers and end of with a layer that is believed to be clay mixed with some amount of sand and silt, the soil profile used is shown below in Figure 35.

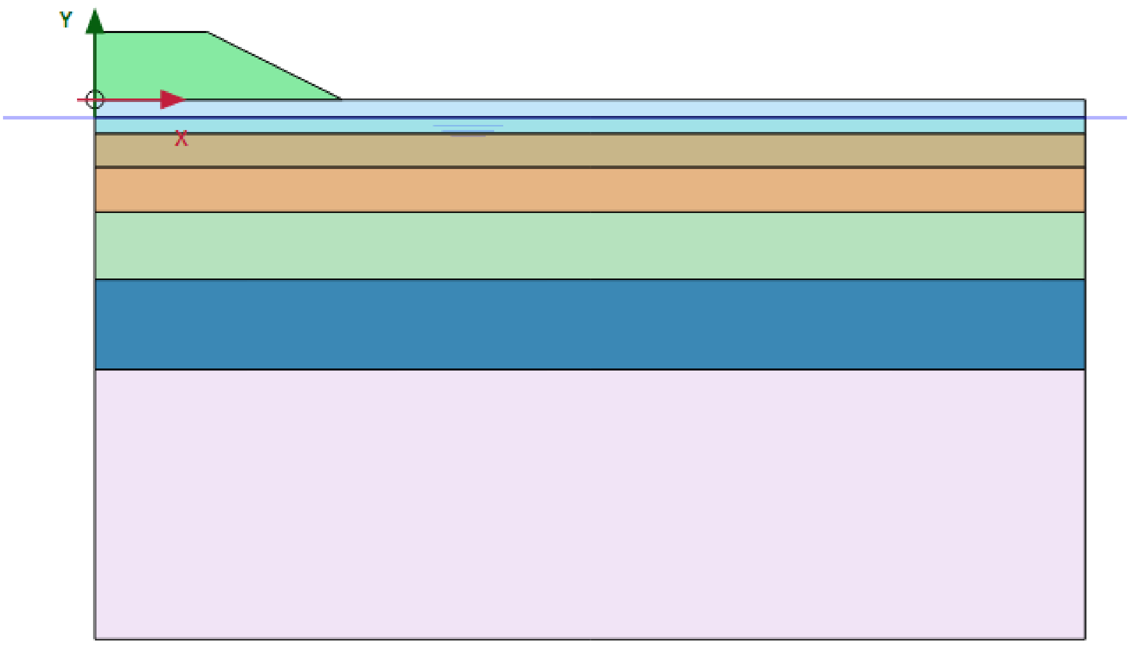

Figure 35. The chosen soil profile in PLAXIS 2D.

The chosen parameters for each layer will be presented in Table 2 and

Table 3, starting from the top with the embankment and dry crust that is modelled in a mohr-coloumb method and ending with the clay layers; those are the layers where it is believed that most of the settlements will happen. The main reason for the layering is to simulate the OCR trough the soil profile as good as possible.

Table 2. The soil layers where no creep occur.

Layer	Soil weight [kN/m^3]	Eoed [MPa]	Φ' [deg]	c ref	v	K0
Embankment	18	5	30	5	0,3	0,5
Dry crust	17	5	30	5	0,3	0,5

Table 3. Chosen parameters for the clay layers that is modelled in Soft Soil Creep.

Layer	λ^*	κ^*	μ*	c_{ref}	Soil weight [kN/m^3]	k [m/day]	OCR
0,8-2 m	0,18	0,022	7,00E-03	3	16	2,97E-04	3,5
2-3 m	0,18	0,022	7,00E-03	3	16	2,73E-04	2,35
3-5 m	0,17	0,02	7,00E-03	3	16	2,39E-04	2,05
5-8 m	0,16	0,018	7,00E-03	3	16	1,84E-04	1,8
8-13 m	0,13	0,017	7,00E-03	3	17	9,42E-05	1,6
13-25 m	0,1	0,01	2,00E-03	3	17	2,19E-03	1,5

Table 4. Chosen parameters for the clay layers that is modelled in Soft Soil.

Layer	λ^*	κ^*	c_{ref}	Soil weight [kN/m^3]	k [m/day]	OCR
0,8-2 m	0,18	0,022	3	16	2,97E-04	3,5
2-3 m	0,18	0,022	3	16	2,73E-04	2,35
3-5 m	0,17	0,02	3	16	2,39E-04	2,05
5-8 m	0,16	0,018	3	16	1,84E-04	1,8
8-13 m	0,13	0,017	3	17	9,42E-05	1,6
13-25 m	0,1	0,01	3	17	2,19E-03	1,5

5.2.2 Used mesh size

The used mesh size for PLAXIS is the medium grid size, where refinements have been done under the embankment and especially in the dry crust. Different types of meshes have been used and it was found that this was suitable when a finer mesh only increased the calculation time with small to nothing changed in amount of settlement calculated.

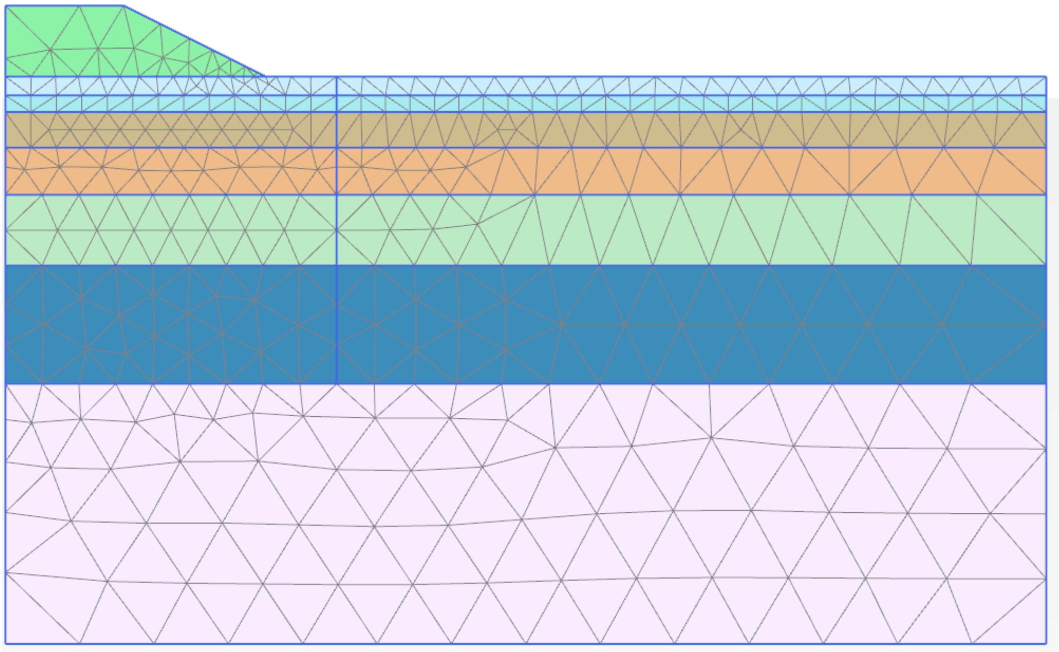

Figure 36. The used mesh size.

5.2.3 Comparison between the parameters

Since the two models use two different sets of in-data it is hard to compare the data, and there no direct translation, however it can be given an estimate of the in-data with help of a transformation matrix presented by Olsson year 2010. The results from that is shown in Table 5.

Table 5. The PLAXIS in data converted to GS parameters.

Depth	M_L	M_0	r_1
Clay 0,8-2	336,1111	2000	143
Clay 2-3	378,8889	2818	143
Clay 3-5	465,8824	3600	143
Clay 5-8	618,75	5000	143
Clay 8-13	1015,385	7765	143
Clay 13-25	1980	18000	500

6 Results from calculations

The calculation results from our best guess is presented here and compared to the calculation from the measured value.

6.1 GeoSuite Settlement Results

The predicted settlements have been calculated using the models Chalmers with and without creep, the in-data values used can be seen in Appendix 7.

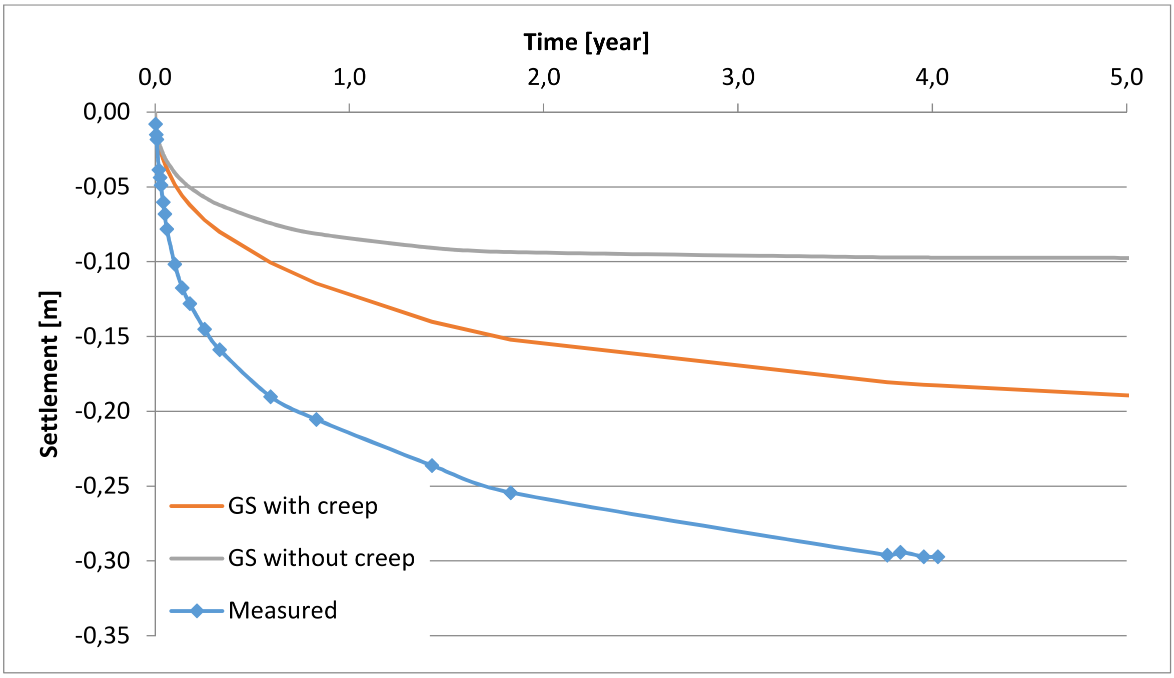

Figure 37. Settlement over time.

The calculated settlements are less than the measured values, but the rate of settlements are similar when simulating with creep between year 2 and year 4 comparted to the measured values.

Table 6 Results from the GeoSuite calculation

Calculation	Settleme nt 4 years	Ratio of actual	Settlement 100 years [m]	Rate 4 years [m/year]	Ratio of real
Measured	-0.297	0.000	-	-0.00384	1
GS without creep	-0.10	-0.67	-0.10	-0.00054	0.14
GS with creep	-0.18	-0.38	-0.24	-0.00884	2.30

6.2 PLAXIS results

The results from the centerpoint of the embankment in PLAXIS when using SSC are after 4 years 254 mm with a rate of settlement of 32 mm per year after 4 year. When using SS the total settlement is 184 mm and the rate is 17 mm per year after 4 years.

When modelling the self-compaction when no embankment is built on the soil for 1 year, is 2,4 mm. This can be compared to the speed of the self-compaction of the clay in Mölndal is 5 -10 mm per year. (Sweco Infrastructure, 2013)

Table 7. Results from the PLAXIS calculation

Calculation	Settlement 4 years [m]	Ratio of actual	Settlement 100 years [m]	Rate 4 years [m/year]	Ratio of real
Measured	-0,297	1,000	-	-0,004	1,000
PLAXIS SS	-0,184	0,620	-0,077	-0,017	4,250
PLAXIS SSC	-0,254	0,855	-1,253	-0,032	8,000

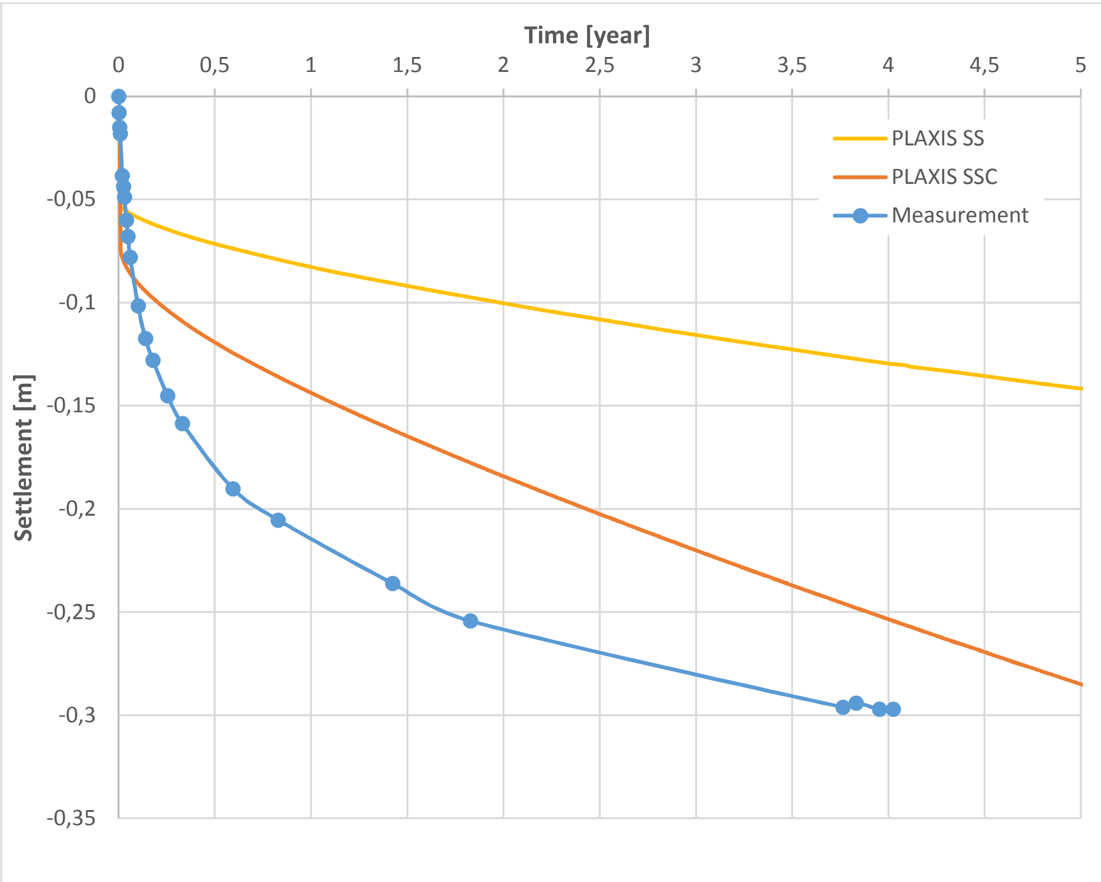

Figure 38. Settlement under the centerpoint of the embankment.

In the PLAXIS model the Pore pressures dissipate very slowly in soil profile, for example after 4 years only 30 % of the total excess amount have dissipated and over 100 years small amount of pore pressures small exist, see Appendix 8.

During the field day samples was taken under the embankment to exam effects of the embankment on the soil. It is believed that the embankment should result in a higher preconsolidation pressure for the soil profile there. In Table 8 the calculation for the expected σ'_c is presented where the maximum pore pressures from the building of the embankment is added to the in-situ stress. This would give the new preconsolidation pressures for the soil.

As seen in the result from this calculation the between 20 to 10 kPa lower than the actual value calculated from the CRS tests, but since the σ'_c is overestimated in Swedish praxis (see Chapter 2.5.1) by about 20 % these values is seen as correct.

Table 8. Comparison of the σ'_c and expected σ'_c from PLAXIS.

Depth [m]	**In-situ stress before embankment [kPa]**	**Excess pore pressures in PLAXIS from the embankment [kPa]**	**Expected σ'_c (In-situ + Excess) [kPa]**	**σ'_c from 16w1 [kPa]**
3	29,3	37,5	66,7	75
4,5	37,6	35	72,6	94
6,5	49,7	32,5	82,2	104
9	65,2	30	95,2	106

7 Sensitivity analysis

To research the uncertainties a sensitivity analysis has been performed where different parameters have been changed to see how it affect the results. The main focus point is to understand how different parameters can affect the rate settlement and the settlement.

7.1 Geosuite sensitivity analysis

The sensitivity analysis in GeoSuite is based on Chalmers with creep, since creep is present thru out the model. Only one parameter is changed each time.

7.1.1 Embankment soil weight

In order to accurately calculate settlements is important to able to define the load accurately. The soil material used to construct the embankment were originally estimated to weight 18kN/m^3. Two different unit weights where tested 15 and 21 kN/m^3. The change in unit weight can account for ether uncertainties in material composition or height of the embankment. The sensitivity in change of embankment soil weight is presented in Table 9.

Table 9 Results from sensitivity analysis for change of weight of embankment.

Calculation	Settlement 4 years	% Ratio actual	Settlement 100 years	Rate 4 years	% Actual
Measured	-0.297	0.000	-	-0.00384	1
GS without creep	-0.10	-0.67	-0.10	-0.00054	0.14
GS with creep	-0.18	-0.38	-0.24	-0.00884	2.30
GS with creep σ'c +10%	-0.17	-0.43	-0.22	-0.00653	1.70
GS with creep embank 15kN/m^3	-0.16	-0.48	-0.21	-0.00730	1.90
GS with creep embank 21kN/m^3	-0.23	-0.23	-0.32	-0.02362	6.14

The total amount of settlement increases as the load increases and the shape of the settlement curve is consistent for the change in load. When increasing the load from 18 to 21kN/m^3 the difference compared to measured results decreases from 38% to 23% and from 0,117m to 0,067m difference in total settlement. Though the difference in total settlement decreases the difference in rate of settlement between measured and calculated increases from 0.14 to 6.14. The settlement over time with different parameters can be seen in Figure 39.

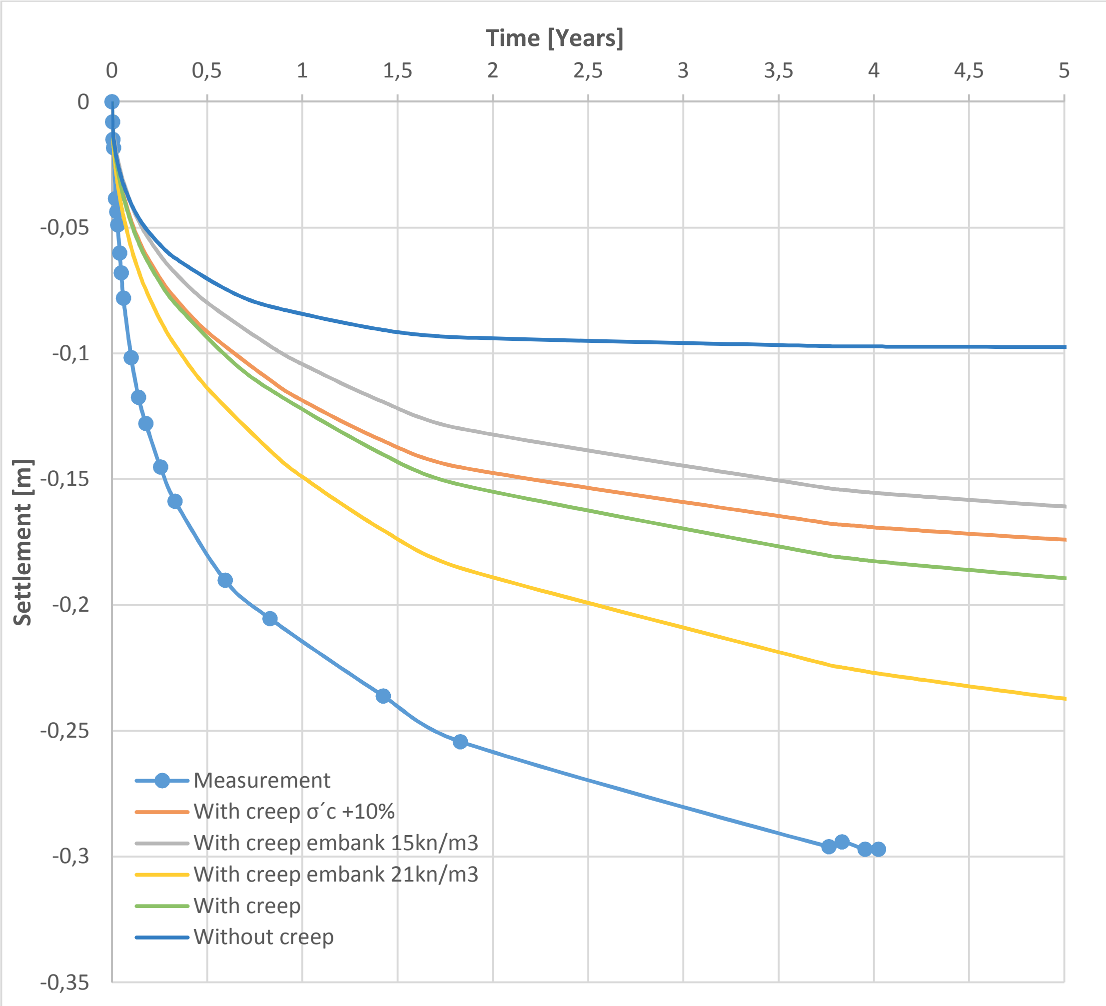

Figure 39 Settlement plotted over time with change in embankment soil weight and initial pressure σ'$_c$.

When changing σ'c +10% the curve follows the original curve for the first year and then the settlements decreases faster than originally. This can be seen when studying the settlement rate at four year which decreases from -0.00884 originally to -0.00653, this change in rate of settlement is closer to the measured -0.00384.

7.1.2 Permeability

Increasing the permeability speeds up the settlement processes and changes the shape of the settlement curve

The result from the settlement analysis, see Figure 40, show that the speed is increasing for the first 2 years. There are small changes in the total amount of settlement both over 4 years and in the calculated for 100 year.

Table 10. Results from sensitivity analysis for change in permeability.

Calculation	Settlement 4 years	Ratio actual	Settlement 100 years	Rate 4 years	Raio Actual
Measured	-0.297	0.000	-	-0.00384	1
GS without creep	-0.10	-0.67	-0.10	-0.00054	0.14
GS with creep	-0.18	-0.38	-0.24	-0.00884	2.30
GS with creep Kint*2	-0.19	-1.65	-0.24	-0.00615	1.60
GS with creep Kint*3	-0.20	-0.34	-0.24	-0.00499	1.30
GS with creep Kint*5	-0.20	-0.34	-0.24	-0.00461	1.20

When studying the rate of settlement, see Table 10, it decreases compared with the initial calculation to become closer to the measured rate of settlement after four years.

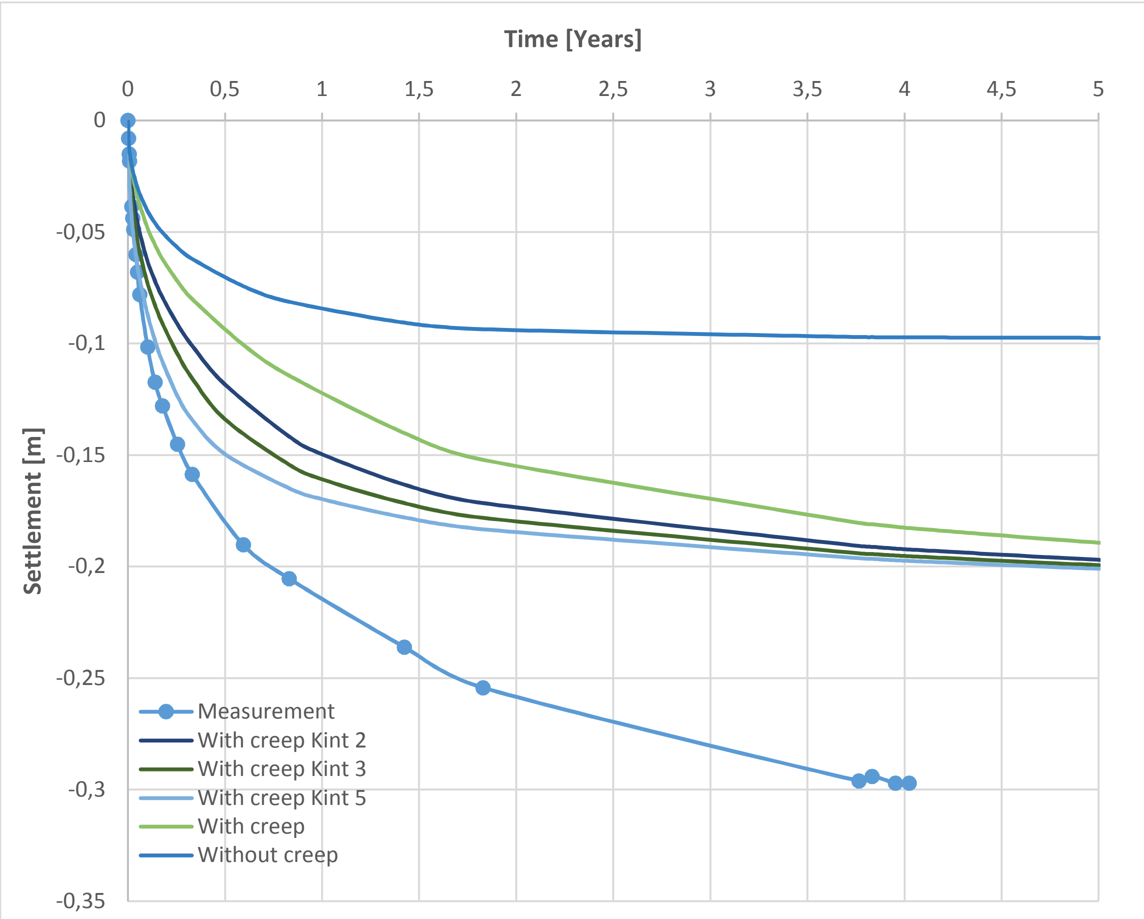

Figure 40 Settlement over time when changing the permeability.

7.2 PLAXIS sensitivity analysis

The sensitivity analysis in PLAXIS is only done in SSC since creep is happening in the area. All other parameters than the changed ones are the same.

7.2.1 Embankment soil weight

Since the density of the embankment and dry crust is uncertain, a change of the soil weight of the embankment to 15 kN/m^3 was done to see how it affected the results. The calculation gives a total settlement after 4 year that is 0,185 m that is a decrease of settlement by 27%. The rate of settlement dropped to 17 mm per year at year 4 that is a decrease of 31 %. All this can suggest that the soil weight could be wrong and should be more looked in to.

Table 9 Results from sensitivity analysis for a change of weight of embankment.

Calculation	Settlement 4 years [m]	Ratio from SSC	Settlement 100 years [m]	Ratio from SSC	Rate 4 years	Ratio from SSC
Measured	-0,297	0,169	-		-0,004	-0,875
PLAXIS SSC	-0,254	0,000	-1,253	0,000	-0,032	0,000
PLAXIS SSC Embankment 15 kN/m3	-0,185	-0,270	-0,977	-0,220	-0,022	-0,319

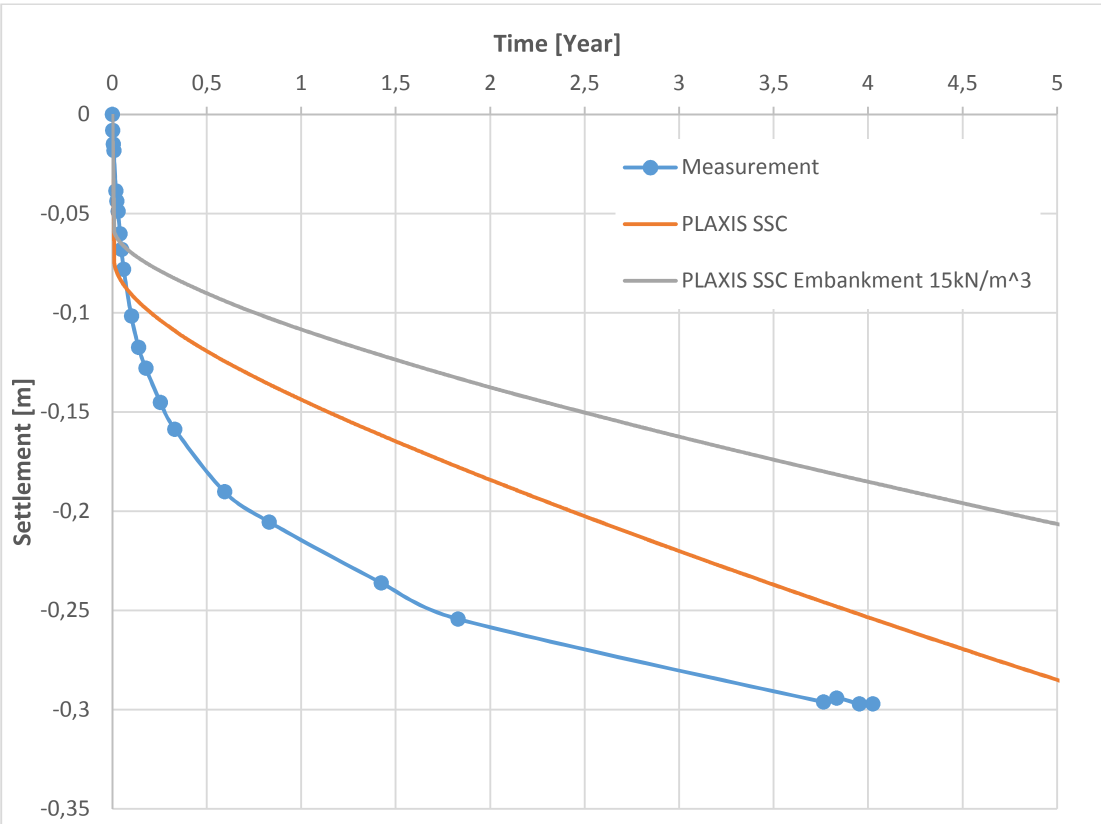

Figure 41. Settlement plotted over time.

7.2.2 Creep parameter, μ*

As seen in the results the rate of the settlements is to large, about 8 times larger than the actual value. Therefore, the μ* value is chosen to look in to how it affects the calculation.

The results from the sensitivity analysis of the μ* shows that the rate of settlement gets closer to the measured value and is decreasing. When the μ* is 50 % of the original μ* the amount of settlement between 2 to 4 years is the same as the measured.

Calculation	Settlement 4 years	Change from SSC	Settlement 100 years	Change from SSC	Rate 4 years	Change from SSC
Measured	-0,297	0,169	-		-0,004	-0,875
PLAXIS SSC	-0,254	0,000	-1,253	0,000	-0,032	0,000
PLAXIS SSC my -10%	-0,233	-0,081	-1,123	-0,104	-0,029	-0,101
PLAXIS SSC my -20%	-0,215	-0,155	-1,000	-0,202	-0,026	-0,199
PLAXIS SSC my -50%	-0,167	-0,343	-0,661	-0,472	-0,017	-0,470

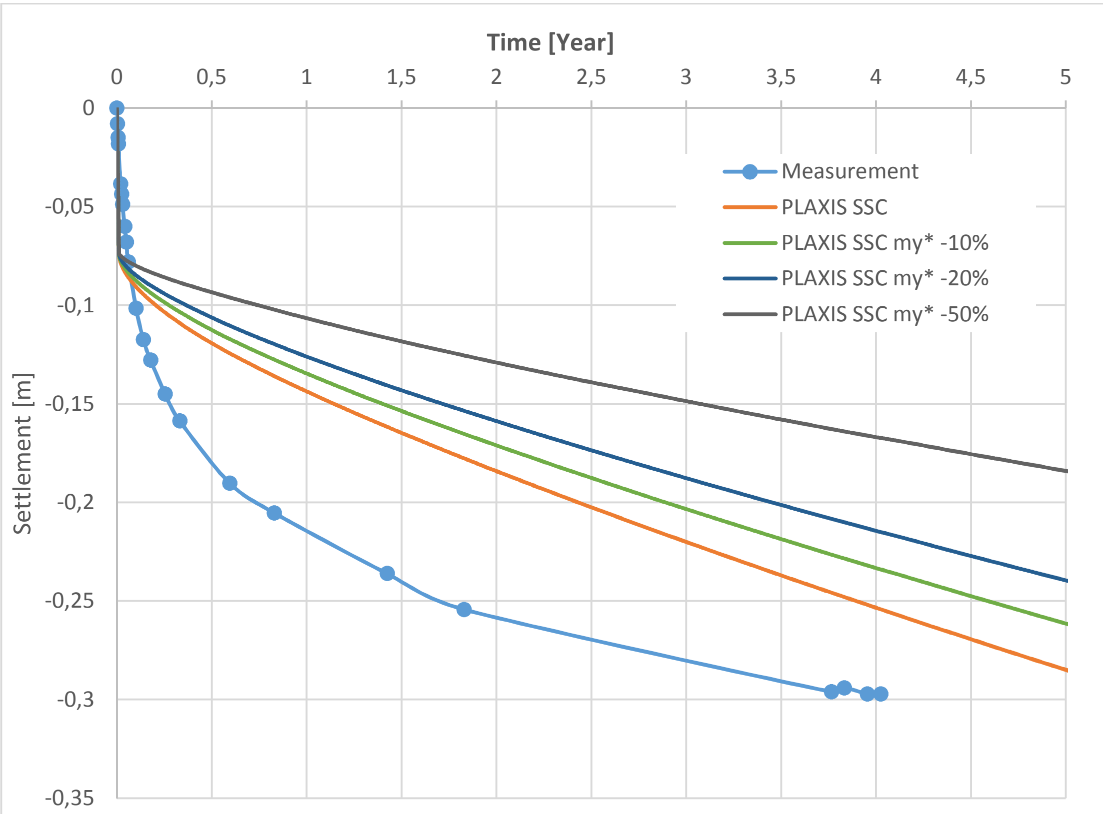

Figure 42. Settlement plotted over time when changing μ.*

7.2.3 Permeability

Since the amount of settlement over time is lower than the measured values it can be believed that the permeability of the soil is too low since it will increase the speed the consolidation.

The result from the settlement analysis show that the speed is increasing and so is the rate of settlement. However the total settlement is almost unchanged after 100 years.

Table 110. Results from the sensitivity analysis when changing the permeability.

Calculation	Settlement 4 years [m]	Change from SSC	Settlement 100 years [m]	Change from SSC	Rate 4 years [m/year]	Change from SSC
Measured	-0,297	0,169	-		-0,004	-0,875
PLAXIS SSC	-0,254	0,000	-1,253	0,000	-0,032	0,000
PLAXIS SSC k*2	-0,342	0,347	-1,351	0,078	-0,048	0,507
PLAXIS SSC k*3	-0,411	0,619	-1,380	0,101	-0,059	0,833
PLAXIS SSC k*5	-0,518	1,037	-1,401	0,118	-0,072	1,252

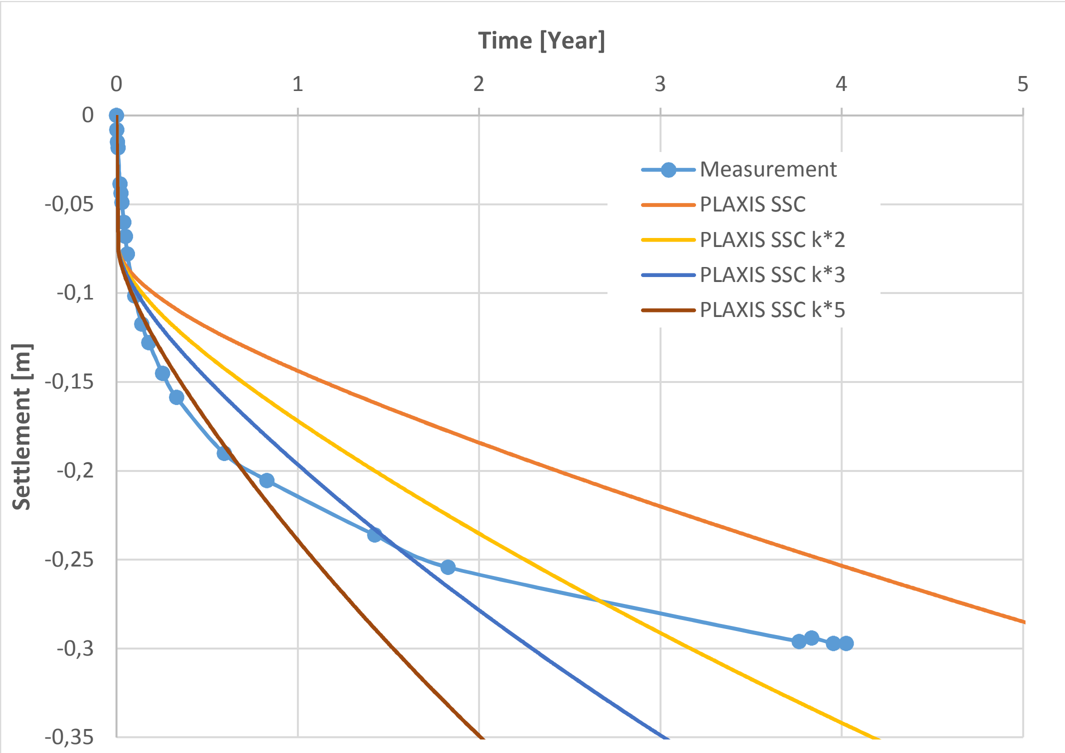

Figure 43. Settlement over time when changing the permeability.

7.2.4 OCR +10%

Since the total amount of settlement after 100 years is too high in the results, the OCR can been too low, therefore a sensitivity analysis have been done where the OCR have been increased.

Calculation	Settlement 4 years[m]	Change from SSC	Settlement 100 years [m]	Change from SSC	Rate 4 years [m/year]	Change from SSC
Measured	-0,297	0,169	-		-0,004	-0,875
PLAXIS SSC	-0,254	0,000	-1,253	0,000	-0,032	0,000
PLAXIS OCR +10%	-0,186	-0,267	-0,960	-0,234	-0,023	-0,268

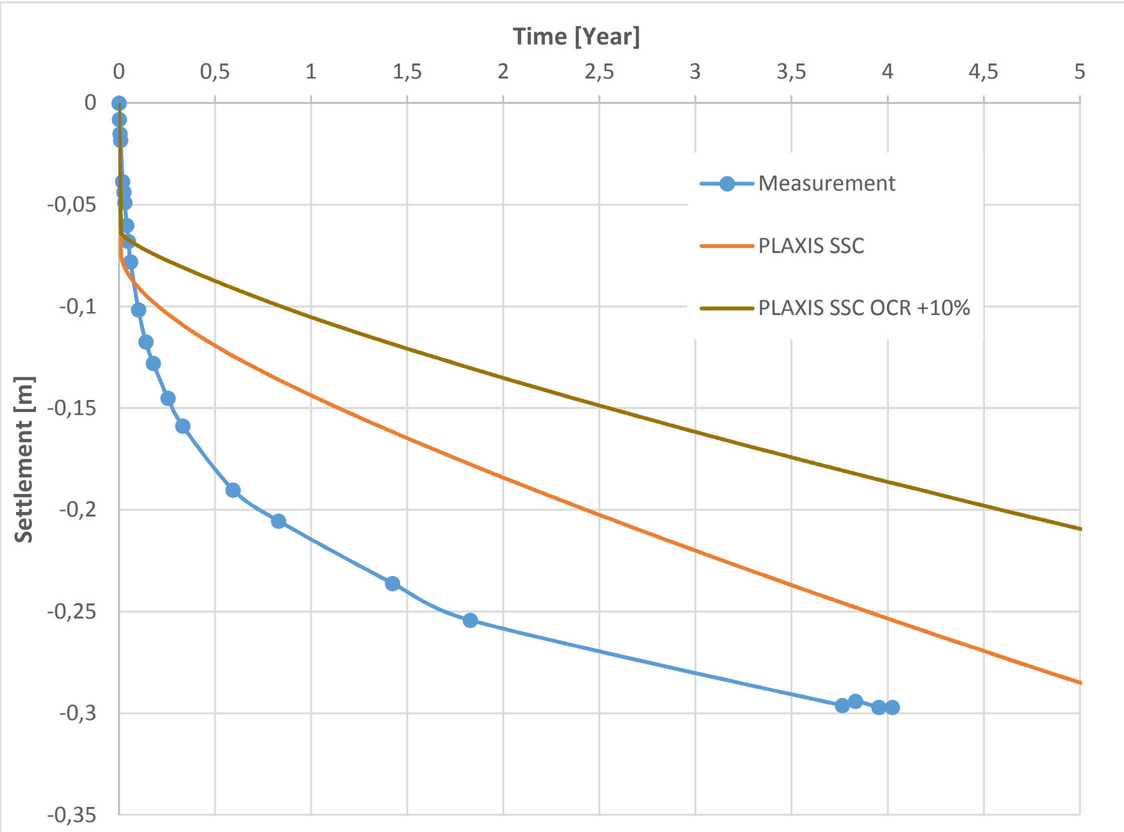

Figure 44. The effect of a change of the OCR.

7.2.5 6 noded model

In PLAXIS there is a 6 noded model instead of the used 15 noded analysis, a calculation have been done with the same parameters as in the original 15 noded calculation.

The results tells us that the settlement in 6 noded is happening faster but the total settlement after 100 years is unchanged compared to the 15 noded model.

Calculation	Settlement 4 years [m]	Change from SSC	Settlement 100 years [m]	Change from SSC	Rate 4 years [m/year]	Change from SSC
Measured	-0,297	0,169	-		-0,004	-0,875
PLAXIS SSC	-0,254	0,000	-1,253	0,000	-0,032	0,000
PLAXIS 6 noded	-0,541	1,131	-1,287	0,027	-0,060	0,880

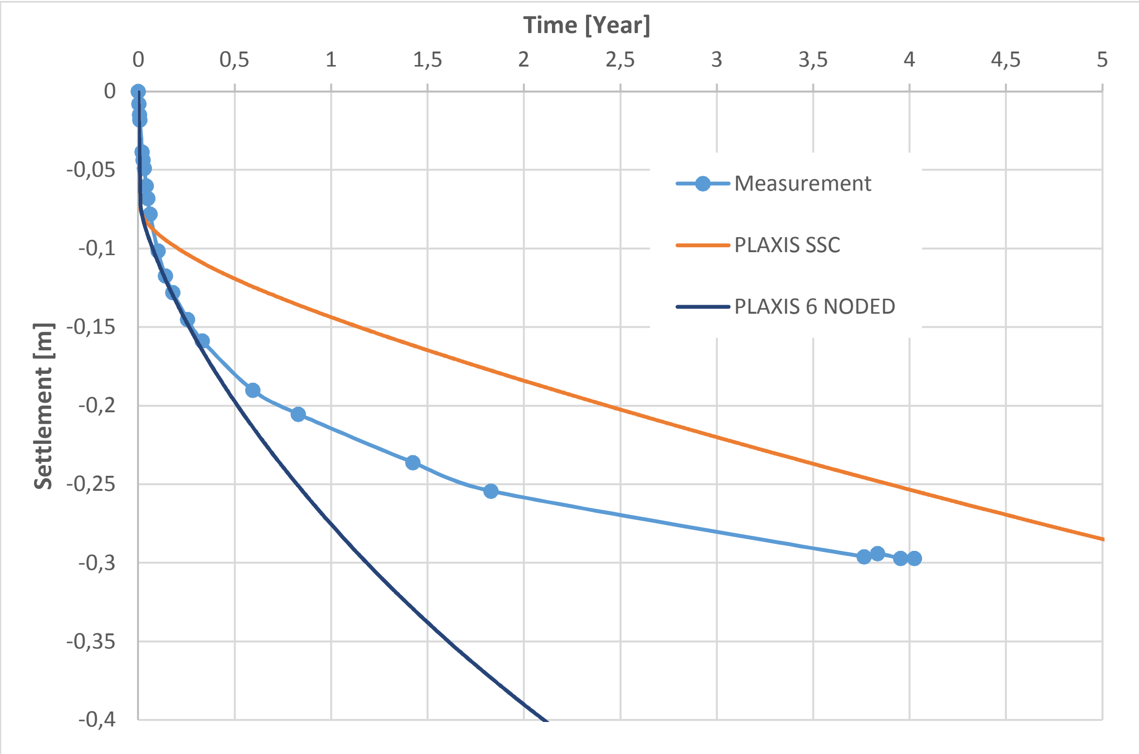

Figure 45. The effect of using a 6 noded model.

8 Discussion

This chapter will discuss the contents of the report and summaries the conclusions

8.1 Discussion of method and in-data

The CRS test from borehole 16w1 at 9,5 meters is excluded when validating the σ'_c of the soil since it does not follow the chosen trend line of the parameters that comes from the CRS test, however it not believed that the complete sample is disturbed, only that particular test. This is probably caused of a disturbance such a sand gland in the small sample that goes in to the CRS test.

It is assumed that creep only exists in the first 13 meters of clay when calculating with GS. It origins from the Swedish praxis that creep only exist if the effective stress is higher than 80% of the pre consolidation pressure. However there are ageing effects and self-compacting effects that can occur even if the stress levels are lower than 80 % of the preconsolidation pressures.

The soil in the test-embankment was assessed during the field day seemed to have lots of boulders and big stones that could increase the mean soil weight of the embankment. This underpins the importance of a good control of the weight of the material that builds up the pressure bank.

The back calculation done in the PLAXIS SoilTest compared to the CRS test show that the selected parameters corresponds rather well to each other. λ^* corresponds well with the CRS test, κ^* could be altered slightly in order to simulate a stronger soil. It was noticed by the authors that a higher c_{ref} also could produce a matching curve, this could origin from the high silt content in the clay at the test-embankment.

Comparing the soil parameters used in PLAXIS and GeoSuite shows that the in-data used in PLAXIS represents a softer soil when comparing the correlation between M_0 and κ^* for the first 8 meters of soil. When comparing the parameter for the deeper soil layers the parameters used in PLAXIS corresponds to a stiffer soil compared to the ones used in GeoSuite. The $r_1/\ \mu^*$ and especially the M_L/λ^* value corresponds well to each other. Is should however be noted that the transformation only gives an estimate for comparing the different in-data.

The IL test is only done for 3 different samples since time restrictions and the time it takes to perform an IL test. More loading steps could have been used and especially when close to the preconsolidation pressures and the unloading-reloading loops.

When constructing a test-embankment it is important to think about what parameters that is being monitored and if they provide the necessary information. Ground Peglar at ground level only tells the total settlement and provides no information about the spread of settlements threw the soil layers. In order to get a better understanding of soil behaviour the change in pore pressure with depth, is something that also is interesting to monitor. These instruments could be quite costly to install. There are different parties that could be interested in the results from a test-embankment, it could then be a good idea to consult universities and researchers in order to split costs.

8.2 Discussion of the results and sensitivity

The results from the hand-calculation using Asaokas and the hyperbolic method shows that the embankment, will soon reach the total settlement point. This means that the soil is settling at a constant rate and is only driven by pure creep. When using Asaoka's and the hyperbolic fixed time step should be used for the continuously monitored data to improve the accuracy of the calculations.

The results from the calculations software is varied and it is found it is hard to predict accurate predictions from the collected data sets. In the GeoSuite with Chalmers Creep model the rate of settlement is very similar between 2 and 4 years compared the measured values, however the settlements is too low for the first 2 years. So in GeoSuite the values for the creep is probably right.

In the PLAXIS software the total settlements is the closest predicted when using the Soft soil model, and with the Soft soil Creep model the total amount of settlement is about 4 times bigger than calculated with Asaokas method. Also the dissipation of the pore pressures is really slow compared to what could be assumed from measured and calculated data from Asaokas and hyberbolic method, since only about 30% of the pore pressures have dissipated after 4 years when calculating in PLAXIS.

In Table 8 it is shown that the maximum stress levels that is calculated in PLAXIS is not the same than in the what could expected when comparing the σ_c from 16w1. In the effort to find an explanation, three different direction could be identified; either is the σ_c overestimated or PLAXIS underestimates the loads, or the soil has been exposed to an earlier even bigger load. This could be the case since the BH1 σ_c samples are similar to the ones from 16w1, however the 16w2 results contradicts this, (Figure 23).

There are many uncertainties in this project and one especially is the permeability, since it can be believed that the mean permeability shall be higher through the soil profile since it is possibly a sand layer at 13 meter that shorten the drainage ways, and there could also occur small layers of sand that increase the permeability and is still unnoticed from the tests in the soil. In both the programs the rate of settlements increases when increasing the permeability but the total amount of settlement remains about the same.

The embankment weight is another uncertainty that should be evaluated more carefully, and the sensitivity results shows that the load have a great influence on the results in both programs.

The creep behaviour in PLAXIS doesn't correlate well with the measured values, it is considerably larger. The sensitivity analysis shows that a change in μ^* gives a result that corresponds better to the measured values, the rate at which the settlements decrees are still not representative for the measured results. The equation used in SSC to calculate the creep factor, $(\lambda^* - \kappa^*)/\mu^*$, gives that a change to a higher κ^* also produce less creep settlement increases the amount of settlement from consolidation in the same time.

When using the 6 noded calculation type, the settlement become very different and looks similar to the result that occurs when performing a sensitivity analysis of the permeability, and especially when the permeability is 5 times the original value.

8.3 Conclusion

As seen from the results and in Figure 46 it is hard to make accurate settlement analysis in both GS and PLAXIS, both the GeoSuite and PLAXIS results are differ in degree of settlement after 4 years and for the total amount. However GeoSuite gives a good estimate of the rate of settlement between year 2 to year 4. It is assumed that Asoka's and hyperbolic method is a good way to predict total settlements if there are continuously monitored data samples.

This report once again show that the influence of the choice that a geo-engineer does when validating the soil parameters is important and good knowledge of the soil history is important, also it good to understand how the calculation program implement the in-data and understand the sensitivity of the results.

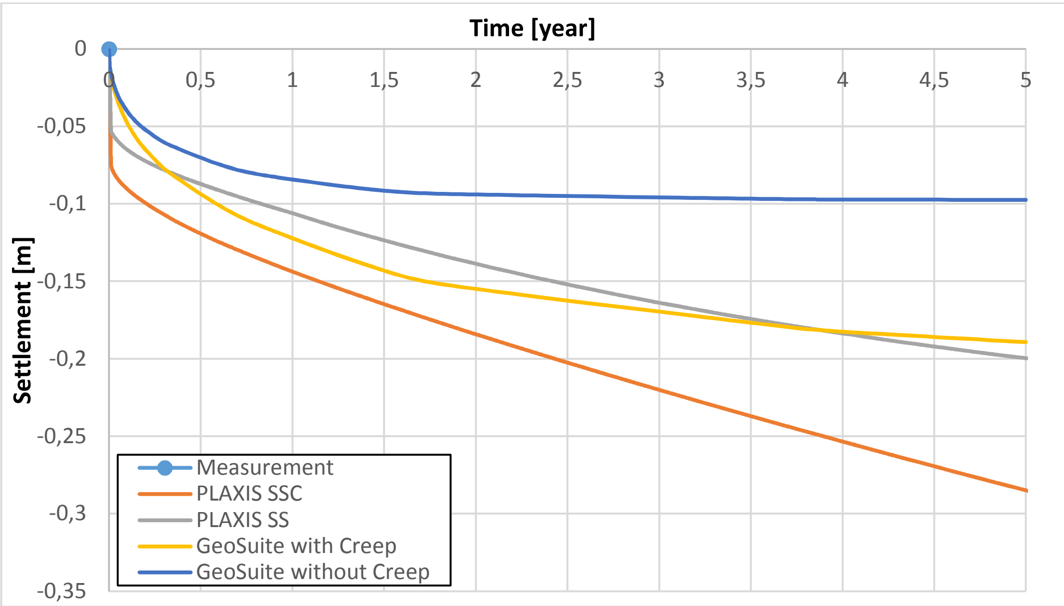

Figure 46. The result from the PLAXIS and GeoSuite.

Finally it should be recommended that when building an embankment it is essential to include more monitoring devices. Measuring pore pressure, and measuring the settlement of different layers makes it possible to understand where the predicted displacement is occurring.

9 Further studies

Using more advanced models that use structure, such as S-CLAY 1 or similar to perform further studies on the test embankment, to capture bonding and structure effects in the soil, however triaxial tests could be needed to get a functional model if no empiric relationship.

Investigate the differences of a Gothenburg clay and the clays around Varberg and especially the influence of the silt content.

Further research could contribute with significant results when using a 6 noded model in PLAXIS and perform a 3D analysis in software such as PLAXIS 3D and GeoSuite 3D to better capture the geometry of the embankment.

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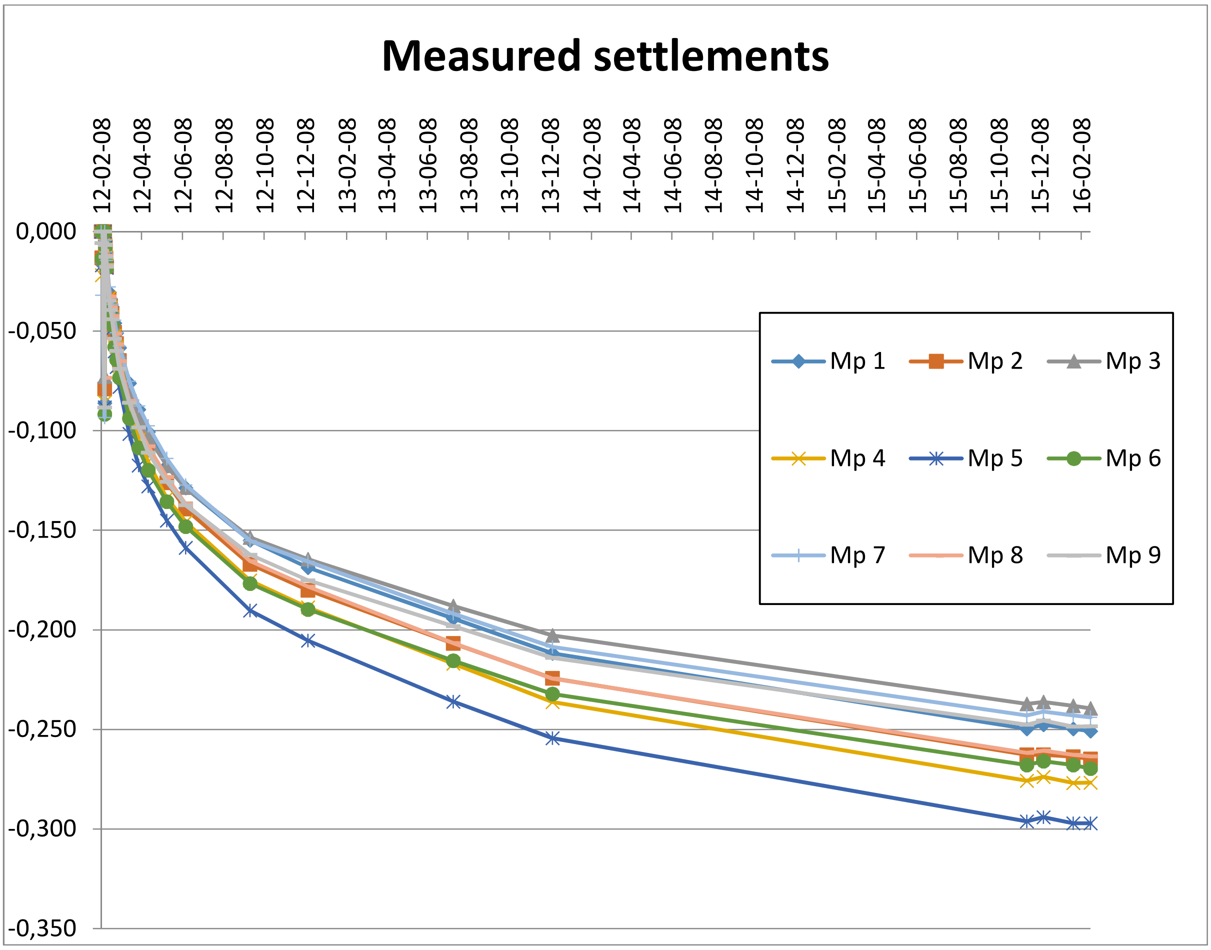
Appendix 1



Settlement of under the embankment

Appendix 2 Conrad 16w1

CPT-sondering utförd enligt EN ISO 22476-1

Förborrningsdjup	0.01 m	Referens		Vätska i filter		Projekt
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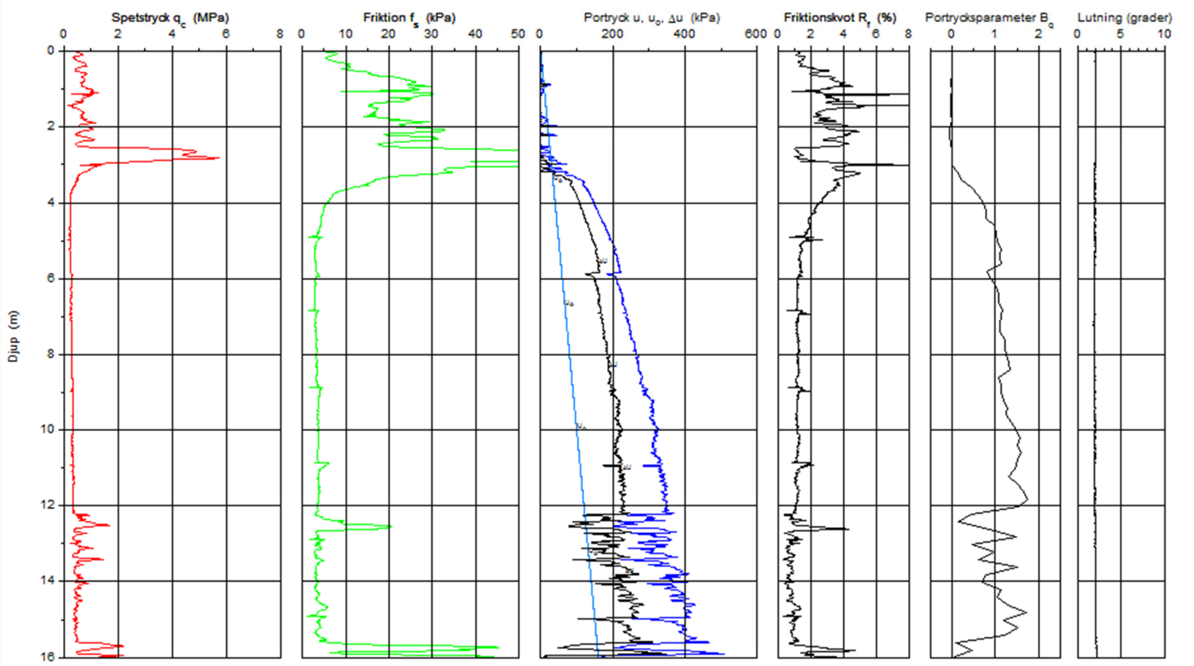
						Datum 20160314



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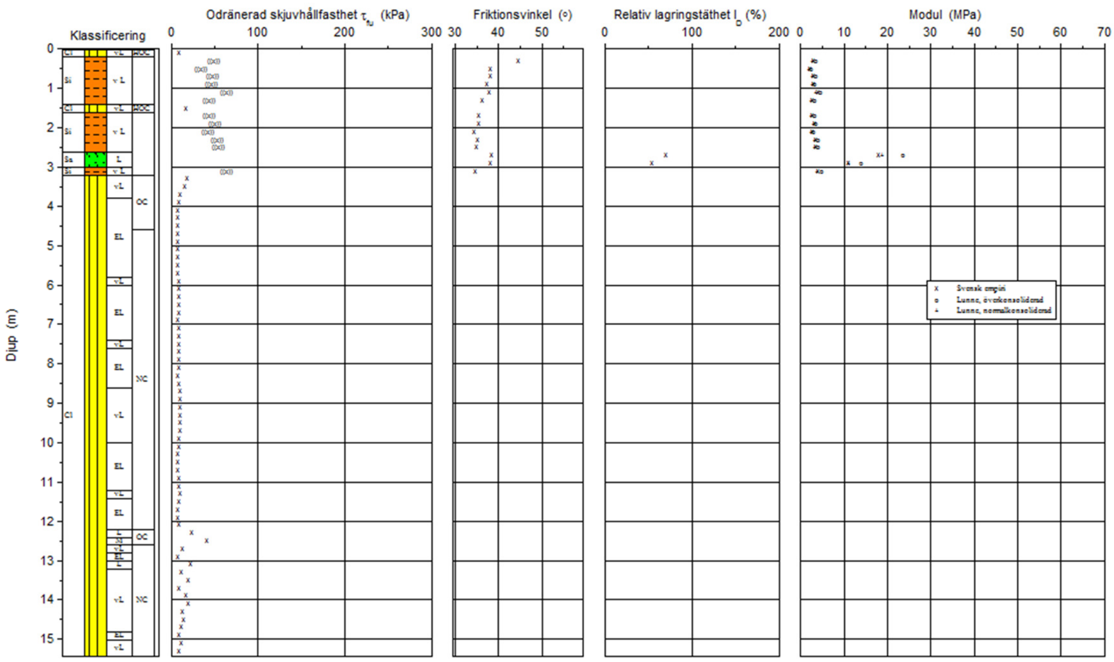
Startdjup	0.01 m	Geometri	Normal		Borrhål 1716
					Datum 20160314



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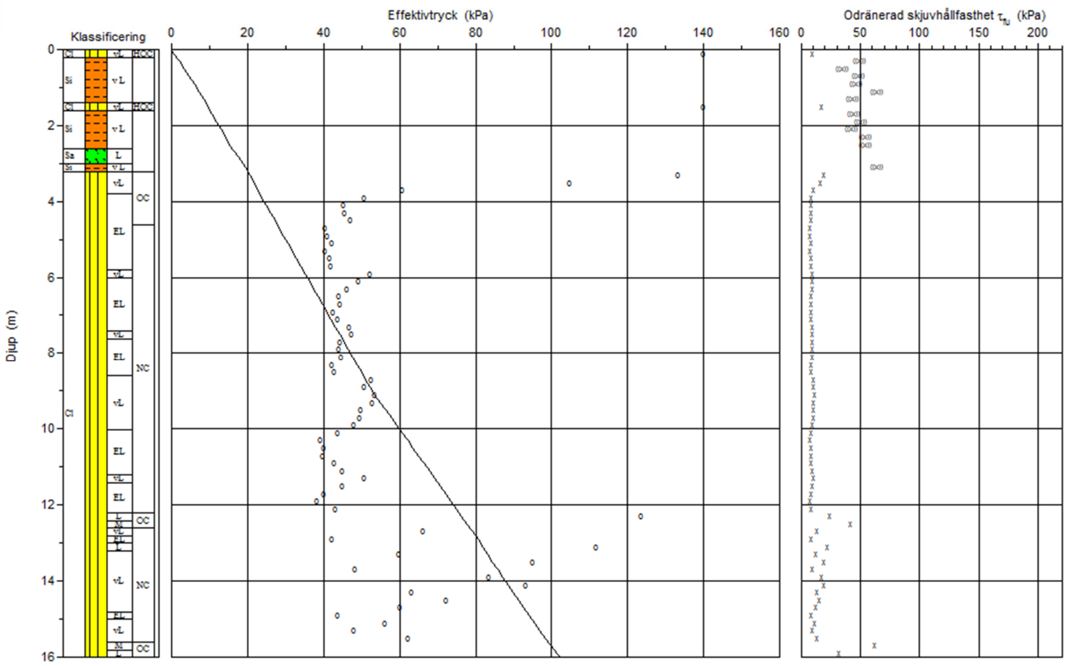
					Datum 20160314



Appendix 2 Conrad 16w2

CPT-sondering utförd enligt EN ISO 22476-1

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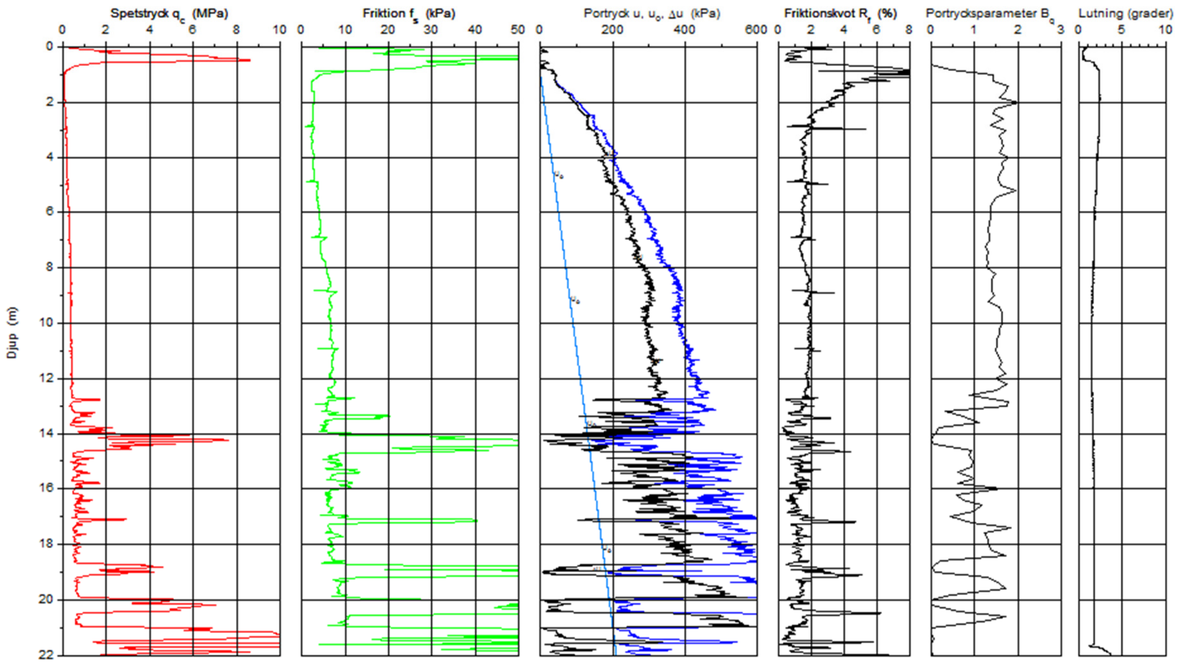
						Datum 20160314



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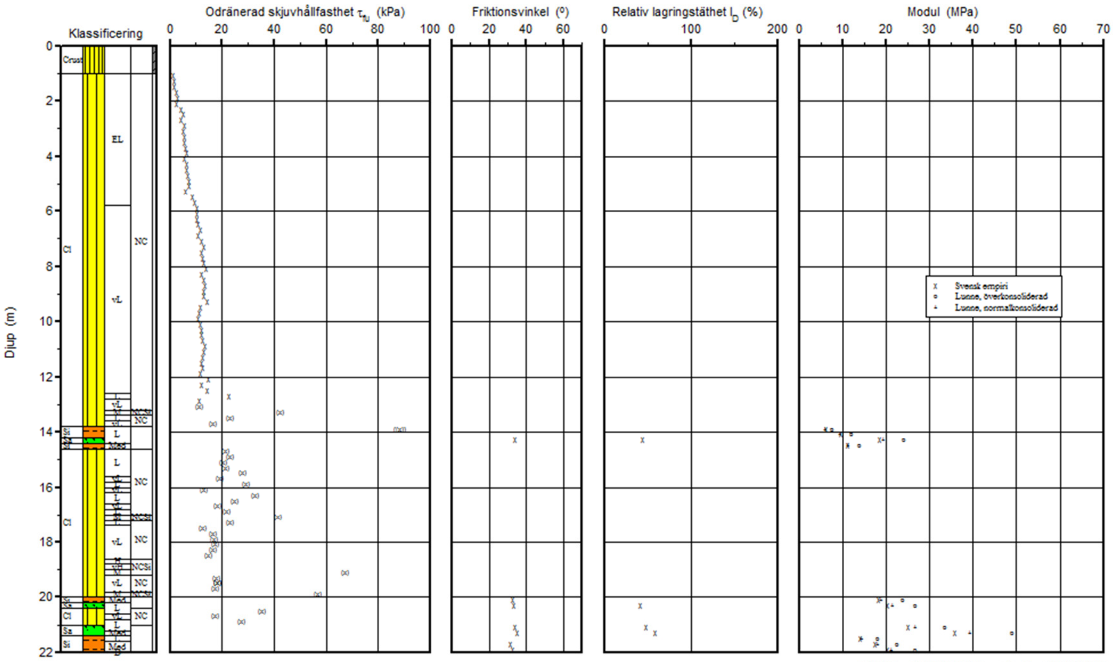
Startdjup 0.01 m	Geometri Normal		Borrhål 1721
			Datum 20160314



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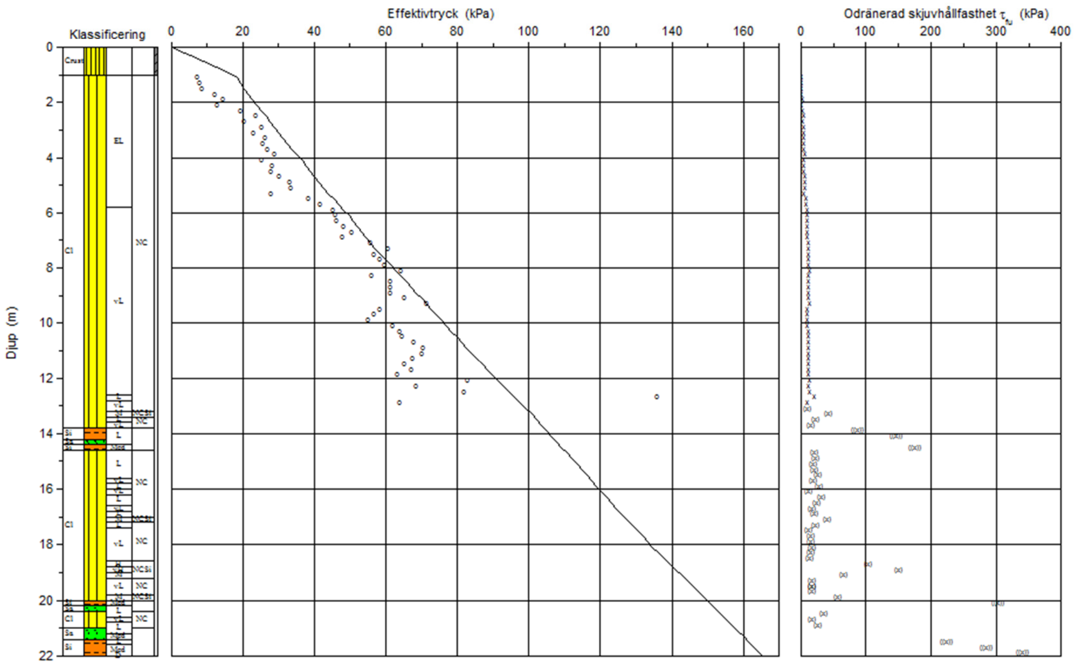

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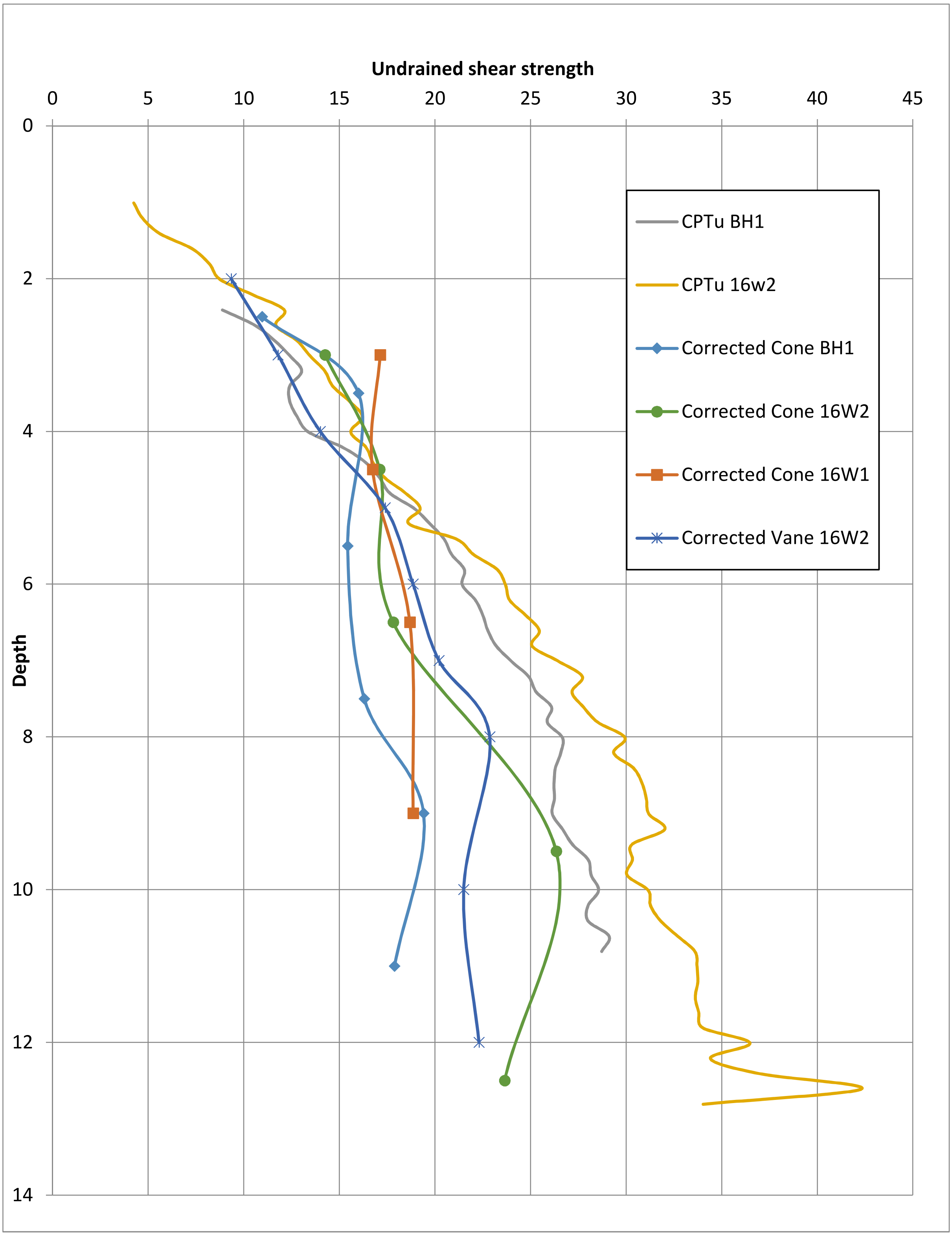
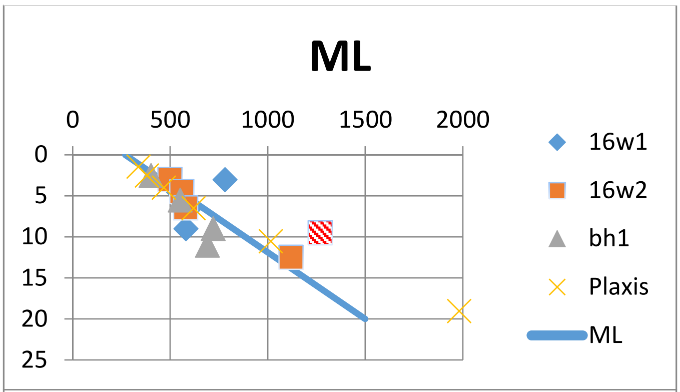
Appendix 3 Soil parameter compilation

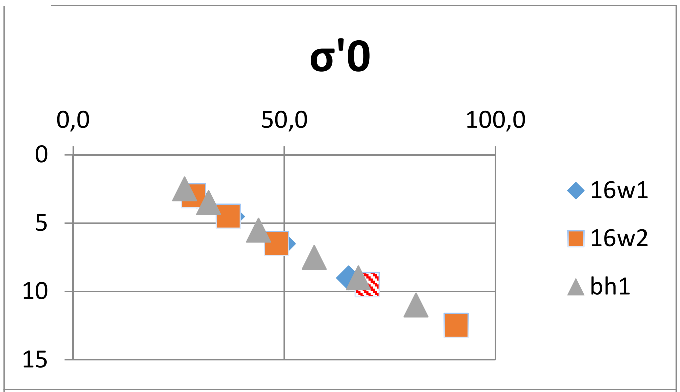
																														σ'0 + Δσ < σ'c		σ'0 + Δσ > σ'c	
Borrhål	Depth	Soil type	soil weight		ML	M'	M0(σ'c)	M0 (Cu)	σ'0	σ'c	σ'L	Δσ	Wn	WL	Cu	μ	OCR	OCR(conra	St	τfu	τt	Kint (m/y)	Kint (m/day)	Kint (m/s)	βk	a0	a1	b0	b1	r0	r1	r0	r1
16w1	3	Le	1.61		780	12.4	3750	5996	29.3	75	132	38.29	68	48	17.13	0.95	2.56		46	18	0.39	2.84E-02	7.78E-05	9.00E-10	5	0.8	1	0.390	1.1	11034	260	2354	226
16w1	4.5	su Le	1.59		540	13.1	4700	5858	37.7	94	133	32.89	67	57	16.74	0.88	2.49		84	19	0.23	3.15E-02	8.64E-05	1.00E-09	4.7	0.8	1	0.401	1.1	164282	144	2271	174
16w1	6.5	su Le	1.63		600	13.7	5200	6541	49.7	104	148	27.31	61	50	18.69	0.93	2.09		63	20	0.31	3.15E-02	8.64E-05	1.00E-09	5.3	0.8	1	0.478	1.1	-8010554	144	2078	212
16w1	9	su Le	1.65		580	13.7	5300	6600	65.2	106	143	22.12	63	49	18.86	0.94	1.63		79	20	0.25	2.21E-02	6.05E-05	7.00E-10	4.5	0.8	1	0.615	1.1	11396	137	1673	219
16w2	3	le	1.57		500	11.5	3200	5423	28.5	64	123	38.29	73	60	15.49	0.86	2.25	2.27	32	18	0.55	9.46E-02	2.59E-04	3.00E-09	6.4	0.8	1	0.445	1.1	5192	195	2126	161
16w2	4.5	le	1.58		560	12.2	3350	6280	36.8	67	128	32.89	77	61	17.94	0.85	1.82	1.78	33	21	0.63	6.31E-02	1.73E-04	2.00E-09	5.4	0.8	1	0.549	1.1	4045	209	1811	157
16w2	6.5	su le	1.6		580	13.3	4800	6280	48.2	96	142	27.31	71	61	17.94	0.85	1.99	1.90	26	21	0.83	4.10E-02	1.12E-04	1.30E-09	5.4	0.8	1	0.502	1.1	28746	151	1951	157
16w2	9.5	susiLe sa	1.75		1270	15.2	6700	9944	69.7	134	216	21.26	49	45	28.41	0.98	1.92	1.86	22	29	1.31	1.58E-02	4.32E-05	5.00E-10	4.6	0.8	1	0.520	1.1	-19927	237	1987	248
16w2	12.5	susasiLe	1.73		1120	14.5	6200	9463	90.7	124	189	17.02	26	33	27.04	1.13	1.37	1.32	18	24	1.31	2.21E-02	6.05E-05	7.00E-10	5.3	0.8	1	0.732	1.1	5044	226	1501	396
bh1	2.5	le	1.61		400	10.7	3050	4177	26.4	61	82	40.39	63	52	11.93	0.92	2.31	2.43	19	13	0.65	6.31E-02	1.73E-04	2.00E-09	5	0.8	1	0.432	1.1	4579	164	2203	200
bh1	3.5	le	1.6					5772	32.1			36.36	52	46	16.49	0.97			39	17	0.43					0.8	1		1.1				
bh1	5.5	su le	1.62		550	13.8	5100	5834	43.9	102	139	29.91	66	51	16.67	0.93	2.32	2.32	65	18	0.27	4.10E-02	1.12E-04	1.30E-09	3.9	0.8	1	0.430	1.1	-91455	135	2215	206
bh1	7.5	le	1.69					5996	57.1			25.03	64	48	17.13	0.95			67	18	0.27					0.8	1		1.1				
bh1	9	susiLe	1.73		720	15.1	5400	6721	67.6	108	145	22.12	56	42	19.20	1.01	1.60	1.57	58	19	0.32	2.52E-02	6.91E-05	8.00E-10	5.6	0.8	1	0.626	1.1	10256	167	1698	276
bh1	11	susiLe	1.71		690	16	6000	6451	81.2	120	157	18.96	55	46	18.43	0.97	1.48	1.45	49	19	0.39	1.26E-02	3.46E-05	4.00E-10	3.9	0.8	1	0.676	1.1	8275	144	1511	240

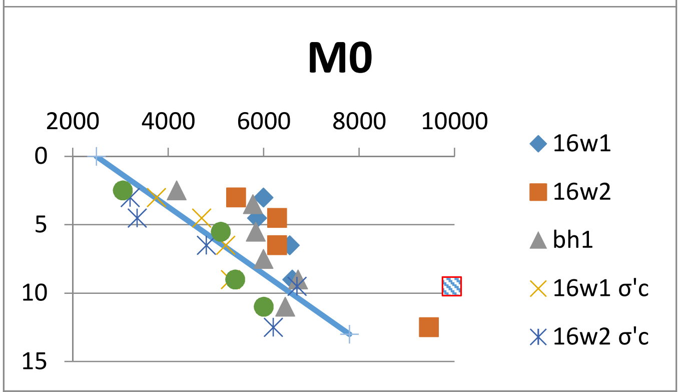
Appendix 4.1 Plotted soil parameters

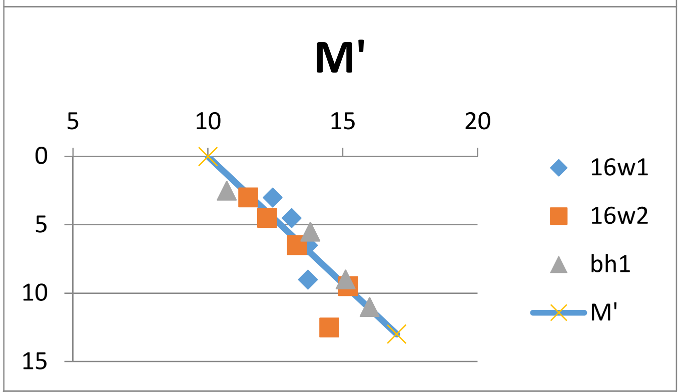


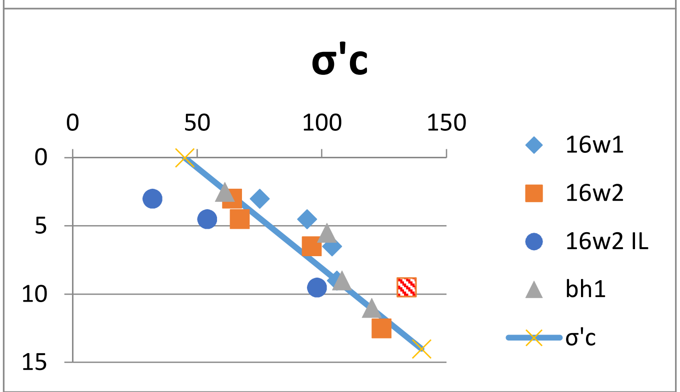
Appendix 5.2 Plotted soil parameters

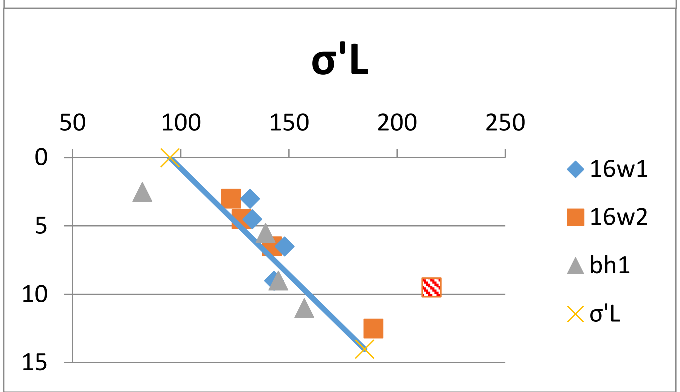


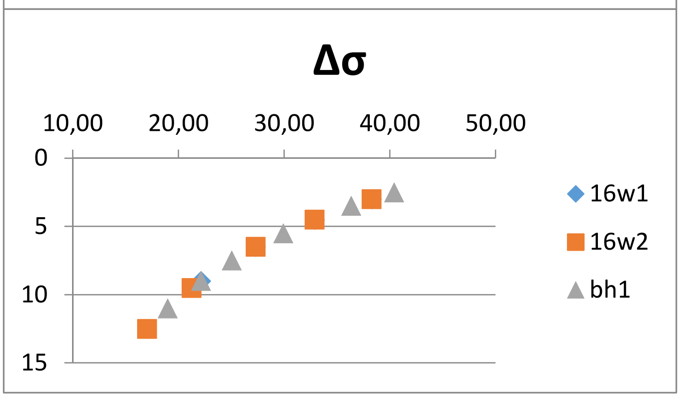


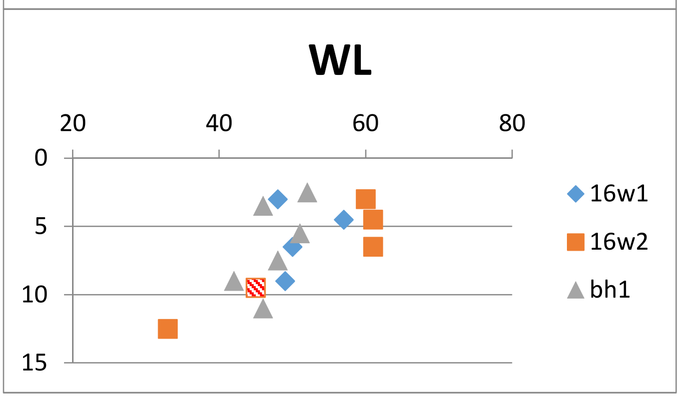




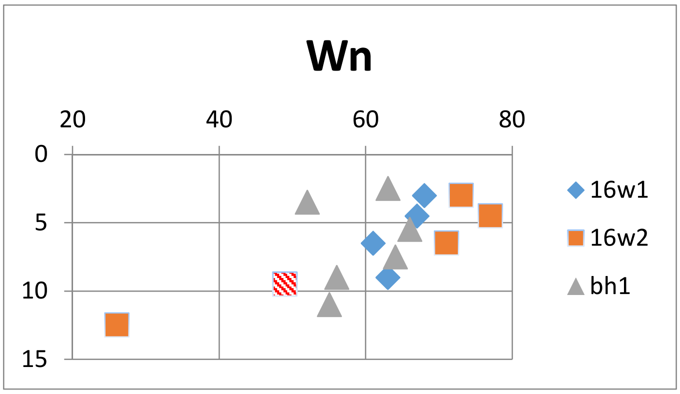


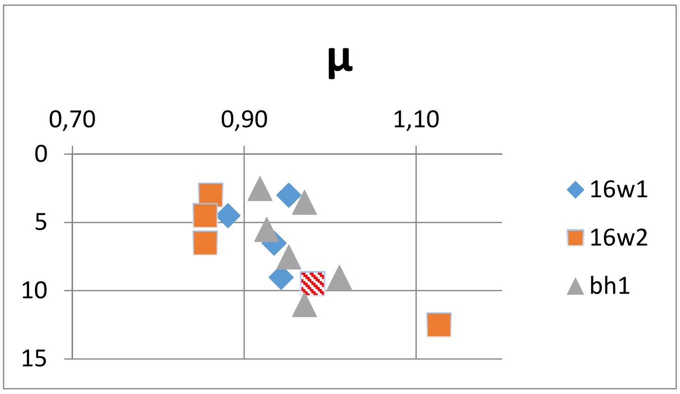


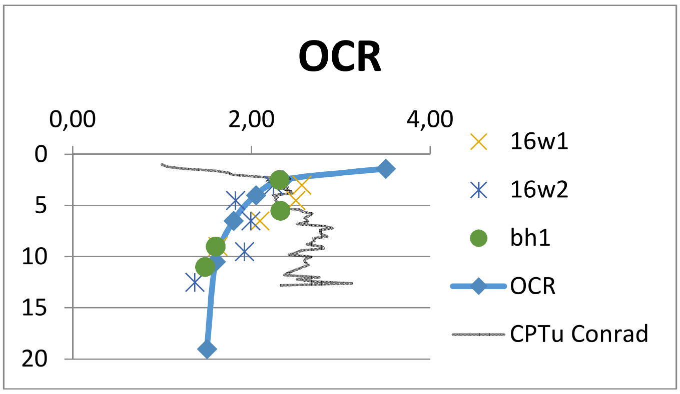


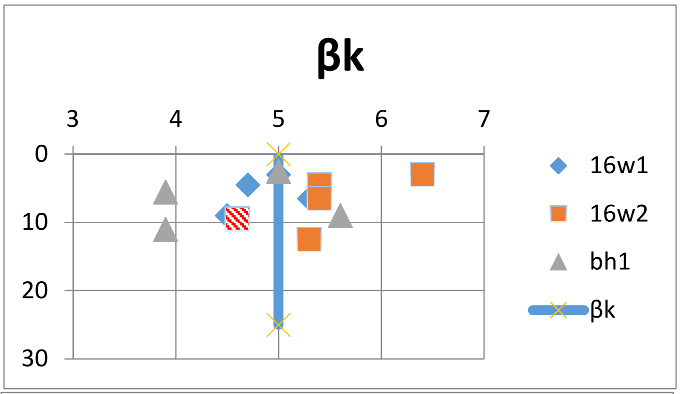


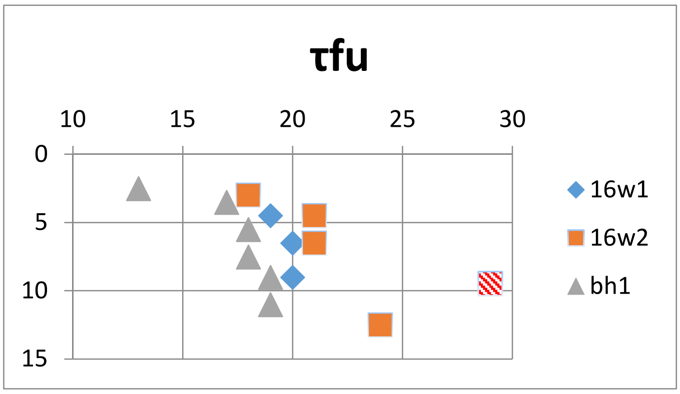
Appendix 4.3 Plotted soil parameters

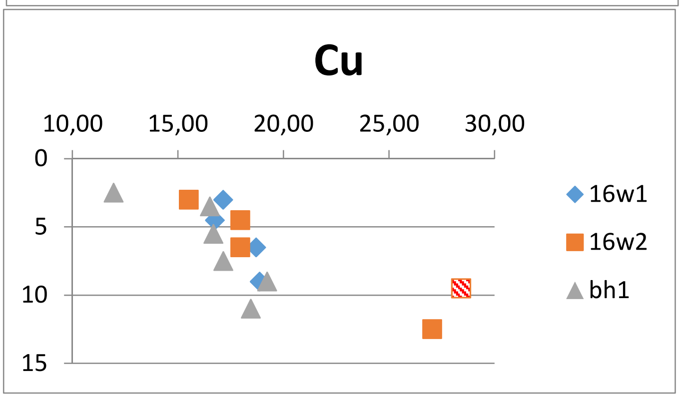


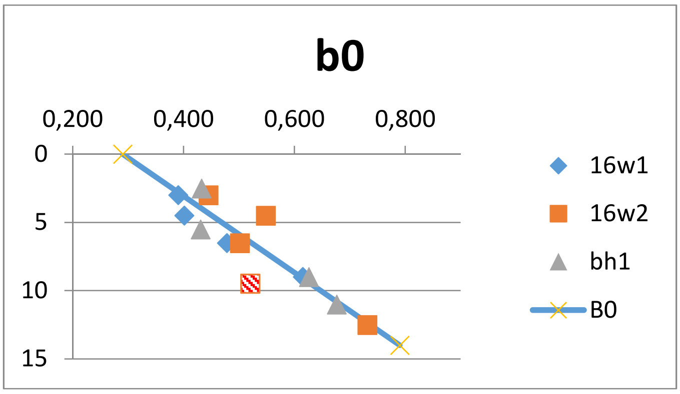


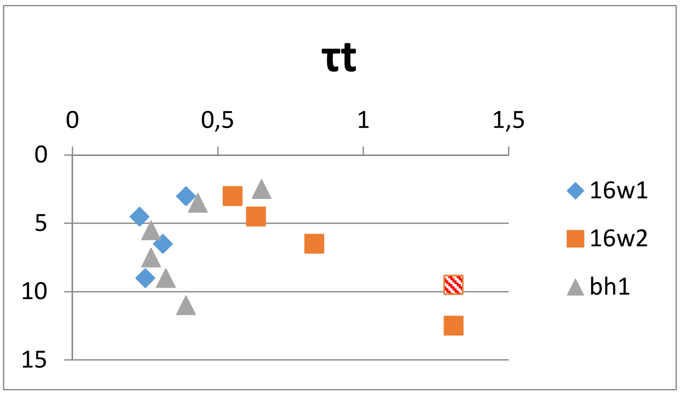




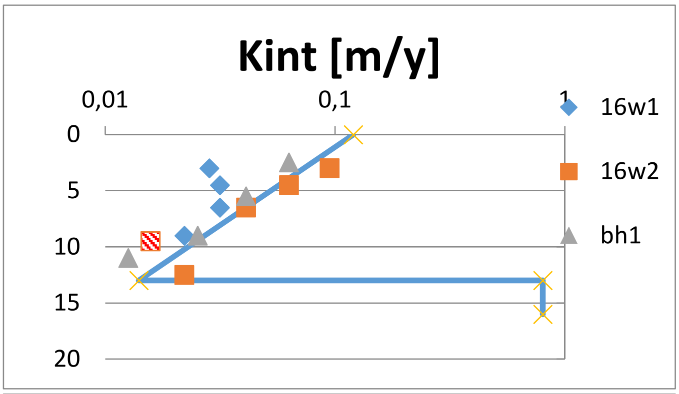


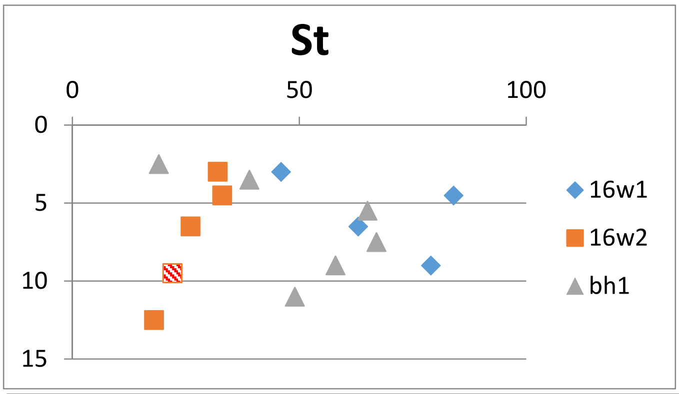


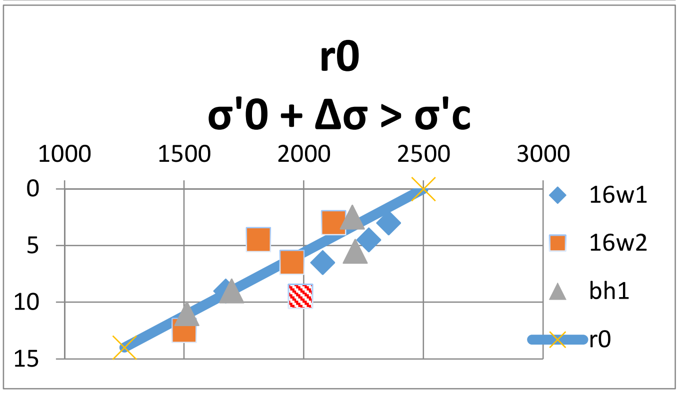


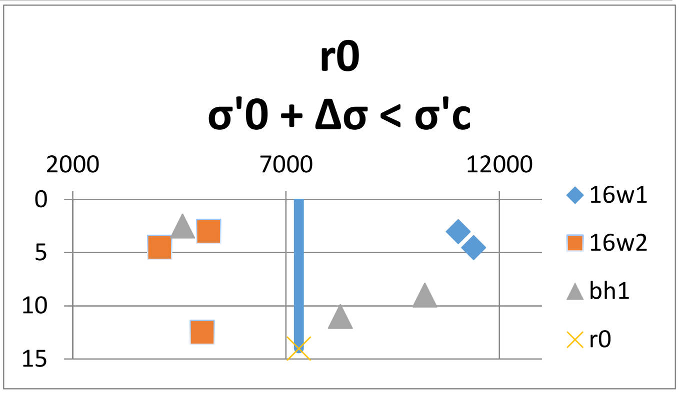


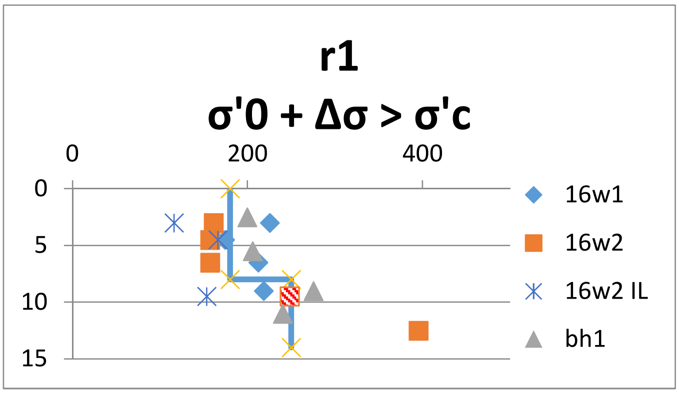
Appendix 4.4 Plotted soil parameters

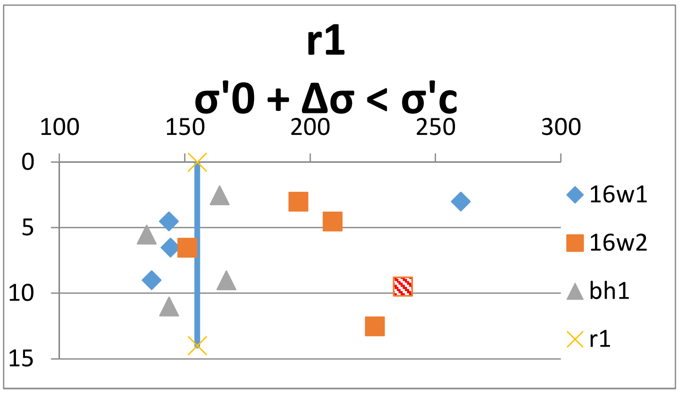


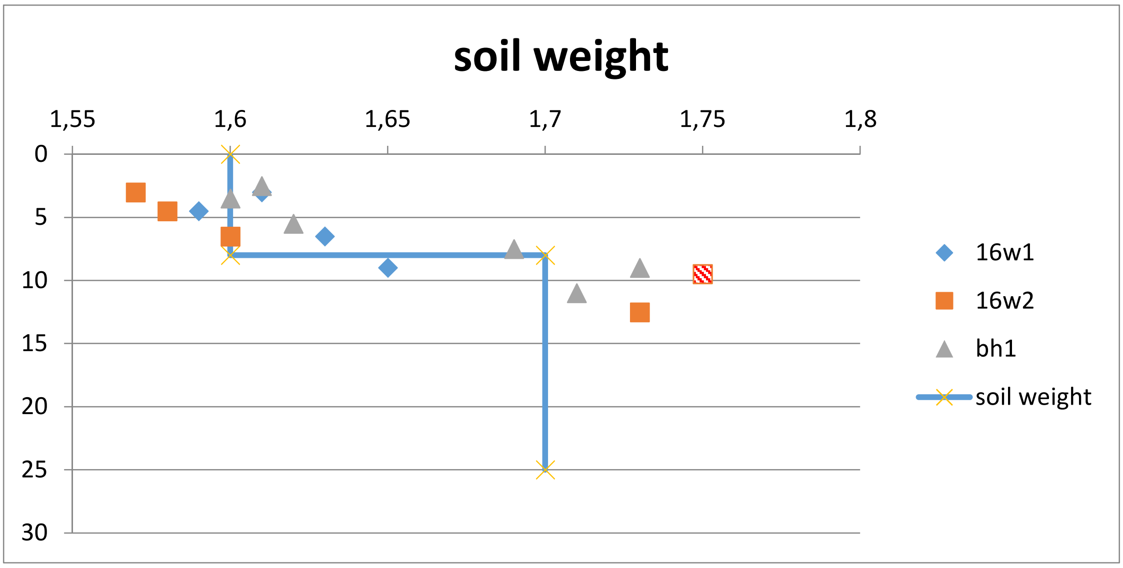




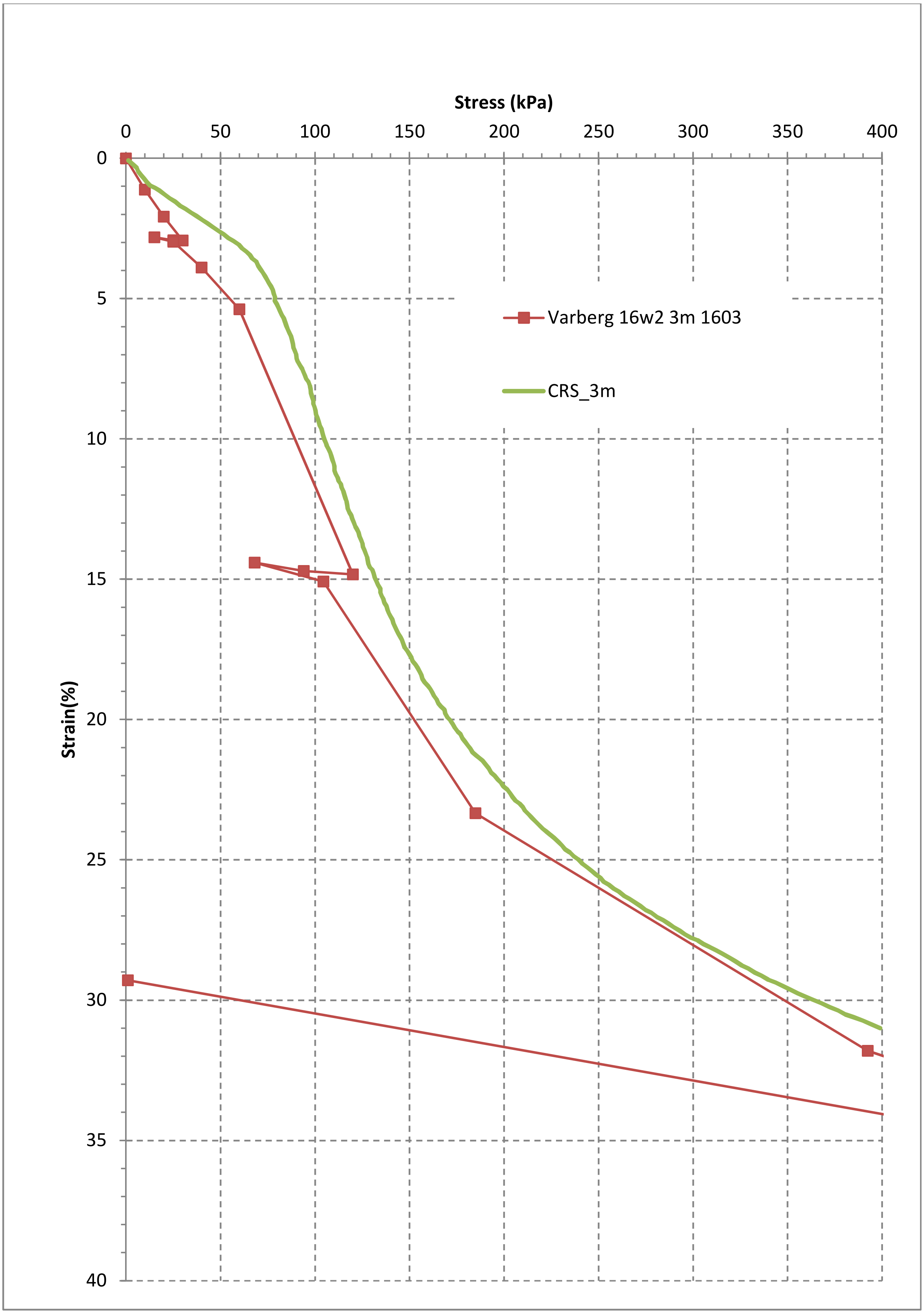






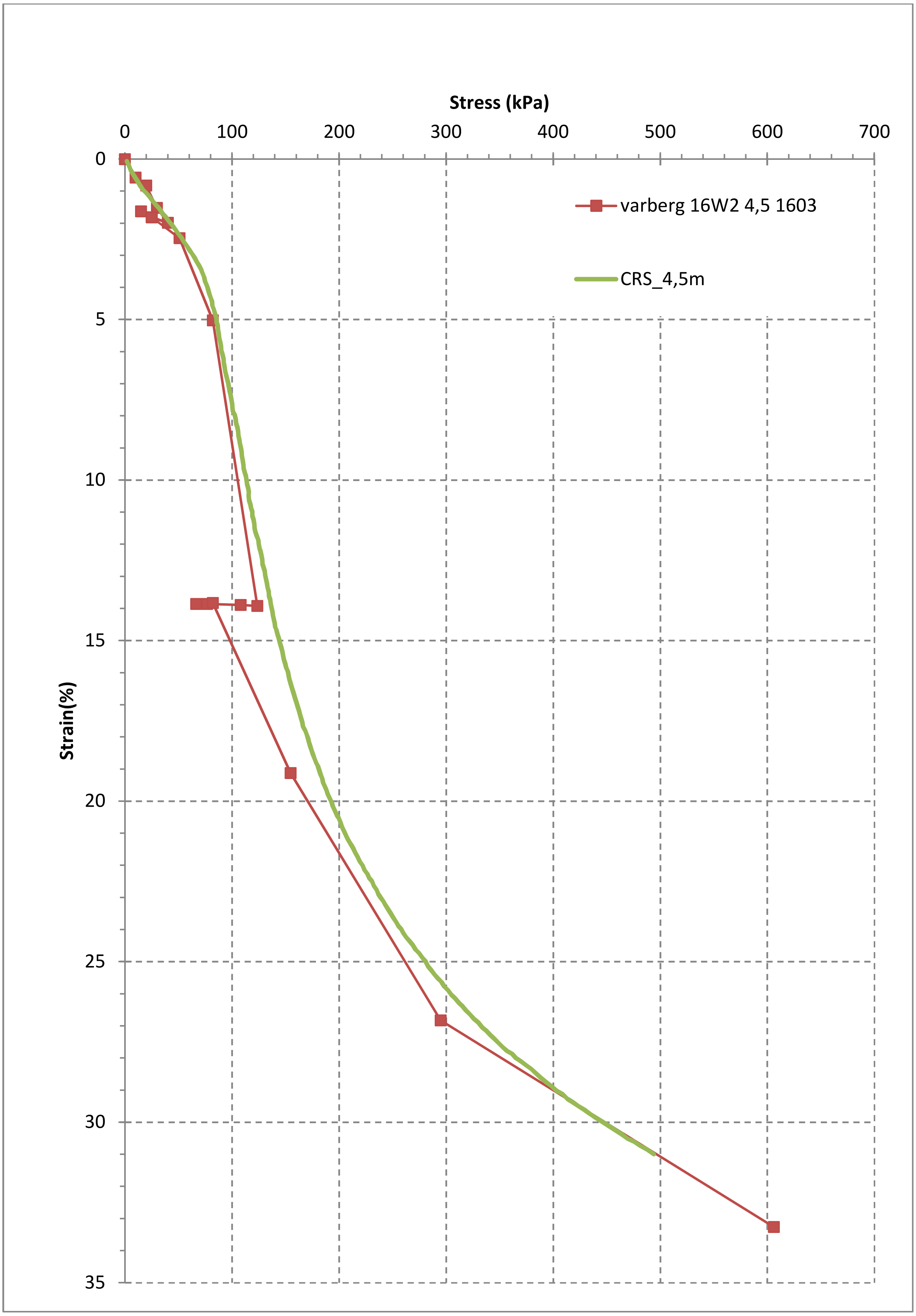


Appendix 5.1 IL results



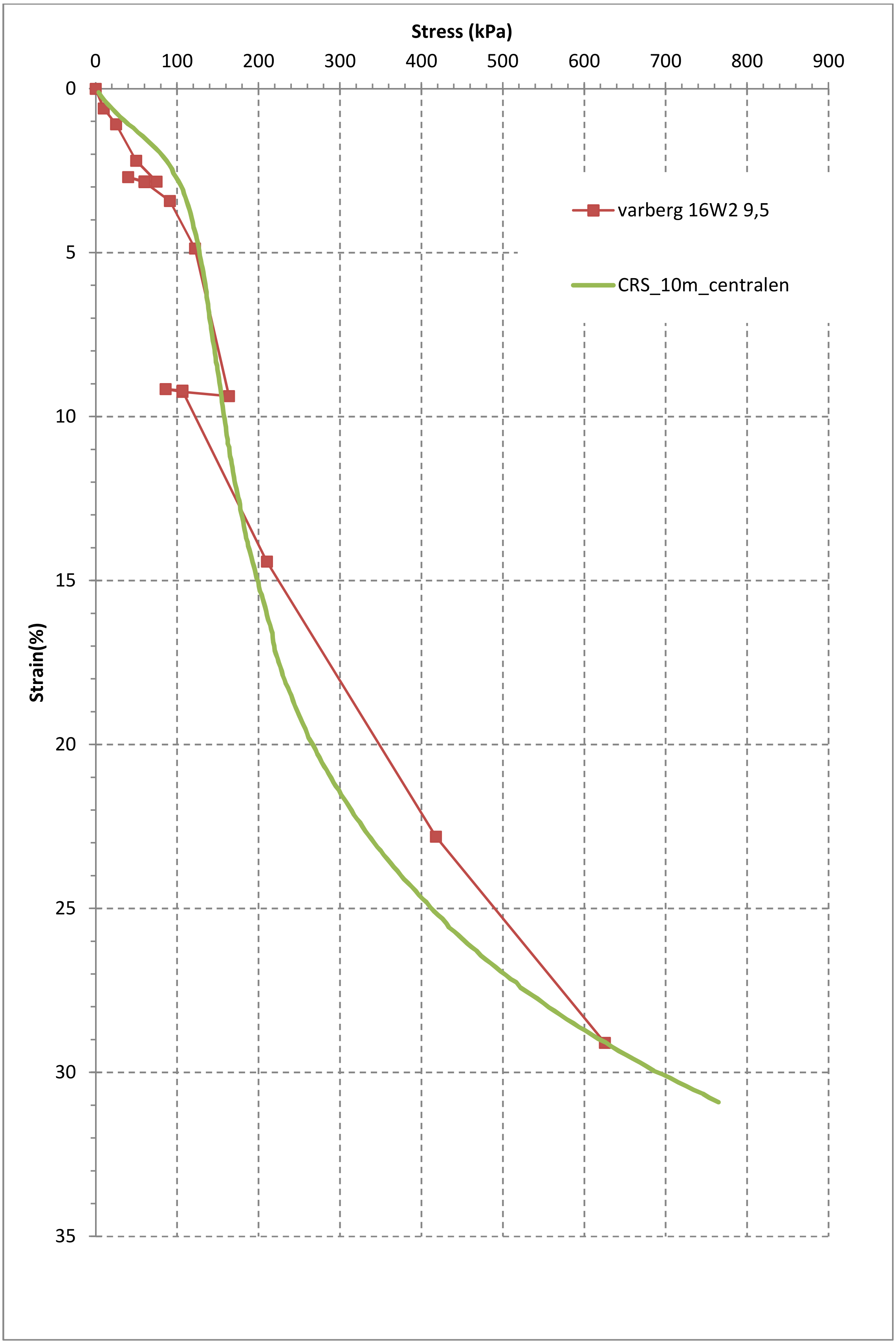
IL and CRS comparison 3 meters, notice the unloading an reloading cycles.

Appendix 5.2 IL results



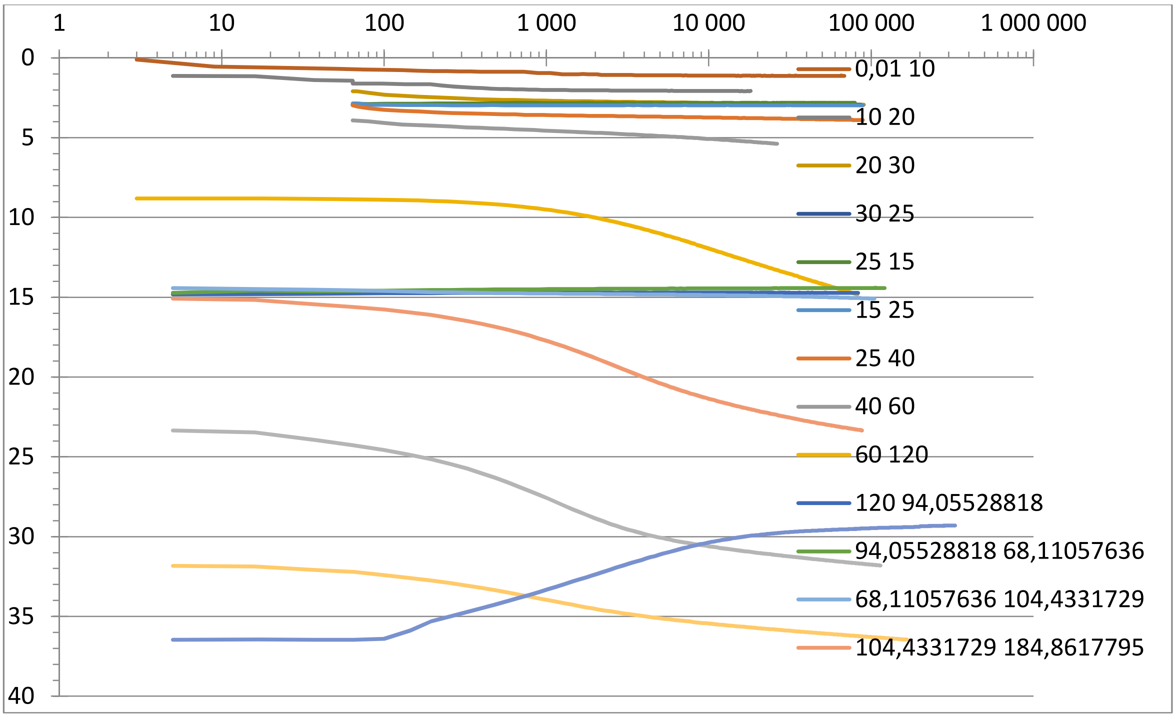
IL and CRS comparison 4,5 meters, notice the unloading an reloading cycles.

Appendix 5.3 IL results

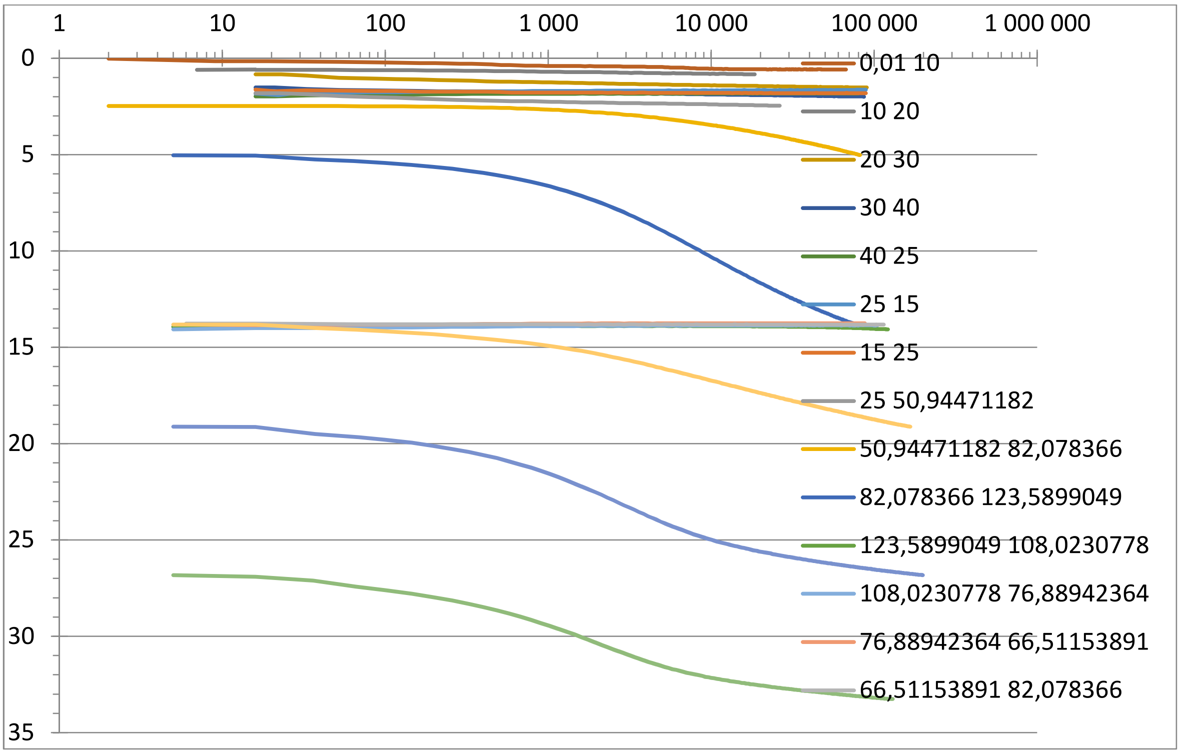


IL and CRS comparison 9,5 meters, notice the unloading an reloading cycles.

Appendix 5.4 IL results

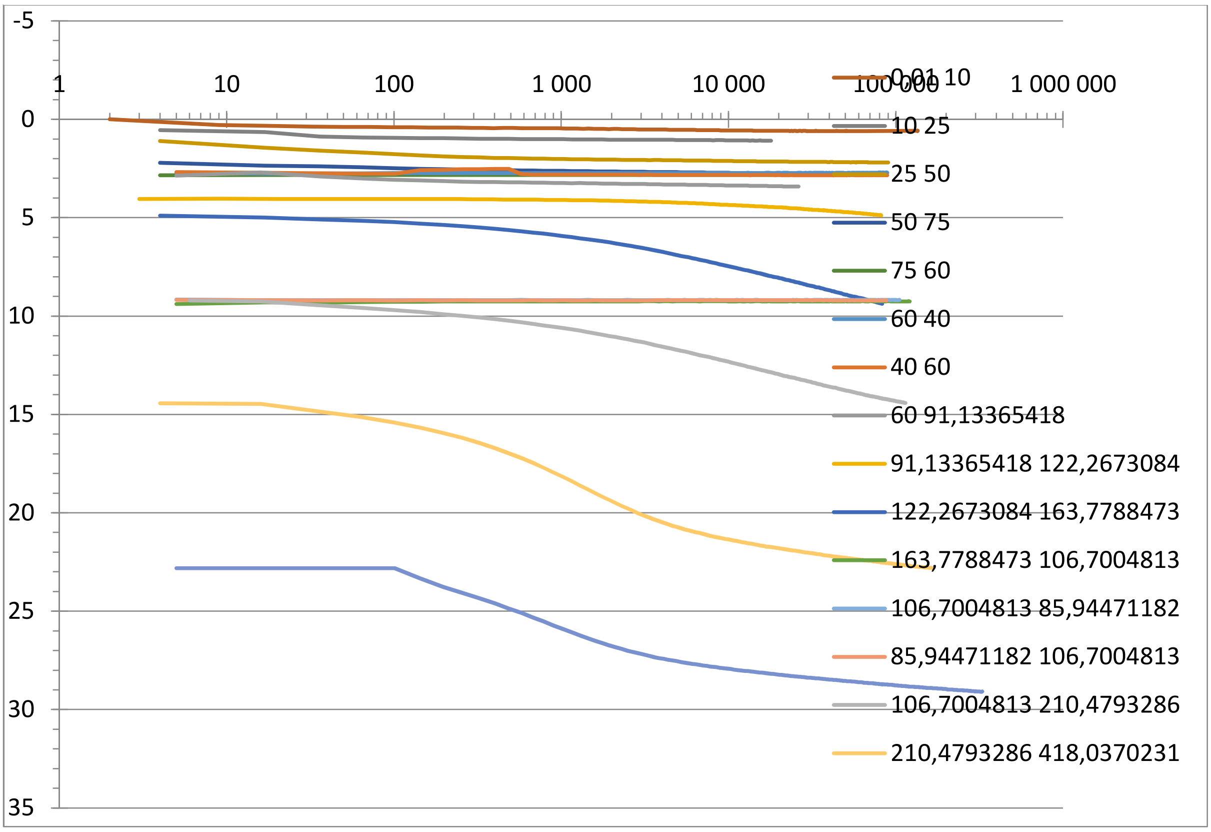


Settlment for each load step 3m



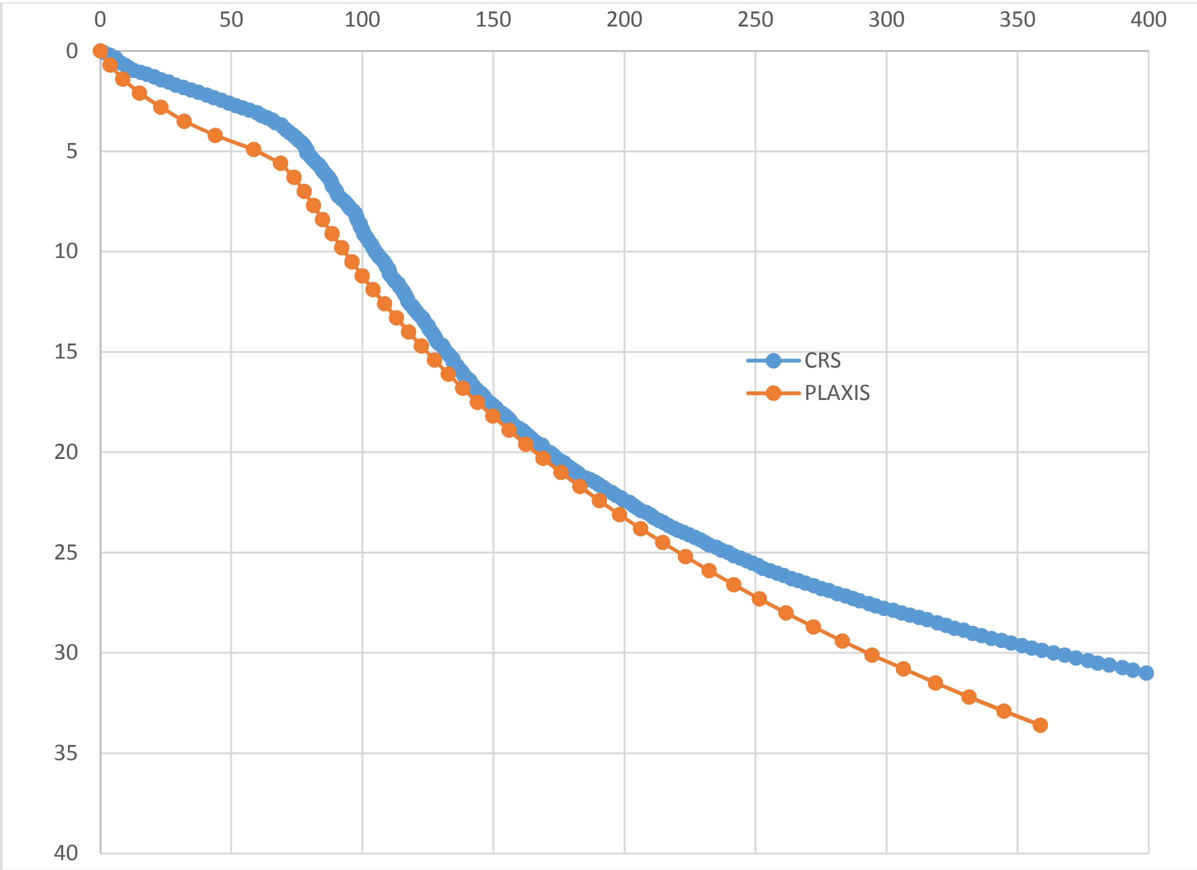
Settlement for each load step 4,5 m

Appendix 5.5 IL results

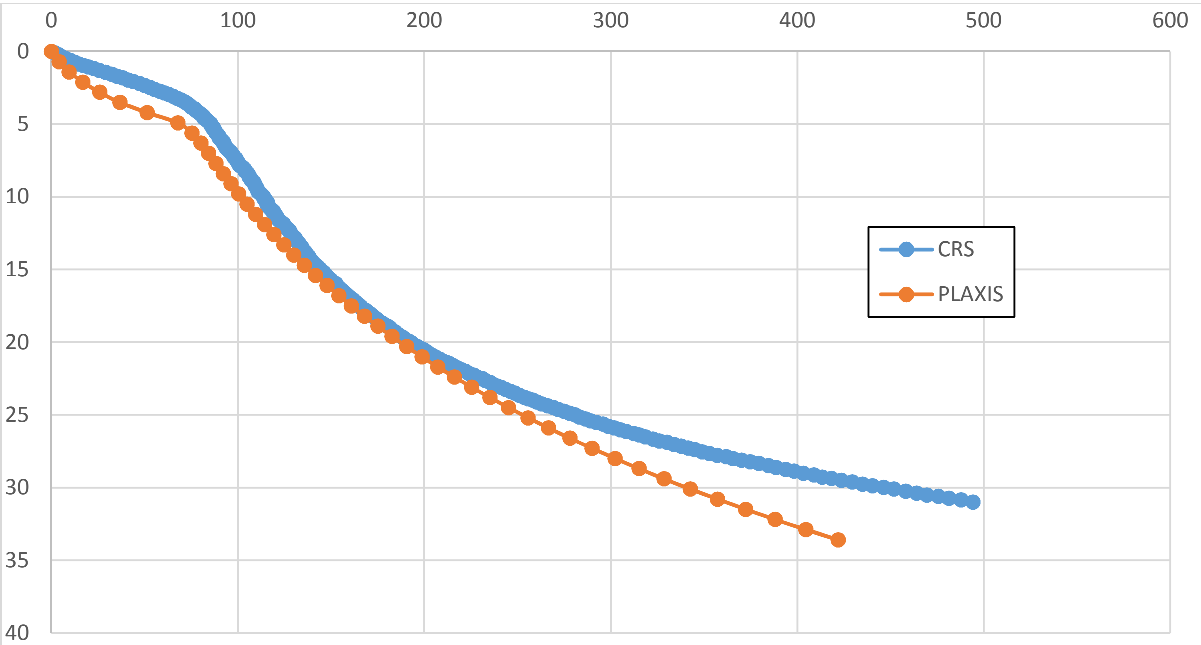


Settlement for each load step 9,5 m

Appendix 6.1 SoilTest Results

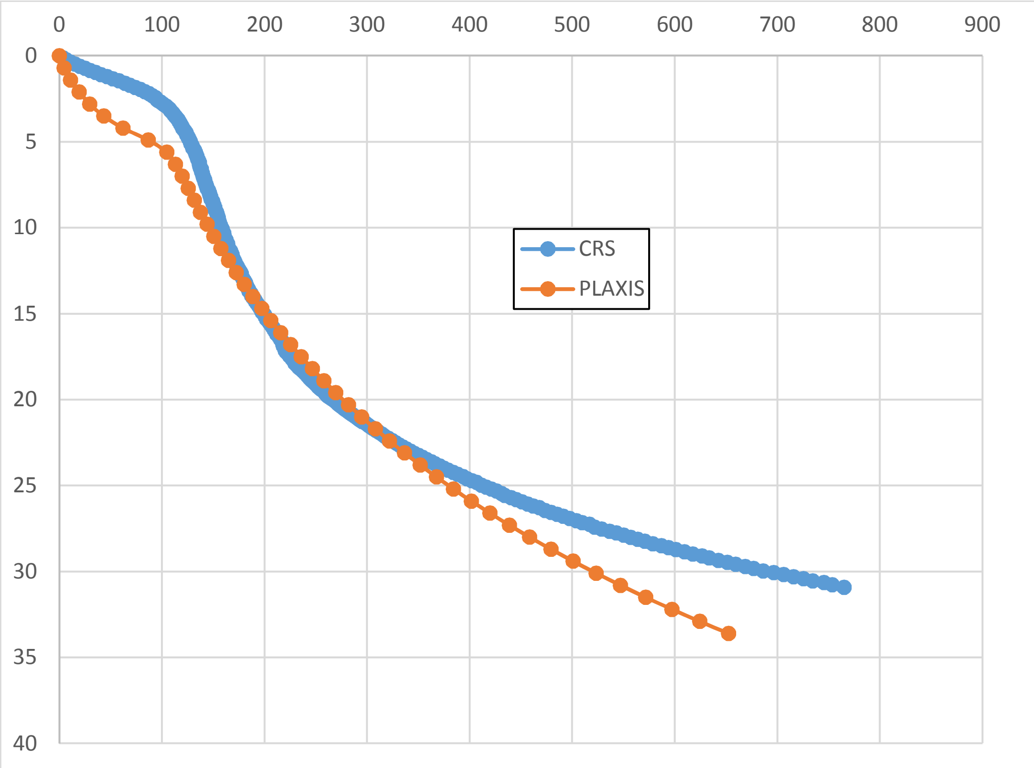


CRS 3m – SoilTest Clay 2-3 m σ_c=64 KPA

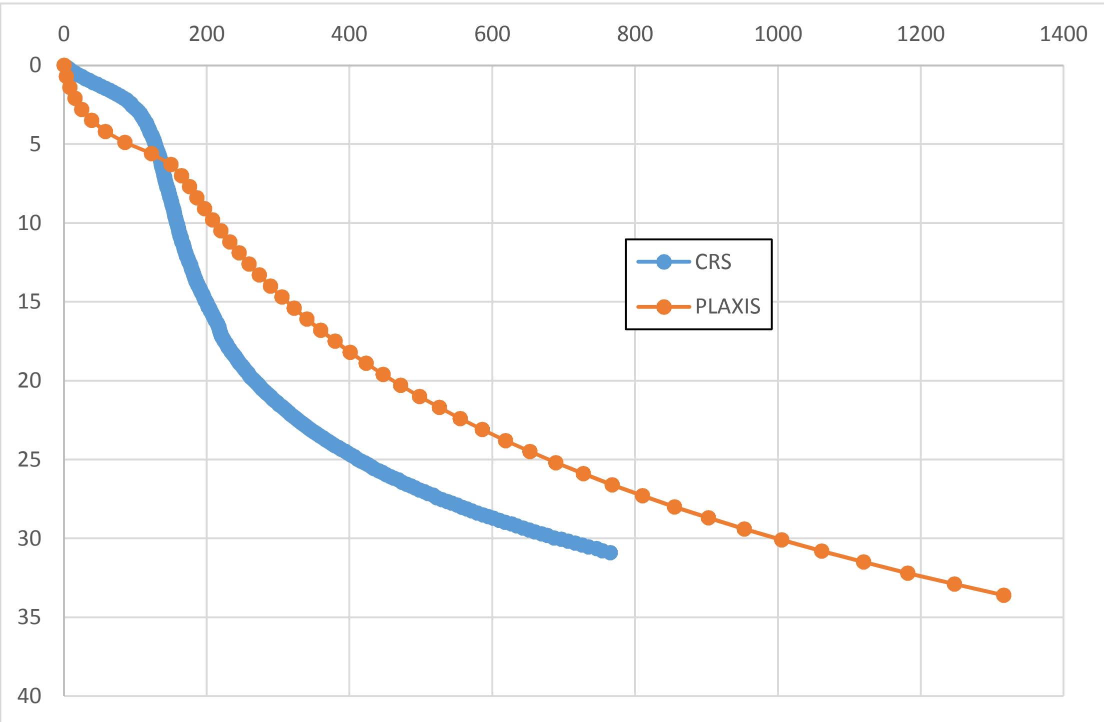


CRS 4,5m – SoilTest Clay 3-5 m σ_c=67 KPA

Appendix 6.2 SoilTest Results

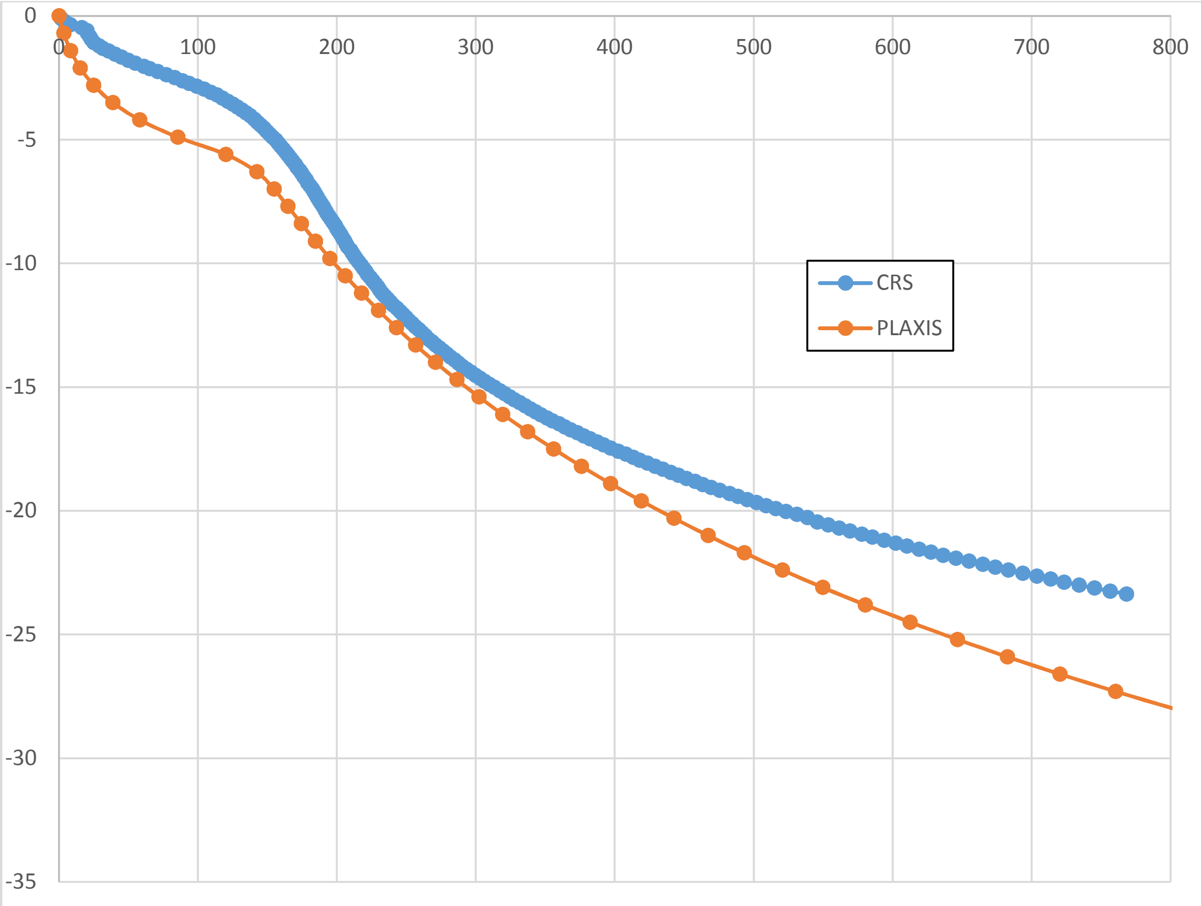


CRS 6,5m – SoilTest Clay 5-8 m σ_c=96 KPA



CRS 9,5 m – SoilTest Clay 8-13 m σ_c=134 KPA

Appendix 6.3 SoilTest Results



CRS 12,5 m – SoilTest Clay 8-13 m σ_c=124 KPA

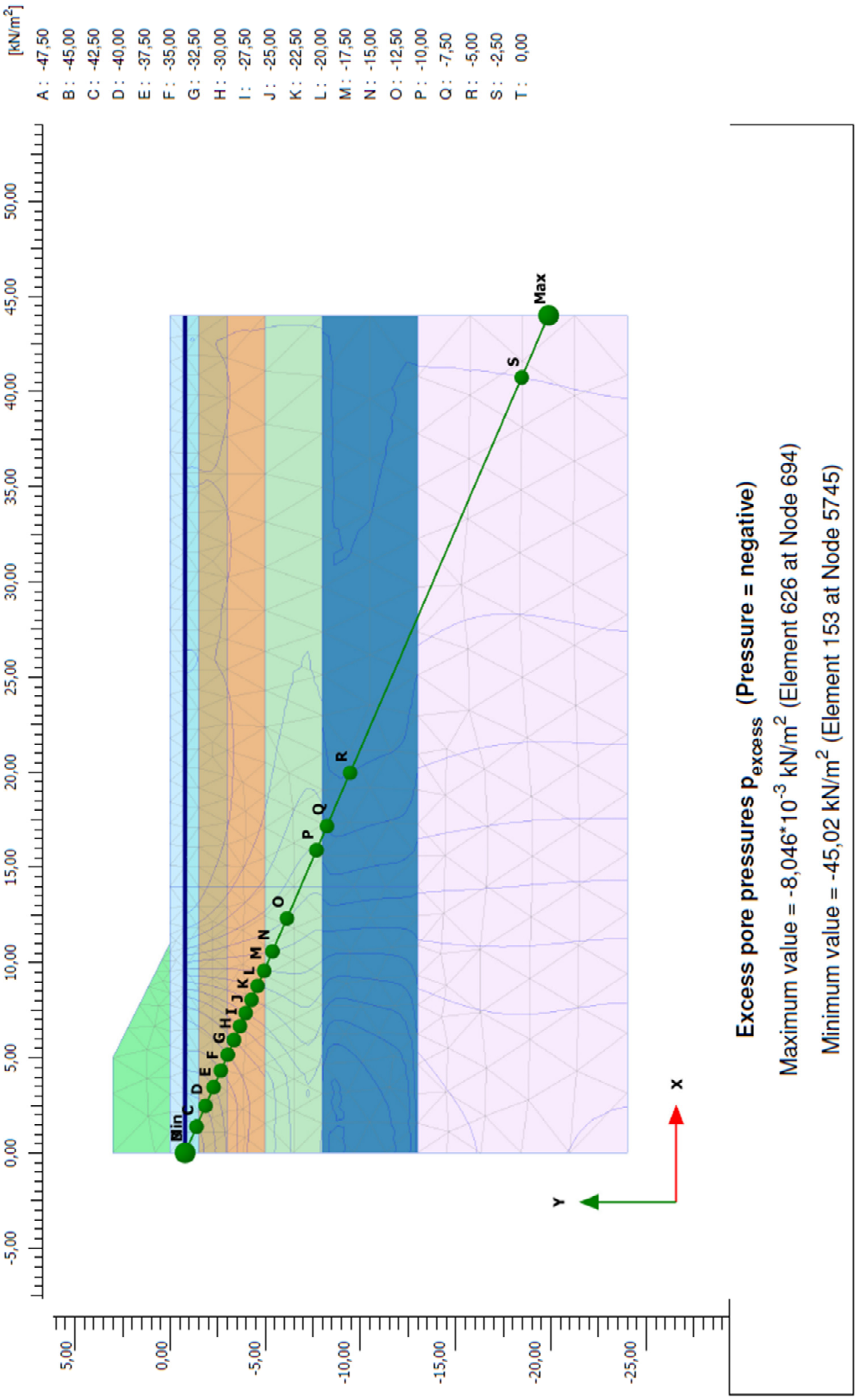
Appendix 7 GeoSuite Indata

Soil Layers

Name	Soil Model	Permeability Model	Depth	Sub Layers	Soil Weight [kN/m3]	M_0 [kN/m²]	M_L [kN/m²]	M' [-]	a_0 [-]	a_1 [-]	σ'_c [kN/m²]	σ'_L [kN/m²]	t_{ref} [years]
Crust	Chalmers without creep	Log based (strain)	0,00	8	17,0	15000,0	1000,0	12,00	0,8	1,0	500,00	550,00	NA
			0,80		17,0	15000,0	1000,0	12,00	0,8	1,0	500,00	550,00	NA
Le	Chalmers with creep	Log based (strain)	0,80	72	16,0	2826,0	319,0	10,00	0,8	1,0	65,00	100,00	-0,00274
			8,00		16,0	5761,5	762,0	14,00	0,8	1,0	99,00	146,00	-0,00274
le	Chalmers with creep	Log based (strain)	8,00	50	17,0	5761,5	762,0	14,00	0,8	1,0	99,00	146,00	-0,00274
			13,00		17,0	7800,0	1070,0	14,00	0,8	1,0	133,00	179,00	-0,00274
mix	Chalmers with creep	Log based (strain)	13,00	120	17,0	7800,0	1070,0	17,00	0,8	1,0	133,00	179,00	-0,00274
			25,00		17,0	12692,0	1808,0	23,00	0,8	1,0	215,00	256,00	-0,00274

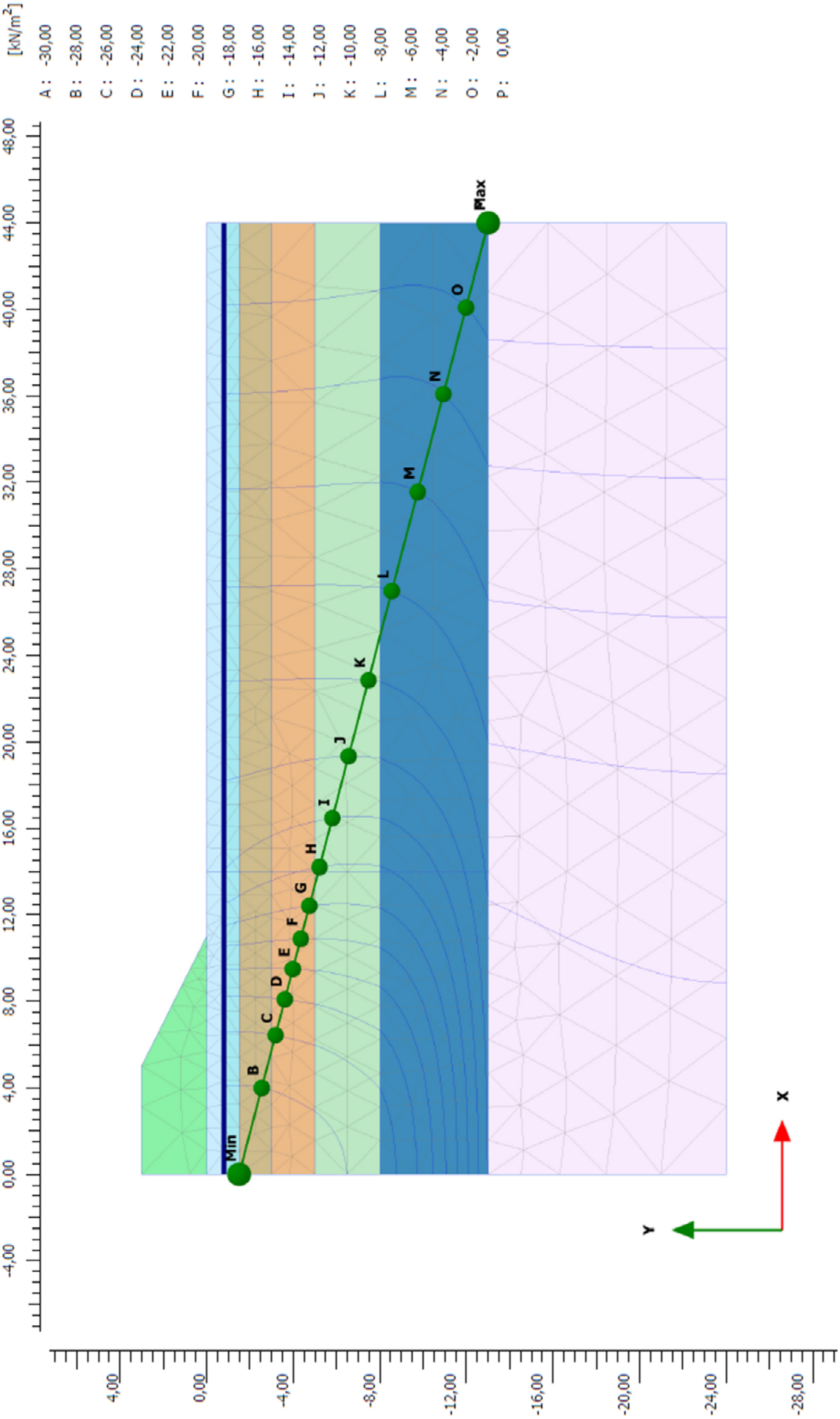
	a_1 [-]	σ'_c [kN/m²]	σ'_L [kN/m²]	t_{ref} [years]	b_0 [-]	b_1 [-]	r_0 [-]	r_1 [-]	k_{init} [m/years]	β_k [-]
,8	1,0	500,00	550,00	NA	NA	NA	NA	NA	0,0300	3,75
,8	1,0	500,00	550,00	NA	NA	NA	NA	NA	0,0300	3,75
,8	1,0	65,00	100,00	-0,00274	0,32	1,10	2428,0	190,0	0,1135	5,00
,8	1,0	99,00	146,00	-0,00274	0,58	1,10	1785,0	190,0	0,0548	5,00
,8	1,0	99,00	146,00	-0,00274	0,58	1,10	1785,0	250,0	0,0548	5,00
,8	1,0	133,00	179,00	-0,00274	0,75	1,10	1340,0	250,0	0,0140	5,00
,8	1,0	133,00	179,00	-0,00274	0,75	1,10	7300,0	150,0	0,8000	5,00
,8	1,0	215,00	256,00	-0,00274	1,10	1,10	7300,0	150,0	0,8000	5,00

Appendix 8.1 PLAXIS Porepressures



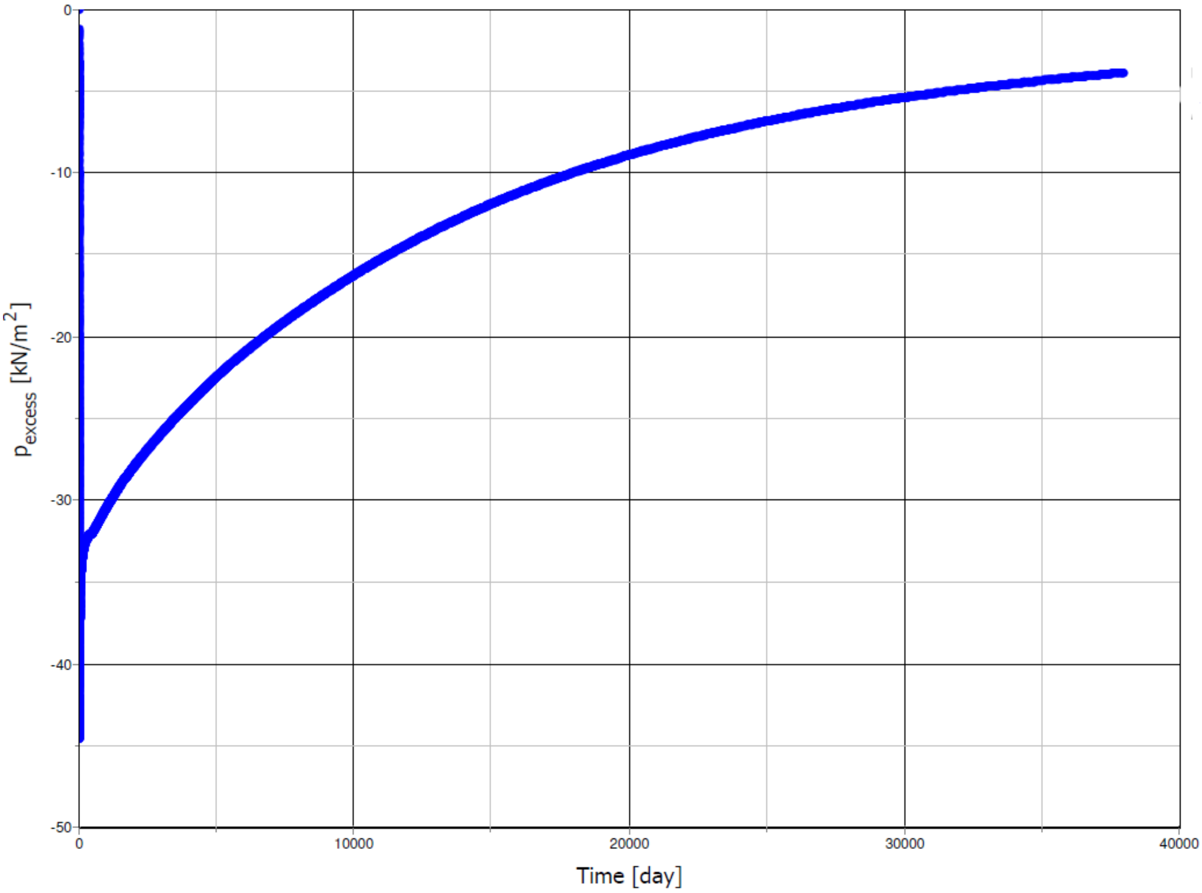
1. Excess pore pressures when the embankment is finished

Appendix 8.2 PLAXIS Pore pressures

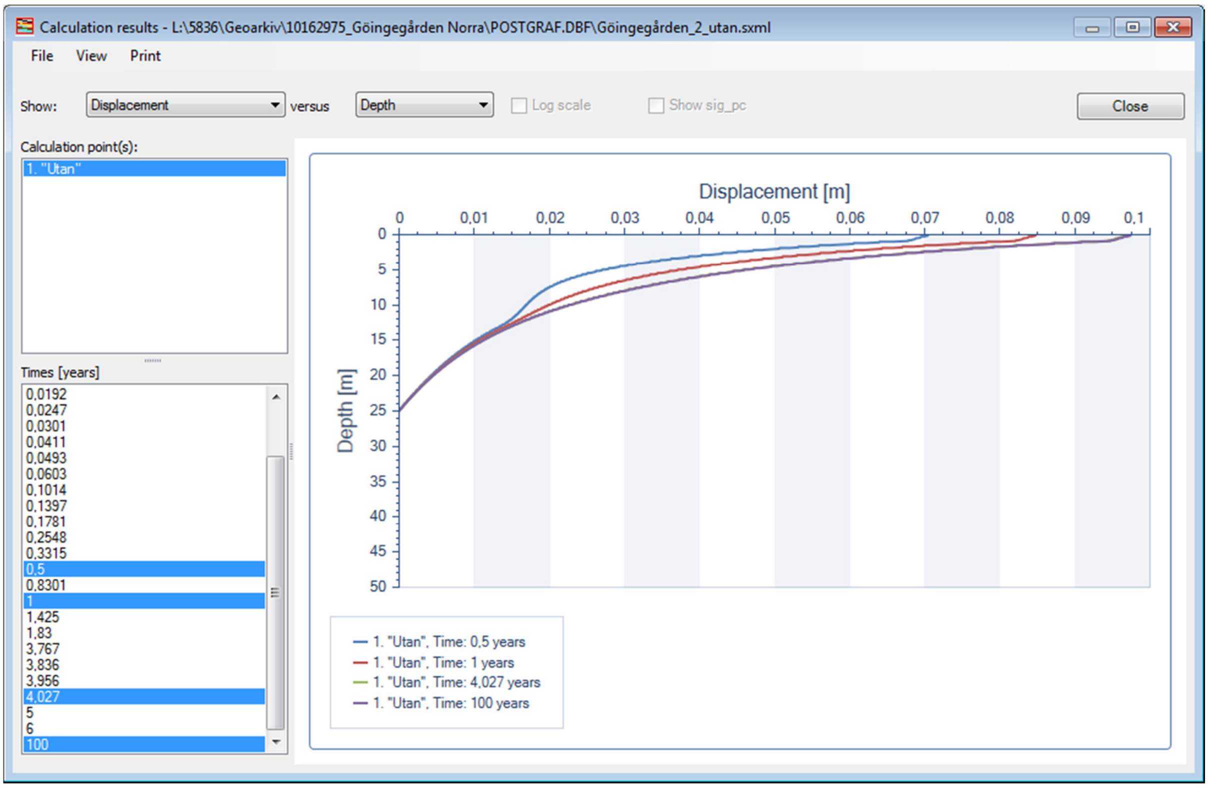


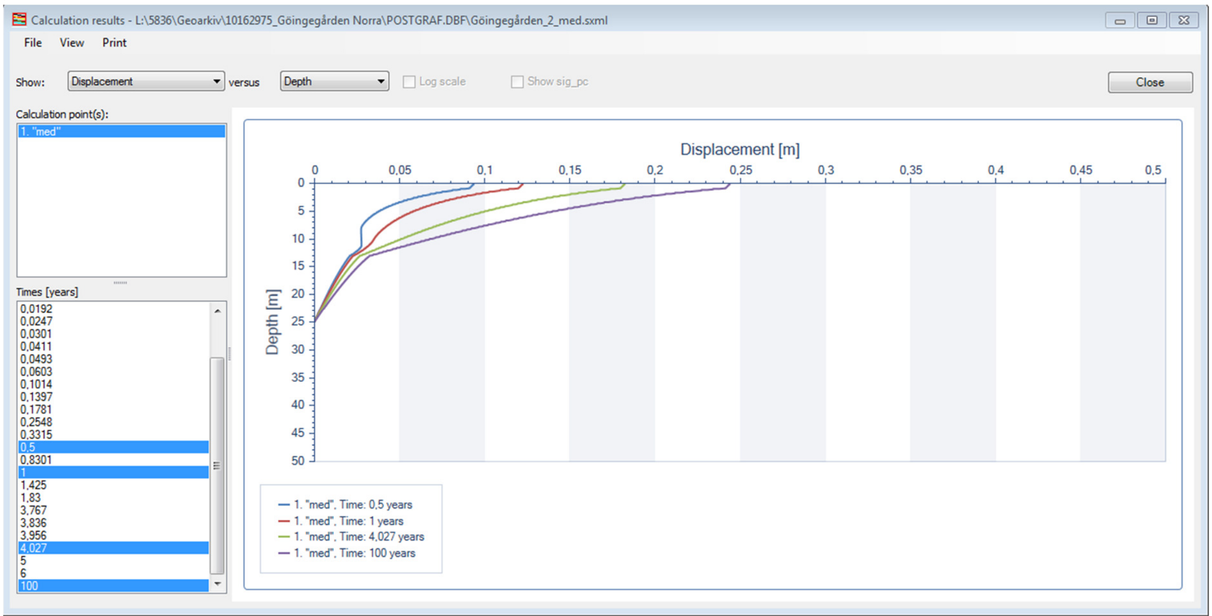
Excess pore pressures after 4 years

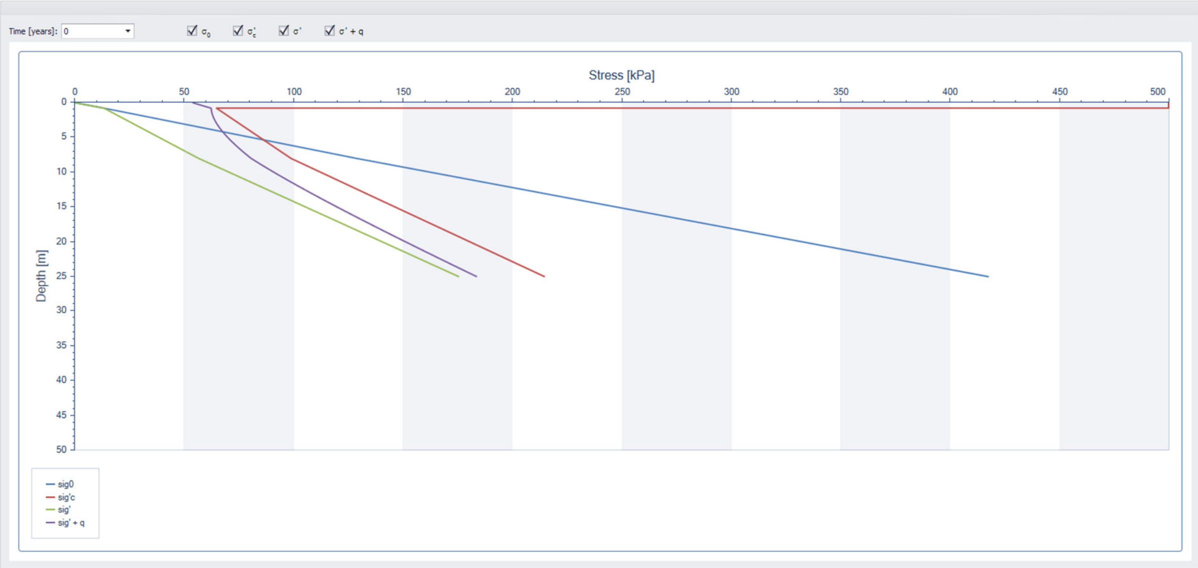
Appendix 8.3 PLAXIS Pore pressures



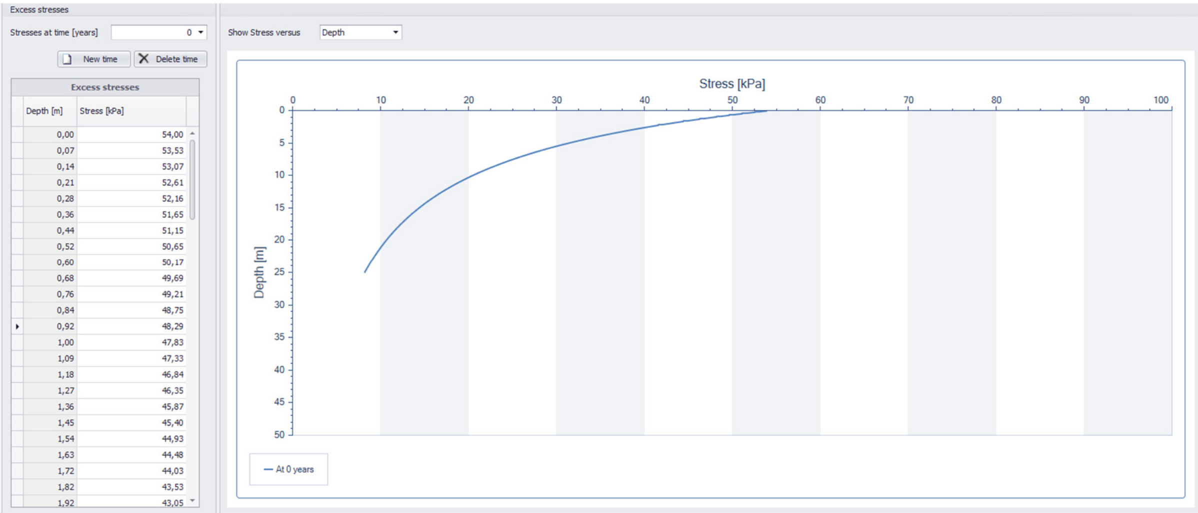
Porepressures and how the dissipated at the centerpoint at the ground water level


Calculation results - L:\5836\Geoarkiv\10162975_Göingegården Norra\POSTGRAF.DBF\Göingegården_2_utan.sxml
File
View
Print
Show:
Displacement
versus
Depth
Log scale
Show sig_pc
Close
Calculation point(s):
1. "Utan"
Times [years]
0,0192
0,0247
0,0301
0,0411
0,0493
0,0603
0,1014
0,1397
0,1781
0,2548
0,3315
0,5
0,8301
1
1,425
1,83
3,767
3,836
3,956
4,027
5
6
100
Displacement [m]
0
0,01
0,02
0,03
0,04
0,05
0,06
0,07
0,08
0,09
0,1
Depth [m]
0
5
10
15
20
25
30
35
40
45
50
1. "Utan", Time: 0,5 years
1. "Utan", Time: 1 years
1. "Utan", Time: 4,027 years
1. "Utan", Time: 100 years

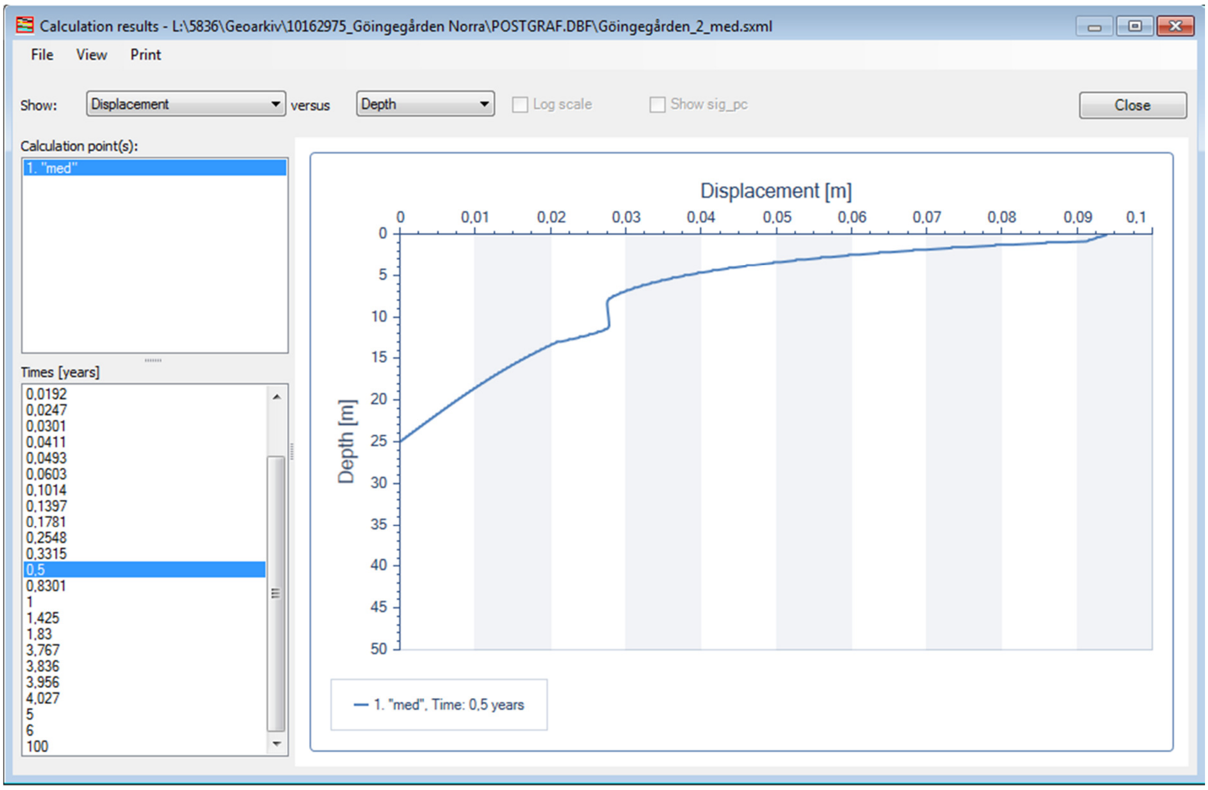

Calculation results - L:\5836\Geoarkiv\10162975_Göingegården Norra\POSTGRAF.DBF\Göingegården_2_med.sxml
File
View
Print
Show:
Displacement
versus
Depth
Log scale
Show sig_pc
Close
Calculation point(s):
1. "med"
Times [years]
0,0192
0,0247
0,0301
0,0411
0,0493
0,0603
0,1014
0,1397
0,1781
0,2548
0,3315
0,5
0,8301
1
1,425
1,83
3,767
3,836
3,956
4,027
5
6
100
Displacement [m]
0
0,05
0,1
0,15
0,2
0,25
0,3
0,35
0,4
0,45
0,5
Depth [m]
0
5
10
15
20
25
30
35
40
45
50
1. "med", Time: 0,5 years
1. "med", Time: 1 years
1. "med", Time: 4,027 years
1. "med", Time: 100 years



Time [years]: 0
σ0
σ'c
σ'
σ' + q
Stress [kPa]
Depth [m]
sig0
sig'c
sig'
sig' + q



Excess stresses
Stresses at time [years] 0
New time
Delete time
Excess stresses
Depth [m] Stress [kPa]
0,00 54,00
0,07 53,53
0,14 53,07
0,21 52,61
0,28 52,16
0,36 51,65
0,44 51,15
0,52 50,65
0,60 50,17
0,68 49,69
0,76 49,21
0,84 48,75
0,92 48,29
1,00 47,83
1,09 47,33
1,18 46,84
1,27 46,35
1,36 45,87
1,45 45,40
1,54 44,93
1,63 44,48
1,72 44,03
1,82 43,53
1,92 43,05
Show Stress versus
Depth
Stress [kPa]
Depth [m]
At 0 years


Calculation results - L:\5836\Geoarkiv\10162975_Göingegården Norra\POSTGRAF.DBF\Göingegården_2_med.sxml
File
View
Print
Show:
Displacement
versus
Depth
Log scale
Show sig_pc
Close
Calculation point(s):
1. "med"
Times [years]
0,0192
0,0247
0,0301
0,0411
0,0493
0,0603
0,1014
0,1397
0,1781
0,2548
0,3315
0,5
0,8301
1
1,425
1,83
3,767
3,836
3,956
4,027
5
6
100
Displacement [m]
0
0,01
0,02
0,03
0,04
0,05
0,06
0,07
0,08
0,09
0,1
Depth [m]
0
5
10
15
20
25
30
35
40
45
50
1. "med", Time: 0,5 years