UNIVERSITÀ DEGLI STUDI DI PADOVA

Dipartimento di Geoscienze

Direttore Prof. Cristina Stefani

TESI DI LAUREA MAGISTRALE
IN
GEOLOGIA E GEOLOGIA TECNICA

A CASE STUDY OF SOIL REINFORCEMENT WITH THE LIME-CEMENT COLUMNS METHOD: DALVÄGEN ROAD, STOCKHOLM

Relatore: Prof.ssa Simonetta Cola

Correlatore: Ing. Luigi Credendino

Laureanda: Cristina Ghirardini

ANNO ACCADEMICO 2015/2016

ACKNOWLEDGEMENTS

I want to thank first of all my family, in particular my parents, who supported me economically but also encouraging me during all this 5 years of study. Without your help I would have never reached this important goal.

Another big thank you goes to all the people of Bjerking and in particular to Luigi, who accompanied me during my traineeship in Sweden with perseverance but even patience. Thank you to Simon, Jens, Sorena, Gunnar, Sofia, Jona, Tomislav, Madelene, Kouroush, Benjamin, Johnny, Fredrik, Markus, Fanny, Ulf, Axel and Elizabeth for always making me feel home and for the kindness and availability demonstrated.

Thank you to my supervisor, prof. Simonetta Cola, who introduced me in geotechnics and helped me with this thesis.

A very big thank you goes to my special boyfriend Sunny who always supported me, persuading me to believe in myself and always being on my side. Without him I would never had the courage to move from home to undertake the traineeship in Sweden. Thank you for loving me as only you can do and for making me happy, the happiest woman in the world.

Thank you to all my friends, in particular to Stefania for sharing our house in Padova but also the ordinary anxiety before the exams and all the trips on the train. Thank you to Debora, Ingrid, Angelo, Alessandra and all the others.

Thank you to Fra Paolo for making me reflect about myself, my life and what I want to become and do in the future.

A last but not least thank you to my dog Giove who always supported me while studying, especially during the summers. Your love helped me to be calm and relaxed even when submerged by many exams. And now, even if you are not anymore in this world, I know you are here with me, so thank you my friend.

ABSTRACT (ITA)

La presente tesi ha come oggetto lo studio delle "colonne calce-cemento", una tecnica nata in Svezia alla fine degli anni '70 e che ha trovato progressivamente un largo impiego soprattutto nei Paesi Scandinavi, Olanda e Giappone. Essa è utilizzata per migliorare le caratteristiche meccaniche di terreni a grana fine, aventi proprietà molto scadenti, e si possono utilizzare fino a profondità medio-elevate (circa a 30m).

La tesi, frutto di un tirocinio di 4 mesi presso uno studio di progettazione di Stoccolma, mira a presentare un'applicazione del metodo, evidenziando le ragioni che hanno portato alla scelta di questa tecnica come metodo di rinforzo, ma anche analizzando gli effetti del miglioramento che si possono ottenere in termini di maggior stabilità meccanica e diminuzione dei cedimenti. Comprende una parte teorica di introduzione al metodo delle colonne calce-cemento, con particolare attenzione alle prescrizioni riportate nella normativa svedese, e una parte pratica con la descrizione dell'area di studio, dell'intervento, i calcoli e le analisi dei cedimenti e della stabilità.

L'area di studio è situata in una zona di residenza estiva fuori Stoccolma che prevede di essere trasformata in zona residenziale permanente, dove quindi si prevede di intervenire con un potenziamento del sistema viario che richiede la costruzione di alcuni rilevati stradali e scavi sotto falda. Il terreno presente è composto da terreni argillosi e organici con caratteristiche e spessori variabili da sezione a sezione. A tale proposito, per facilitare l'esecuzione dei calcoli, il tracciato stradale è stato diviso in 4 tronchi aventi stratigrafia abbastanza omogenea, in termini di spessore dell'argilla, di valore di resistenza al taglio e di parametri di deformazione.

I cedimenti sono stati calcolati mediante il metodo edometrico, a partire dai parametri deformativi ricavati su alcuni campioni indisturbati di terreno con la prova CRS: è stata eseguita anche un'analisi di sensibilità volta ad individuare i parametri che più influenzano i cedimenti e la loro possibile variabilità in funzione dell'eterogeneità del suolo. Per completare l'esame del progetto sono state eseguite alcune analisi di stabilità con il metodo dell'equilibrio limite evidenziando come in condizioni naturali i terreni non garantiscano la stabilità degli scavi.

Nell'ultimo capitolo si esaminano i vantaggi tecnici ed economici che inducono a utilizzare la stabilizzazione calce-cemento e si effettua una valutazione dei cedimento ottenibile con il trattamento. Infine, le analisi di stabilità dopo l'intervento sono state svolte cambiando i valori della resistenza al taglio fino a individuare i valori minimi necessari al fine di avere una situazione di completa sicurezza, e da questi è stato possibile calcolare l'interasse tra le colonne necessario per ottenere l'effetto di rinforzo desiderato.

Il lavoro termina con alcune conclusioni atte ad evidenziare il notevole miglioramento nel comportamento del terreno.

Contents

Nomenclature

Roman symbols

a (Täckningsgrad) ratio of total column area to total area of reinforced soil

A area of cross section of columns

c distance between column centres

c_1 columns distance for a single columns pattern

c_2 columns distance for a slab columns pattern

c_3 columns distance for a slab columns pattern (overlapping)

c_{s} effective cohesion of the slice

c_{uk} characteristic undrained shear strength

$c_{\mathrm{k}}^{'}$ characteristic effective cohesion

c_{v} vertical consolidation coefficient

c_{h} coefficient of consolidation for horizontal flow

d_{col} diameter of columns

d vane width

$E_{1\mathrm{s}}$, $E_{2\mathrm{s}}$ forces acting on a slice

E_{col} elasticity modulus of the columns

f_{s} frictional resistance (CPT)

g gravitational acceleration

Δh stratum thickness

H drainage length

k permeability

k_{clay} permeability of the un-stabilized clay

k_{col} permeability of the columns.

L_{D} drainage length

l_{s} width of each slice

M_{L} compression modulus

M_0 modulus below pre-consolidation pressure
M' modulus number
M_{clay} compression modulus of the clay
M_{col} compression modulus of the columns
M_{t} torque moment
M_{stab} stabilizing moment on the slice
$M_{\text{un-stab}}$ un-stabilizing moment on the slice
n ratio of influence radius of column to column radius
N_{s} force acting on a slice
Q net penetration resistance
q total load on the reinforced area
q_1 load carried by single column
$q_{1\text{max}}$ maximum load carried by single column
q_2 load carried by un-stabilized clay
q_{t} point resistance (CPT)
R influence radius of column
r column radius
R_{s} radius of the sliding surface
S_{s} force acting on a slice
S_1 settlement in column
S_2 settlement in un-stabilized clay
S_{eff} stabilizing effect
t time of consolidation
Th time factor
U degree of consolidation
u water pressure
w water content
W_{s} weight of each slice
w_{L} liquid limit
X_1, X_2 forces on a slice

Greek symbols

γ specific weight
γ_{k} characteristic unit weight of the stabilized clay

ε deformation

ρ density

ρ_{w} water density

$\sigma^{'}$ effective vertical stress

$\sigma^{'}_{0}$ effective vertical stress in situ

$\sigma^{'}_{\mathrm{C}}$ pre-consolidation pressure

$\sigma^{'}_{\mathrm{L}}$ limit pressure

σ_{h} horizontal stress on columns

σ_{ult} ultimate strength of columns

τ shear strength

τ_{col} undrained shear strength of the columns

$\tau_{\mathrm{fdk/fuk}}$ drained/undrained shear strength

τ_{stab} shear strength of the stabilized soil

$\tau_{\mathrm{un-stab}}$ shear strength of the un-stabilized soil

ν_{col} Poisson ratio of the columns

ϕ internal friction angle

$\phi^{'}_{\mathrm{k}}$ characteristic effective angle of internal friction

Acronym

CPT: cone penetration test

DJM: dry mixing methods

DMM: deep mixing methods

SF: safety factor

GWT: ground water table

$Jb2$: soil/rock probing test

LCC: Lime-Cement columns

LCN: line of normal consolidation

OCR: over-consolidation ratio

SGI: Swedish geotechnical institute

SlB: percussion sounding test

Tr: pyramid penetration test

Vim: weight sounding test

INTRODUCTION

Stockholm is the capital of Sweden and also the most populated city in the Nordic countries. It is located on the south-central east coast of Sweden in fact it is exposed on the Baltic Sea and it is formed by a lot of islands, which compose the Stockholm's archipelago.
Its urban area has a population of more one million people, and if we consider the entire metro area around Stockholm its population exceeds the two million people, on a total of 10 million people in the whole Sweden. This means that the concentration of people in Stockholm area is very high, and according to the statistics it is set to increase in the next years.
Stockholm city has been constructed on soft marine clays which are subjected to important settlements, and being a boomtown its future is to grow on this type of soil. For this reason in the middle of 1970 a new method for stabilizing soils has been developed in the Nordic countries, and in the following decades is has been subjected to a large research and improvement. This method is currently widely common in the whole world, and it is called Lime-Cement Columns method. It consists in the creation of strong and resistant columns by mixing the soil with some binders as lime and cement, with the aim to avoid the settlements and to improve the stability of eventual excavations.

This thesis is the result of a study of theoretical knowledge but also of practical work, which has been developed during a 4 months internship in Sweden.

The thesis is organized in this way:

- The theoretical background is reported in chapter 1 through the several sub-chapters, mainly focused on the Lime-Cement columns and their Swedish regulations.

- The practical part starts with chapter 2 which is focused on the geographical location and the geotechnical conditions of the soil in the study area. The main road of the study, Dalvägen, has been divided into 4 different areas, and for each area a geotechnical model of the soil has been created. The division is based on the thickness of clay, the shear strength and the deformation parameters; 4 colours have been confered to each area to distinguish between they.

- In chapter 3 there is the description of the CRS test, which is used to obtain the deformation parameters which are necessary to calculate the settlements. The used method of settlement calculation in the clay is described and the results of the settlements are presented for each area.

- In chapter 4 there is the description of the slope stability analysis with the two methods of Bishop and Fellenius, and the resulting slope stability analysis of each area is reported.

- In chapter 5 the reason for using the Lime-Cement columns is explained, in both practical and economical point of view. It continues with the description of the method of settlements calculation after reinforcement with the Lime-Cement columns and the results are reported. Then the shear strength of the clay is increased to reach the correct safety factor and this permits to make a planning of the columns necessary for the stabilization. The improvement reached using this method is highlighted through the several results.

- The works ends with chapter 6 with some conclusions of the work.

Capitolo 1

BACKGROUND ON LIME-CEMENT COLUMNS

The Lime-Cement columns is classified as a deep mixing method and also as a dry mixing method, so some knowledge about these general categories are necessary.

1.1 Deep mixing methods

The deep mixing methods are techniques used since 1970/1980 to stabilize soft soil improving their geotechnical and engineering properties, in particular the shear strength and the deformations properties. These methods were born simultaneously in Sweden, Finland and Japan due to the necessity to build new infrastructures on bad quality soils, because of the over-population in the urban areas. In fact, both in Japan and in Sweden we can often find important layers of soft alluvial or marine clay which need to be stabilized. Currently these techniques find several applications in (Holm, 1999 & prEN 14679, 2005):

- ground improvement;
- improvement of slope stability (structures and embankments);
- reduction of settlements (embankments and structures);
- support of slopes and excavations;
- improvement of bearing capacity;

- reduction of vibrations and their effects on structures;
- seismic and liquefaction mitigation;
- construction of containment structures;
- immobilization and/or confinement of waste deposits or polluted soils.

The operating principle of these methods is the injection of different stabilizing agents called binders which are mixed with the soil creating a uniform, homogenous, and more resistant structure.
These methods are now very common because from an economical point of view they are very competitive and have some advantages compared to other techniques (Broms, 1999), but also thanks to their high reliability.
Several categories of the methods exist, mainly distinguished on the type of binders, its type of injection and the mechanism of mixing (Bruce et al., 1998):

- the most used binders are cement, lime, lime-cement, gypsum and ash. The choice of the binder depends on many factors, included the composition of the soil, the geotechnical characteristics and the desired depth of work;
- the two types of injection of the binders are the wet deep mixing methods and dry deep mixing methods. The main difference between the two is that in the wet methods the binders are injected through the water, while in the dry methods the binders are injected through compressed air;
- the two corresponding types of mechanism of mixing are the rotary mixing and the jet-assisted mixing.

The mixing process is one of the most important stages that affect significantly the improved properties of the stabilized soil. The installation process of the columns in the Nordic countries can be divided in three principal phases (Larsson, 2005):

- penetration of the mixing tool to the desired depth, usually at a rate of 100 mm/rev;

- dispersion of the binder all over the cross section of the columns. In this phase it is necessary to reach a complete remoulding of the clay, because this leads the clay to release water which is available for wetting the lime and the cement. More than this, the dispersion of the binder is favored by the complete disaggregation of the soil;

- molecular diffusion: there is the migration of the calcium ions from the stabilized soil into the un-stabilized soil, but it is difficult to evaluate the extent of this process.

Essentially, the deep mixing methods lead to a reinforcement of the soil thanks to the mixing of this with the stabilizing agents, creating resistant, stiff and strong columns. A simple representation of the construction of the columns is showed in Figure 1.1.

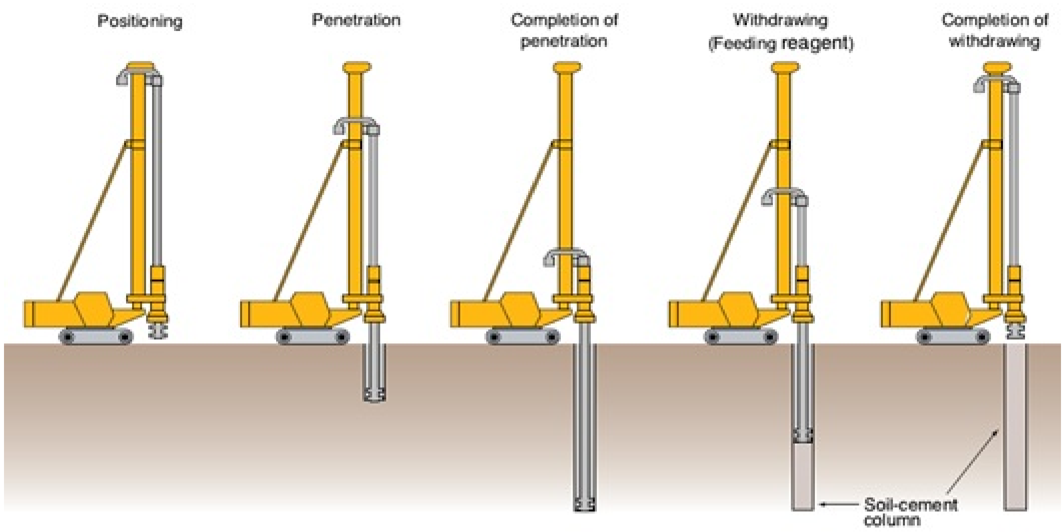


Figure 1.1: Construction sequence of the columns in a DMM method (Raito kogyo co., ltd, DJM system).

The improved properties of the soil must be verified by field tests, because the laboratory tests on prepared specimen can't provide accurate shear strength and deformation properties (Larsson, 2005).
Specifically, the several factors that affect the soil improvement are reported in Table 1.1.

Characteristics of the binder	1. Type of binder 2. Quality of binder 3. Mixing water and additives
Characteristics and conditions of the soil	1. Physical, chemical and mineralogical properties of soil 2. Organic content 3. pH of pore water 4. Water content
Mixing conditions	1.1. Degree of mixing 2. Timing of mixing 3. Quantity of binder 4. Water to cement ratio
Curing conditions	1.Temperature 2. Curing time 3. Humidity 4. Confining pressure 5. Wetting/drying/freezing

Table 1.1: factor affecting the soil improvement (Terashi 1997).

1.1.1 prEN 14679 for deep mixing.

In 2005 the European Committee for Standardization (CEN) published the document prEN 14679, which is the Eurocode regarding the "Execution of special geotechnical works – Deep Mixing". This is a European standard and it establishes the general principles for the execution, testing, supervision and monitoring of deep mixing works, including both dry and wet methods. The standard is referred to methods that involve (prEN 14679, 2005):

- the mixing by rotational mechanical mixing tools where the lateral support provided to the surrounding soil is not removed (substantially is doesn't include the mass stabilization);
- the treatment of the soil to a minimum depth of 3 m;
- different shapes and configurations, consisting in single columns, panels, grids, blocks, walls or any combination of more than one single column, overlapping or not;
- treatment of natural soil, fill, waste deposits and slurries.

Moreover the standard gives instructions about:

- the geotechnical investigations that permit to fix the ground conditions, the physical, the mechanical, the environmental, the chemical and biological characteristics;
- the materials used, that have to comply with European or national standards. In fact, the properties of the binder must be investigated by laboratory and in-situ tests, the quantity of binder along the column shall be measured during the installation and the speed of rotation, the rate of penetration and retrieval of the mixing tool shall be adjusted to produce sufficiently homogeneous treated soil;
- the supervision and the monitoring during the construction of the columns and the testing after the construction, that has to verify the strength characteristics, the deformation properties and the homogeneity of the columns.

The principles of execution of the deep mixing methods are then summarized as showed in Figure 1.2.

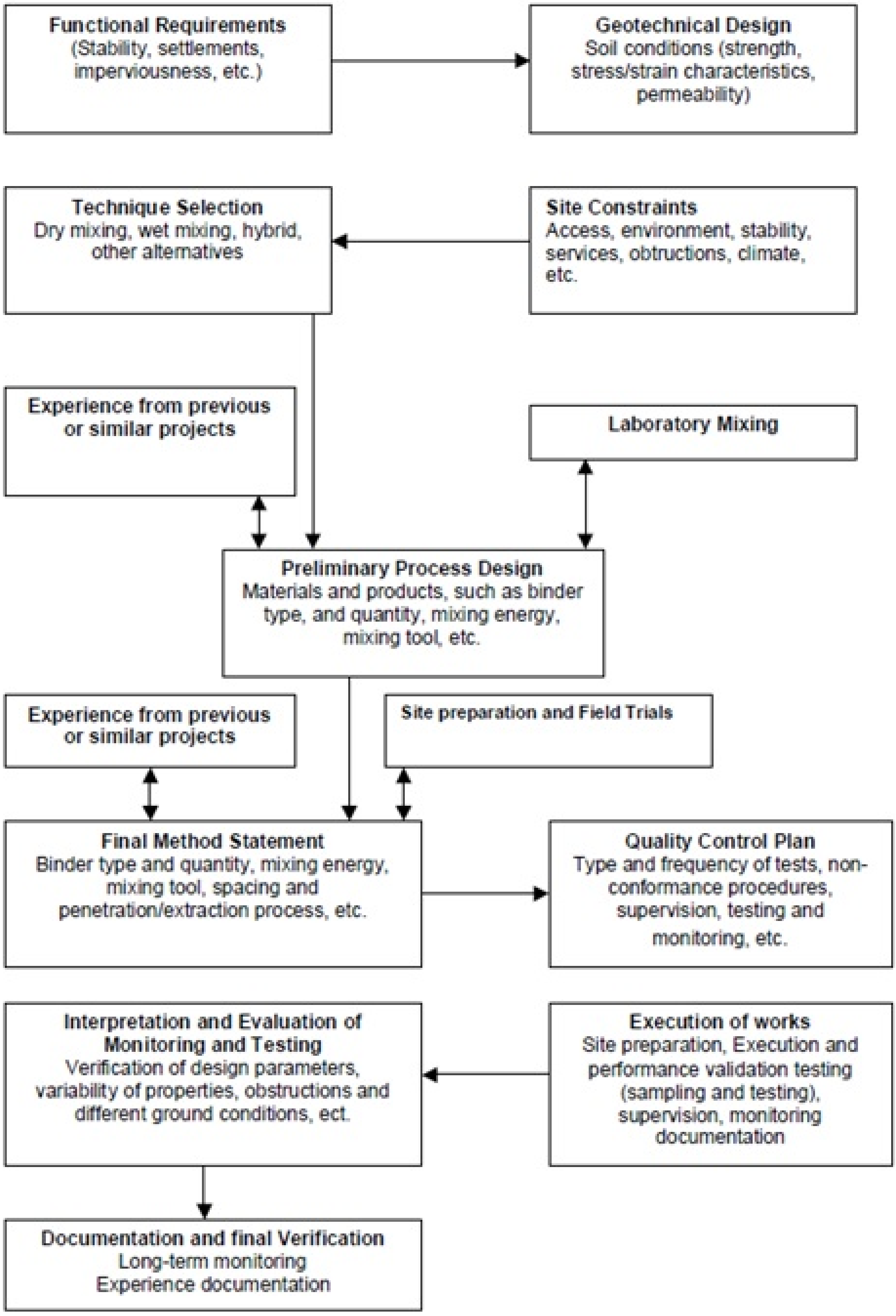


Figure 1.2: principle of execution of DDM (prEN 14679, 2005).

1.1.2 Dry mixing methods.

The DJM was developed first in Scandinavia around 1970 and currently it is still used to reduce the compressibility and increase the resistance of soft clayey soils. As already mentioned the dry mix methods inject the binders in the soil through compressed air. A proper pressure of the injected air is one of the most important parameter, because if the pressure is too low the binder may not spread to the whole cross-sectional area of the column, while if the pressure is too high there will be problems of air entrainment and ground movement (prEN 14679, 2005).
A representation of the equipment of a DJM method is showed in Figure 1.3.

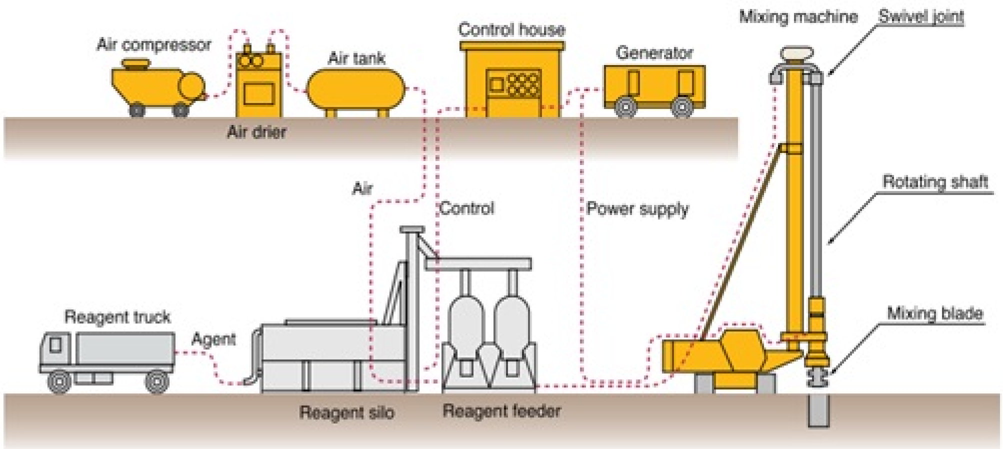


Figure 1.3: principle of execution of DJM (prEN 14679, 2005).

We can notice that the air is given by a compressor and then contained in an air tank; on the other hand, a truck transports the binders at the working site and discharge they in a binder silo. After that, the air and the binders are carried in a single pipe that goes to the DJM machine, which will inject the resulting blend in the soil while mixing all.
Since the method doesn't provide for the use of water, the soft soil where the DJM is applied should have a water content of at least 20 % (Shenghua et al., 2012). Alternatively, some additive to lime and cement as ash or gypsum can be added; these additive have the skill to promote pozzolanic reactions. This type of chemical reaction happens when silicaceous or aluminous materials, in presence of water and calcium hydroxide, form cemented products (Esrig, 1999).

Obviously, as every method the DJM methods present some advantages but also some disadvantages. As part of the advantages, if compared with the columns produced with the wet techniques, the columns formed with DJM methods present higher shear strength, produce less noise, less vibrations and less spoil material, and less binder to inject is required (Bruce, 2000). As part of the disadvantages, the most important and limiting factor is that only a limited range of soils and with a minimum water content can be treated. (Lang et al., 1999). During the past, another limiting factor was the maximum depth of treatment, which was around $15 - 18\ m$, while now it has been significantly improved up to $25 - 30\ m$.
Several configurations of the method exist producing columns of different diameter and length, but the factor that influences mostly the final product is the stabilizer injection method, keeping a dry configuration:

- injection of the binders during the withdrawal of the mixing tool;
- injection of the binders during the penetration of the mixing tool;
- injection of the binders during the penetration and the rest during the withdrawal.

The best of these methods would be the second one because the soil and the binders are mixed for all the time the mixing tool remains in the soil, resulting in a better homogeneity and a greater unconfined compressive strength (Hayashi et al., 1999), but in the practical reality the binders are injected during the withdrawal of the mixing tool, as showed in Figure 1.1.

1.1.3 prEN 14679 for dry mixing.

The European standard distinguishes the two major dry methods: the Japanese DJM and the lime cement column method, today known as the Nordic technique.

- The equipment used in the Nordic countries is able to install columns up to a depth of $25\ m$ with a column diameter between $0,6\ m$ and $1,0\ m$. The mixing energy and the amount of binder are monitored and in some cases automatically controlled to achieve sufficiently uniform treated soil. During the retrieval phase, the soil and binder are mixed by continued turning of the mixing tool, eventually changing the direction. The rotation speed of the mixing tool and the speed of withdrawal are adjusted to produce uniform mixing.

- In Japan, there are several variant machines, which are able to install columns up to a depth of 33 m. The air pressure and the amount of binder are automatically controlled to achieve homogeneity of the treated column. The binder is injected during the penetration stage or both during the penetration and retrieval stages. The standard Japanese mixing tool is a bit different compared to the Swedish one (Figure 1.5), and it is showed in 1.4.

Figure 1.4: standard Japanese mixing tool (Larsson, 2005).

The main differences between the Nordic and the Japanese dry mixing technique are then summarized and reported in Tables 1.2 e 1.3.

Equipment	Details	Nordic technique	Japanese technique
Mixing machine	Number of mixing shafts	1	1 to 2
	Diameter of mixing tool	0,4 m to 1,0 m	0,8 m to 1,3 m
	Maximum depth of treatment	25 m	33 m
	Position of binder outlet	Bottom of shaft (single)	Bottom of shaft and/or mixing blades (single or multiple)
	Injection pressure	Variable 400 kPa to 800 kPa	Maximum 300 kPa
Batching plant	Supplying capacity	50 kg/min to 300 kg/min	50 kg/min to 200 kg/min.

Table 1.2: Comparison between the Nordic and the Japanese techniques (prEN 14679, 2005).

Mixing machine	Nordic technique	Japanese technique
Penetration speed of mixing shaft	2,0 m/min to 6,0 m/min	1,0 m/min to 2,0 m/min
Retrieval speed of mixing shaft	1,5 m/min 6,0 m/min	0,7 m/min to 0,9 m/min
Rotation speed of mixing blades	100 revolutions/min to 200 revolutions/min	24 revolutions/min to 64 revolutions/min
Blade rotation number[1)]	150 per m to 500 per m	≥ 274 per m
Amount of binder injected	100 kg/m^3 to 250 kg/m^3	100 kg/m^3 to 300 kg/m^3
Retrieval (penetration) rate	10 mm/rev to 30 mm/rev.	10 mm/rev to 35 mm/rev.
Injection phase	Typically during retrieval	Penetration and/or retrieval

Table 1.3: Typical execution values of the Nordic and the Japanese dry mixing techniques (prEN 14679, 2005).

1.2 Lime-Cement columns.

The Lime-Cement columns method is one of the most used in Sweden to stabilize the soft soils as clay, to avoid the settlements and to increase the stability. It is a dry mix method because no water is required to inject the binder into the soil. The main functionality of the method is to improve the geotechnical properties of the soil, thanks to the formation of stiff reinforcing columns which can act also as a drain and accelerate the consolidation process. Moreover, the load applied on the surface is carried partly by the columns and partly by the surrounding soil, as will be seen in the settlement calculations.

Although from a physical point of view the columns and the clay are two distinguished objects, both from an analytical and practical point of view, the columns and the surrounding clay are considered as a homogeneous body with average characteristics of the two parts.

The stabilizing effect of the columns depends on many factors as (SGF Report 4:95, 2000):

- the area of the columns compared with the total area of the work;
- the diameter, depth and spacing of the columns;
- the properties of the soil, included the permeability;
- the effect and the amount of the binder;
- the time of the pre-loads application.

The method is usually matched with the use of the pre-loads that consists on the application of excessive loads to force the settlements to happen during the construction phase and to avoid long term creep settlements that can be strongly dangerous for the future constructions. More than this, another aim of the pre-loads is to consolidate the soil with a higher load than the final load.

Since the lime cement columns method was born in around the middle of 1970's, it has been improved and new applications have been found. For example, at the beginning of '70 the method used only lime as binder, then cement was add to accelerate the column formation and to achieve a considerably higher shear strength, and finally the cement has almost completely replaced the lime. The installation of the columns occurs with the instrument of Figure 1.5.

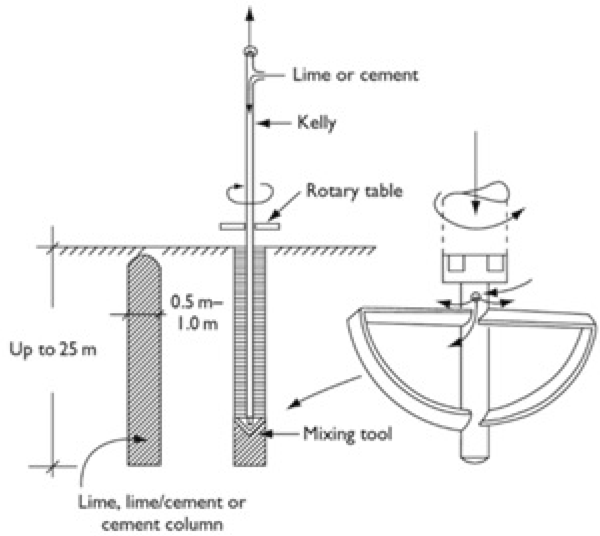


Figure 1.5: Swedish standard mixing tool (Broms 2004, Larsson et al., 2005).

The construction sequence of the Lime-Cement columns corresponds to the one for the DJM methods and is showed in Figure 1.1, while in figure 1.5 there is the Swedish standard mixing tool specific for the Lime-Cement columns. The mixing tool rotates with a specific velocity, while compressed air is injected, and the binders are mixed with the soil. The mixing tool is progressively pulled upward while the column is forming and the binders are distributed so the chemical reactions can take place and produce uniform columns. The design of the mixing tool is that to distribute uniformly the binders both in the cross section and along the length of each column.

1.2.1 Major applications.

The main growing of the Lime-Cement columns method occurred after the construction of Kansai international airport in Japan, between 1989 and 1994. This airport has been built on an artificial island constructed thanks to the excavation of some of the mountains surrounding Osaka Bay, and has been stabilized with Lime-Cement columns because very big settlements (some meters) have occurred (Funk, 2014).
In Sweden one of the biggest and most important application of the lime cement columns has been the stabilization of the road E4 at Ullånger, 500 km north of Stockholm. The road was subjected to landslides, so a reinforcement was necessary, and the method used was the Lime-Cement's one. The reason of the use of the Lime-Cement columns was the continuously working of these and the no needs of maintenance. Some soil samples were extracted from the soil and mixed in the laboratory with lime and cement, to test the new strength parameters. Then the columns were designed with a definite diameter, a specific ratio of lime-cement, a specific pattern and finally were installed to stabilize the soil (Viberg et al., 1999).
The design process procedure adopted in Ullånger continues to be the nowadays procedure in Sweden.

1.2.2 Properties.

The properties of the stabilized soil that affect significantly its behavior are:

- Undrained shear strength: increases with the lime-cement content and with the normal pressure. It is determined by unconfined compression tests or direct shear stress by the following equation:

$$\tau_{\mathrm{fuk}} = c_{\mathrm{uk}} \tag{1.2.1}$$

  Test results according to Broms (1999) show an internal friction angle varying between 25 and 45 degrees. According to the Swedish regulations,

$$c_{\mathrm{uk}} = a \times c_{\mathrm{uk(col)}} + (1-a) \times c_{\mathrm{uk(clay)}} \tag{1.2.2}$$

  where a is defined as the ratio between the area of the columns and the total area of the reinforced soil as follows:

$$a = \frac{A}{c^2} \tag{1.2.3}$$

- Drained shear strength: it influences the long term stability, and it is determined by the following equation:

$$\tau_{\mathrm{fdk}} = c'_{\mathrm{k}} + \sigma' \times tan\phi'_{\mathrm{k}} \tag{1.2.4}$$

  According to Broms (1999) the effective internal friction angle varies between 30 and 35 degrees, but for the Swedish regulations it is assumed to be 30 degrees and c'_{k} it is calculated as:

$$c'_{\mathrm{k}} = a \times c'_{\mathrm{k(col)}} + (1-a) \times c'_{\mathrm{k(clay)}} \tag{1.2.5}$$

  Where $c'_{\mathrm{k(col)}} = c_{\mathrm{uk(col)}}$.
  The peak of shear strength for the columns happens at the same time as the peak of shear strength for the unstabilized soil between the columns, see Figure 1.6. This confirms the full interaction between the soil and the columns.

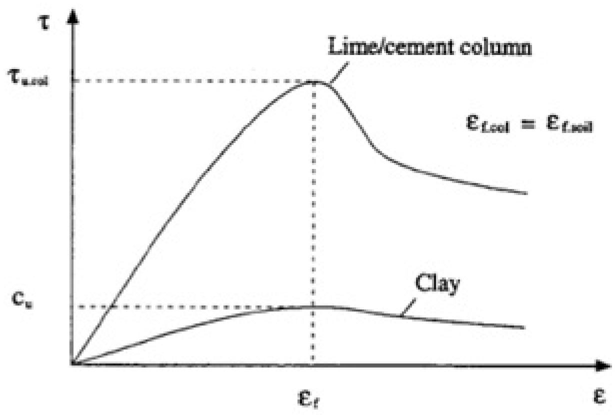


Figure 1.6: strength-strain relation for calculating the average shear strength (Kivelö, 1998).

- Compression modulus: it is determined by consolidation test as CRS test and it is one of the most important parameter for settlement calculations. It raises with time because the increase of shear strength thanks to the columns installation. It can be estimated from the following equation:

$$M_{\text{col}} = \frac{E_{\text{col}} \times (1 - \nu_{\text{col}})}{(1 + \nu_{\text{col}}) \times (1 - 2\nu_{\text{col}})} \tag{1.2.6}$$

- Elasticity modulus: it is the main elastic parameter, it influences the deformation way. It is determined by unconfined compression tests, and can be assumed $E_{\text{col}}/c_{\text{u,col}} = 200$ for Lime-Cement columns, according to Broms.

- Permeability: it is determined by oedometer or triaxial tests, though the permeability measured in the laboratory is usually lower than the one measured in situ. By the way, it is strongly influenced by the amount of cement because this tends to reduce the permeability, and it decreases with time because the cementation.

1.2.3 Binders.

As suggested by the own name of the method, the stabilizer used with the LCC method are the lime and the cement. Some knowledges about the properties of these binders and the chemical reactions that give binders their strength are necessary. The lime (CaO) is obtained from the calcination (heating) of the limestone, accordingly to the following equation:

$$CaCO_3 + heat \rightarrow CaO + CO_2 \tag{1.2.7}$$

When the lime is mixed with water it reacts: the water is absorbed, some heat is released and the lime increases his volume. The specific reaction that happens is called hydration and is represented by the following equation:

$$CaO + H_2O \rightarrow Ca(OH)_2 + 65,3kJ/mol \tag{1.2.8}$$

The reactivity of the lime depends on its particle size, in particular finer lime reacts more rapidly.

The mostly used cement is the standard Portland cement, which leads to the process called cementation, where the clay minerals hydrates forming new crystals, resulting in an higher resistance and in the formation of hard cement paste. While the reaction proceeds the voids between the cement particles will be filled and the cement paste grows denser, as showed in Figure 1.7.

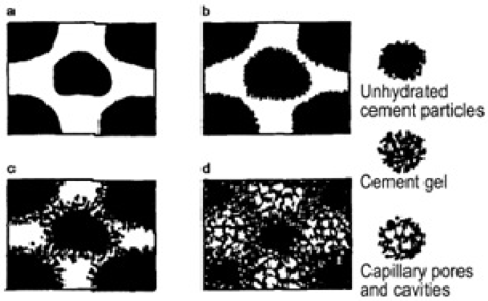


Figure 1.7: structure of the cement paste (Janz et al.,2002).

The Portland cement contains approximately 5% of gypsum and its specific surface is in the range of $300 - 550\ m^2/kg$. The specific surface is the surface area of the material that is exposed to water and influences the reaction rate.

A typical composition of the standard Portland cement is reported in Table 1.4.

Oxide	CaO	SiO_2	Al_2O_3	Fe_2O_3	MgO	SO_3	K_2O	Na_2O
Conteent [%]	60–70	17–25	2–8	0–6	0–6	1–4	0,2–1,5	0,2–1,5

Table 1.4: typical composition of the standard Portland cement (Janz et al.,2002).

The reactivity of the cement depends on many factors:

- the ratio of lime to silica $CaO : SiO_2$: larger this ratio, more hydraulic the material;
- the porosity of the cement paste;
- the water cement ratio $wcr = W/C$ where W is the weight of mixing water and C is the weight of cement. An high wcr implies high water content, so high porosity and low strength, as showed in Figure 1.8.

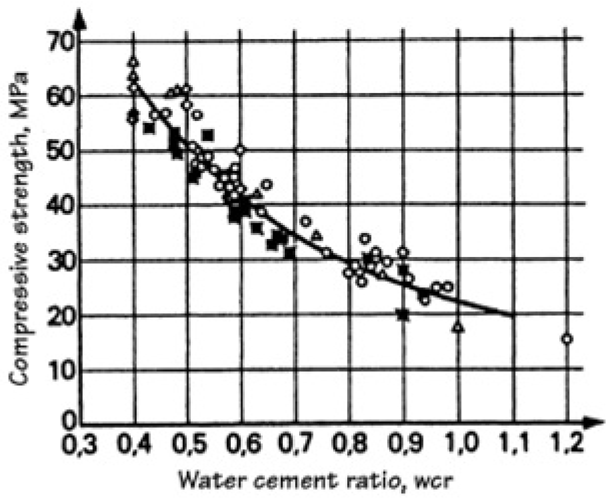


Figure 1.8: relation between wcr and the strength (Janz et al.,2002).

Usually half of the cement has already reacted after 3 days, and 90% after 3 months (Janz et al., 2002).

So, the main function of the binders is to produce chemical reactions that leads to the formation of new products characterized by a higher shear

strength and resistance, and to improve the deformation properties of the soil. In particular, the cement reacts immediately with water giving a high strength, while the lime reacts slower, resulting not in a strength gain but in a temporary effect on stability due to the water consumed.
The efficiency of the mixing in the soil depends on many factors, but the one that mostly influences the results is the quantity of binders, as showed in Figure 1.9.

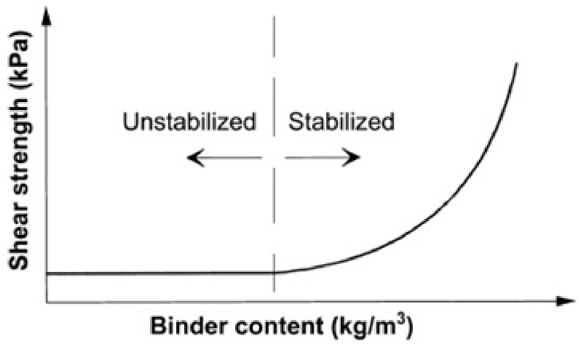


Figure 1.9: relation between the binder quantity and the shear strength of the soil (Janz et al.,2002).

The effect of the different type of binders is showed in Figure 1.10, where it is important to notice that the effect of lime only is always less than the one of cement or lime-cement, while cement or lime-cement have always a good effect on the clayey soils.

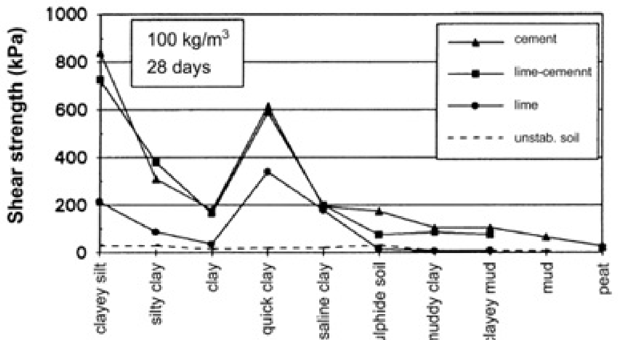


Figure 1.10: shear strength of stabilized soil with different binders, measured 28 days after the mixing (Janz et al.,2002).

It is often advantageous to combine lime and cement because the hydration of cement gives a rapid strength improvement, while the lime accelerates the pozzolanic reactions. In 2005 Jacobson et al., tried different lime-cement mixture to determine the best effect on alluvial deposits of very soft soil and highly compressible silts and clays, and obtained a contour plot of strength in function of cement and lime, as showed in Figure 1.11.

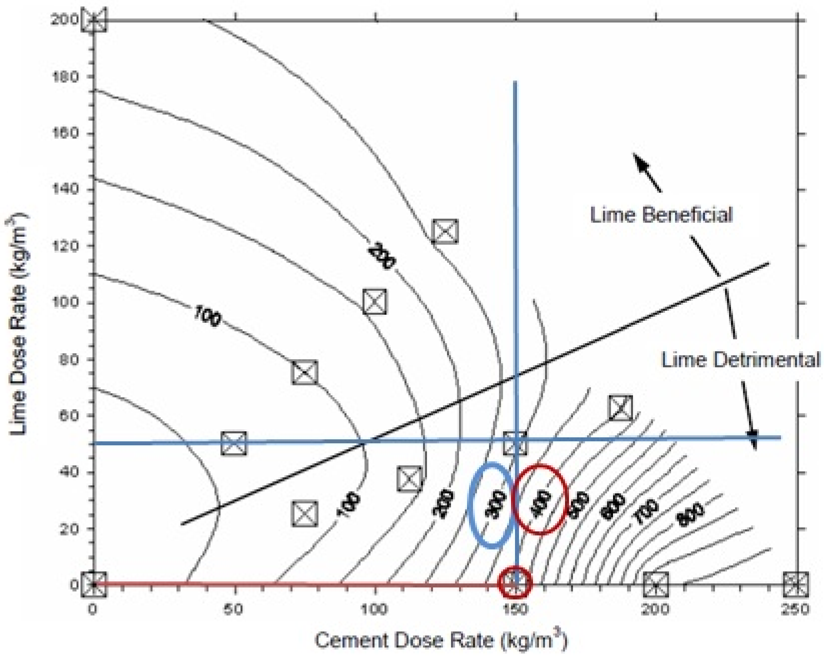


Figure 1.11: contour plot of unconfined compressive strength in function of cement and lime (Jacobson 2005, modified).

The first important thing to notice is that the strength is strongly dependent by the amount of cement rather the lime's one. In fact, observing the Figure 1.11 it can be noticed that for small increments in the amount of cement the strength raises significantly, while even for big increments of lime the strength increases slowly.
Another important thing to notice is the splitting of the graph in two areas, the lime beneficial and the lime detrimental. It means that in strength terms and in some configurations the addition of lime is negative, even if the lime, which is a stabilizer, is introduced. For example, considering a cement dose of 150 kg/m^3 and a lime dose of 0 kg/m^3, the strength reached is 400 kPa, as highlighted by the red line and circles. On the other hand, considering always a cement dose of 150 kg/m^3 with a lime dose of

50 kg/m^3, the strength is 300 kPa, as highlighted by the blue lines and circle. In this case the lime is detrimental because even if it is a stabilizer, its additions reduces the strength of the mixture.

In 1995, Åhnberg et al., stabilized that the optimal mix for reaching a high shear strength is in the range of 60-90% cement and 40-10% lime, but the determination of the right lime-cement ratio varies from case to case, because it is strongly dependent on the type of soil and its properties.

1.3 Swedish regulations.

The Swedish regulations regarding the lime cement columns have been collected by the Swedish Geotechnical Society in the Lime and Lime-Cement columns guide for project planning, construction and inspection for soft and semi-hard columns (that means max shear strength 100 kPa), report 4:95, 2000. Notwithstanding these regulations have been written 16 years old, they are the regulations currently adopted, so they will be presented because the Lime-Cement columns of this work will be constructed accordingly to these.

1.3.1 Investigations and first checks.

Before installing the Lime-Cement columns, some field and laboratory investigations are necessary to determine some important conditions, as:

- the sequence of strata: thickness, composition, stability;
- the groundwater conditions;
- the presence of obstacles as stones, boulders and tree roots;
- the properties of the soil before and after the mixing with lime-cement;
- SF without column reinforcement.

1.3.2 Fixed properties of the columns.

Some of the properties of the columns seen before are fixed:

- γ_{k} is the characteristic unit weight of the stabilized clay, it is equal to the one of un-stabilized clay;
- c_{uk} has a maximum value of 100 kPa;
- M_{col} is $50 - 150c_{\mathrm{uk}}$ for Lime-Cement columns;
- k is assumed 500 times the k of the un-stabilized clay;
- $c^{'}_{\mathrm{k(col)}}$ is equal to $c_{\mathrm{uk(col)}}$;
- $\phi^{'}_{\mathrm{k}}$ is equal to 30°.

1.3.3 Columns patterns.

The lime cement columns can be installed in different patterns, as showed in Figure 1.12.

- If the safety factor (SF) is more than 1, the columns can be placed singularly in a square or rectangular pattern. Exception: if the slope of the ground surface is greater than 1:7 and the SF is less than 1.2 the columns must be placed in slabs or grids.
- If the SF is less than 1, the columns must be placed in slabs, grids or blocks to improve the interaction with the surrounding clay, thanks to the overlapping of the columns.

The Swedish regulations also specify that in presence of excavations, the Lime-Cement columns should always have been placed in slabs.

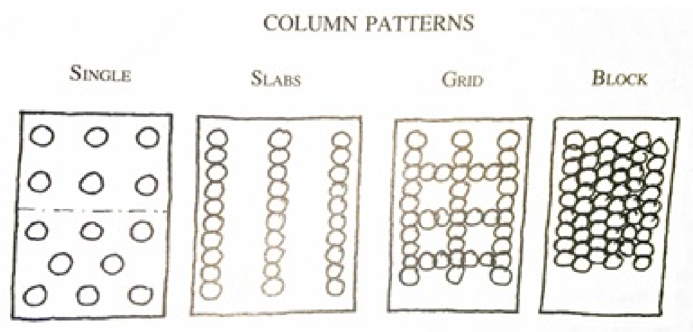


Figure 1.12: example of patterns for the Lime-Cement columns(SGF report 4:95, 2000).

1.3.4 Stabilizers.

For 600mm diameter columns, the quantity of lime-cement added is determined by some laboratory tests, in particular by the mixing trials, and it is approximately $80 - 130\ kg/m^3$, equal to $23 - 36\ kg/m$.
Anyway, some advices for the choice of the stabilizer for different type of soils are given and are basically defined by the stabilizing effect S_{eff} which represents the ratio between the shear strength of the stabilized soil and that of the un-stabilized soil:

$$S_{\text{eff}} = \frac{\tau_{\text{stab}}}{\tau_{\text{un-stab}}} \tag{1.3.1}$$

The main advices for clayey soils are:

- Clay containing gyttia: the addition of cement/lime produces a good effect, in fact $S_{\text{eff}} = 10 - 20$. Before the full load is applied, it is better to let through 3 months.

- Clay containing sulphides: there are large differents between clays in west and east Sweden, so it is important to perform mixing trials in each case. Anyway, lime-cement is recommended because usually produces higher strengths than lime only.

- Clay: is highly suitable for stabilization with lime and lime/cement. There's a good effect on the stabilization, in fact $S_{\text{eff}} = 10 - 20$.

- Clay with silt strata, silty clay: suitable for stabilization with lime and lime/cement. There's a good effect on the stabilization, in fact $S_{\text{eff}} = 10 - 20$. Anyway the results are better with the lime/cement than with lime alone.

Usually, before the reinforcement of the soil some mixing trials are performed in the laboratory, to determine the best ratio of lime-cement, the correct quantity of stabilizers and the improved properties of the soil. In Figure 1.13 there is the picture of a mixing trial conducted on Dalvägen clays, where can be notice the difference between the clay (on the right) and the clay mixed with lime and cement (on the left).

Figure 1.13: Mixing trial conducted on Dalvägen clays (courtesy of Sweco).

1.3.5 Construction.

For Lime-Cement columns the ideal value of advance is 15 $mm/revolution$ for clay containing gyttja and 20 $mm/revolution$ for the other soils. The speed of rotation for the mixing tool during admixture is in the range of $80-120$ $revolutions/minute$, but it is important to consider that normally we obtain a disturbed zone below the columns of approximately 0.5 m, and also a heterogeneous zone on the top. This is because the injection of binders is stopped at about $0.5-1$ m under the ground level, so this will result in a column top with varying properties.
Some of the most important properties for the Lime-Cement column machines are described in Table 1.5.

Characteristic	**Existing machines**
Machines weigth	12-39
Ground pressure installer with tank (kPa)	24-38
Ground pressure bulk trailer (kPa)	40-60
Column diameter (m)	0.4-1.0
Column depth (m)	<25
Lime/cement or cement/lime (%)	0/100-100/0
Sloping ground, max slope	1:7-1:11

Table 1.5: data for Lime-Cement Columns machines (SGF report 4:95, 2000).

1.3.6 Inspections.

Some inspections are always performed to check if the columns are constructed in accordance to specific requirements, as the correct placing of the columns, a correct length, a correct quantity of stabilizer and a correct uniformity of this along the columns and in their cross section.
The inspection is necessary to verify that the construction of the columns occurred accordingly to the initial project. A journal diary shall register some construction's informations as the quantity and type of stabilizer placed in the machine tank, so it will be possible to identify the column constructed with that stabilizer. Also any deviation from the project has to be reported in another specific journal.
Anyway, during the design of a soil reinforcement, some deviations are always considered and some tolerance are given. An example of deviation is when a column is not continuous until the depth of the project, or when a column is incorrectly located. Because of this, some tolerance requirements shall be set: length, position, inclination. If a columns doesn't comply in the tolerance requirements, an additionally column shall be placed and installed to obtain the wanted soil reinforcement.
The main tolerance requirements are:

- the permitted tolerance quantity of stabilizer mixed in is 10 %;
- for single columns the permitted deviation in plan shall be 20 % of d_{col}, 10 % of c or $0,1\ m$ and the maximum inclination permitted for a 10 m column is 15 mm/m;
- For columns in slabs, grids or blocks, the minimum overlap should be 50 mm and the distance between the columns centers in slab and blocks should not exceed 0,8 of d_{col}.

For single columns, if it contains less than the minimum quantity of stabilizer, it's shear strength should be checked. If the shear strength is low, it is possible that a new column replacing the one it is necessary. The decision is handed by a consultation with the client. On the other hand, for columns in slabs and grids, if one of them doesn't reach the tolerance requirements, certainly a new column is built to replace the old, because otherwise the function of the slab is compromised.

Some check calculations regarding the settlements and the stability are always necessary, and in particular the parameter that is most checked is the safety factor:

- SF before the column reinforcement;
- SF during load application to determine the maximum permissible load increment;
- SF during construction of the columns;
- SF when the embankment is completed.

The inspection shall regard columns from all representative soils and strata sequences inside the reinforced area, and a representative shear strength should be obtained for each stratum. The number of columns to be inspected depends on the reinforcement, the SF and the extent of the area reinforced, see Table 1.6.

Factor of safety un-stabilized embankment+load	**Extent of reinforcement**	**Inspection**
SF>1,0	<5000	no inspection or increase columns number
SF>1,0	>5000	1% of columns
SF>1,0	>50000	> 0,5% of columns
SF<1,0	regardless of extension	2% of columns

Table 1.6: inspections on the columns (SGF report 4:95, 2000).

In small works where the load is small it is not economically convenient to perform an inspection, but it's cheaper to install extra columns to increase the SF.
Several inspections methods exist, but in general the most used method is the "Conventional column penetration test" (KPS), which can be performed in a conventional or reverse mode. The specific tool of this test is a probe of 0.01 m^2 area fitted with vanes. The test permits to calculate the undrained shear strength of the constructed columns. The size of the vanes is different according to the column diameter, see Table 1.7.

Column diameter (mm)	Thickness of vanes (mm)	Width of vanes (mm)
500	20	400
600	15	500
800	15	600

Table 1.7: inspections on the columns (SGF report 4:95, 2000).

Moreover, the probe has a conical point of 50 mm diameter, a distance between the point and the vane attachment of 500 mm, a probe diameter of 36 mm or if the column is predrilled $50 - 65$ mm.
The main difference between the two mentioned modes of performing the method is the direction of sounding: in the conventional mode the probe is pushed down into the column, while in the reverse mode the probe is pulled up by a wire rope, as showed in Figure 1.14.

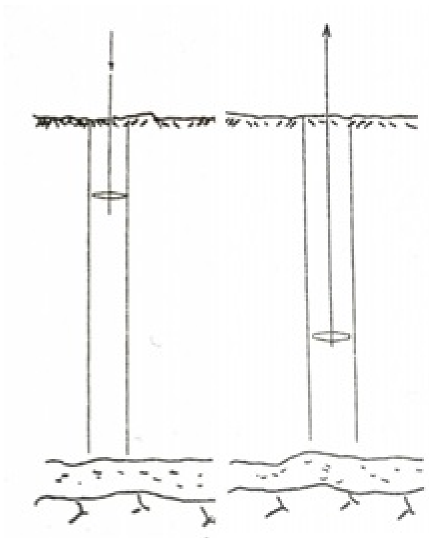


Figure 1.14: comparison between the conventional mode on the left and the reverse mode on the right (Holm, 1999).

In details, the different methods are:

- Traditional column test: used for columns of $500-800\ mm$ diameter, $8\ m$ of maximum length and maximum shear strength of $150\ kPa$. The traditional test is performed with the probe showed in Figure 1.15 at the center of the column, and it is pushed down at a constant rate of penetration of $20\ mm/s \pm 20\%$. The test ends at least $2\ m$ below the bottom of the columns, and it is performed also in the unstabilized clay.
  The shear strength of the columns can be calculated as 0.1 times the net pressure against the vanes, as in the following equation:

$$\tau_{\text{col}} = 0.1 \times \frac{Q}{A} \tag{1.3.2}$$

  where Q is the net penetration resistance measured during the test.

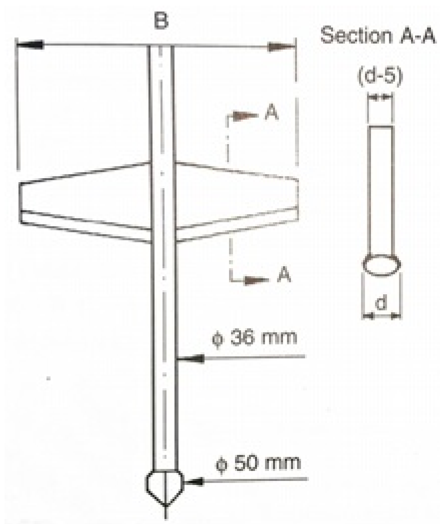


Figure 1.15: column probe for traditional test (SGF report 4:95, 2000).

- Reverse column test: the test is performed with a probe fitted with vanes showed in Figure 1.16, but in this case the probe is attached to a wire rope which is placed below the bottom of the columns. When the column reached the desired age the rope is pulled up and the probe rises in the surface while measuring the pressure.

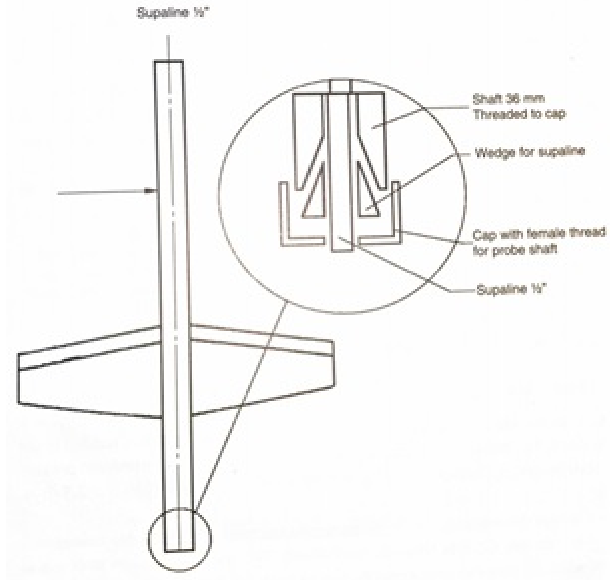


Figure 1.16: column probe for revers test (SGF report 4:95, 2000).

Method 1 is used for $500 - 1000\ mm$ columns diameter, the probe is installed immediately after the construction of the column, it can be used for column with maximum length of 15 m and 600 kPa maximum shear strength. The test is performed at a constant rate of 20 $mm/s \pm 20\%$. The skin resistance can be found with the use of a straight rope installed in some columns; the probe resistance is equal to the skin resistance against the rope. The shear strength of the column can be calculated as 0.1 times the net pressure on the probe.
The method 2 is performed when the column probe is installed prior to the columns construction and it can be used for column with maximum length of 20 m and 600 kPa maximum shear strength. The determination of the skin resistance, the net pressure and the shear strength is equal to the Method n.1.

Capitolo 2

SITE DESCRIPTION

The area of the study is located in northern Europe, on the Scandinavian Peninsula and in particular in Sweden. The definite area of the study is near Stockholm, see Figure 2.1, which is located on the eastern coast and is formed by a set of islands called Stockholm's archipelago.

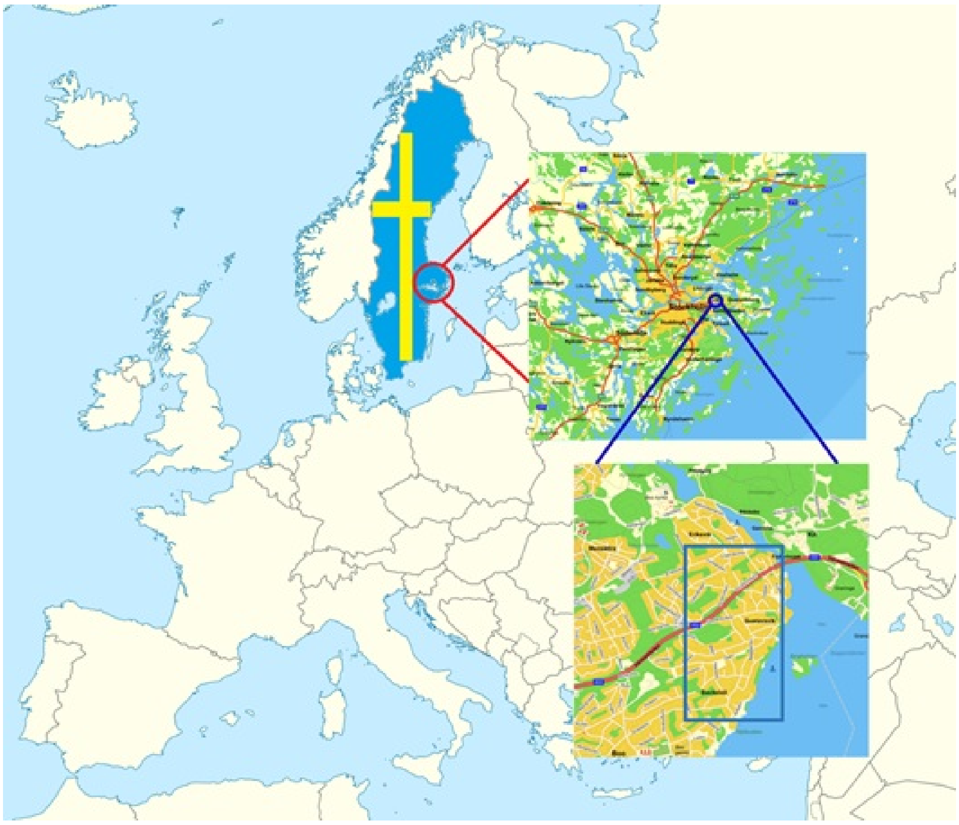


Figure 2.1: geographic location of Sweden and the specific area of the study.

2.1 Geological introduction.

Sweden is part of the Fennoscandian Shield also called Baltic Shield, where the bedrock is composed mainly of rocks corresponding to several periods: pre-Cambrian crystalline rocks, formed between the Earth's formation (4600 myr) and the Cambrian period (545 myr). These rocks compose the crystalline basement, formed through igneous and metamorphic processes.
During the Sweden geological history, several stages of magma intrusion and consequently solidification have been alternate or simultaneous to sedimentation stages. This succession made what is today's Scandinavia. According to the SGU (Geological Survey of Sweden), the oldest Swedish rocks are about 2500 millions of years old, and are located mostly in a limited area in the northern part of Sweden. In the remaining northern area and moving to the south the age of the rocks is around 1600 millions of years. These are metamorphosed rocks because of the Sveco-Fennian orogeny, that leads to the formation of a lot of the continental crust of the presents Sweden and Finland. The bedrock in the southern part of Sweden is made of 1700-1550 millions of years old rocks which were metamorphosed during the Sveco-Norwegian orogeny.
The types of metamorphic rocks that we currently find in Sweden are mostly gneiss, granite, marble and leptite, a rock formed from lava, that's very common in Sweden and Finland. In the Phanerozoic period (541 myr – present) sedimentary rocks were formed, such as sandstone, limestone and siltstone, that are now spread all over the Precambrian and Cambrian shield area. These rocks were formed thanks to the weathering and the erosion mainly in marine environments, in fact the sediments like clay, silt and lime consolidated and have been transformed into rocks. At present, most of these rocks formed before the last glacial period have been eroded.

2.1.1 Quaternary period.

The period that affected significantly both the geology, the landscape and the development of soils of Sweden is the Quaternary period. This has been decisive for the history because of the glaciations that were so important also because of the high latitude of Sweden.
The youngest glacial period reached the maximum expansion around 23000

years ago when Sweden was covered with a big ice sheet ($2 - 3$ km thickness) that pushed down the earth's crust with its weight. Around 17000 years ago a climatic change caused a slowly melting of the ice, and thanks to the disappearance of the pressure induced by the weight of the ice the land rose. In this period the formation of a peninsula between Denmark and Fennoscandia was fundamental for the development of the Baltic Ice Lake, whose water flowed in the sea around 11000 years ago, changing the composition and the present ions. Around 10000 year ago the ice was retreating 500 m per year, and around 8000 year ago the ice was far away from actual Stockholm, which was 150 m under the sea, notwithstanding the land was raising. 5000 years ago the sea level was 30 m above today's Stockholm's beach level. Currently the land continues to raise, with a rate of 4 $mm/year$ in Stockholm's land (SGU).
The current landscape of Sweden testifies the big influence of the last glaciation, in fact the presence of numerous lakes, the rough routes of the rivers, the rounded morphology of the landscape and also the type of soils are due to the glaciation period.

2.2 Soils of Sweden.

Regarding the soil of Sweden, a main division is based on two chronological periods: Pre-Quaternary and Quaternary, which is in turn divided into glaciation and post-glaciation periods. Another division is necessary between the soil formed from fresh-water sediments, as the sediments of the Baltic Ice Lake, and the soil formed from salt-water sediments, as the sediments deposited in a marine environment.

- The pre-Quaternary soils have been removed or heavily weathered by the ice sheet of the glaciation, so what today remains of these type of sediments is substantially rock decomposed to clay.

- The Quaternary soils of glacial origin are influenced by the bedrock: as seen before, the bedrock in Sweden is mainly of metamorphose or sedimentary origin. The sedimentary bedrock in the southern regions has been widely eroded, but it had a great importance in the formation of glacial deposits, in particular of boulders and till. Till is the most common type of soil in Sweden, in fact it occupies around 75% of the total area, and consists of material eroded and then transported by the ice sheet.

- The Quaternary soils of the post-glacial period are sand, clay and silt. Usually the clay is with high organic content, and contains layers of sand and silt.

Other types of soil that we can find in Sweden are:

- Peat: soil formed from the leftovers of plants, can vary in composition because of difference plants;
- Gyttja: is a sort of mud formed from the leftovers of plants and animals;
- Dry crust clay: particular type of clay that it is formed between the ground water surface and the ground surface, and is characterized by different properties from the normal clays;
- Friction soil or non-cohesive soil.

2.2.1 Clays.

The most important factor regarding the soils in Stockholm's area is the amount of the soft soils as clay that's very important and significant.
The clay is a type of soil, defined by the USCS (Unified Soil Classification System) as fine grained soils, where the 50% of the grains pass the No. 200 sieve which has a diameter of the openings of $0,075\ mm$. More than this, the clays are distinguished from the silt thanks to the liquid limit, which has to be less than 50%. The clays are composed of solid particles, which can be clay minerals, but also other type of minerals or particles.
The clay minerals are organized in layers, which can be composed of different structures, in particular tetrahedral or octahedral. The main 4 groups are: the kaolinite, the montmorillonite, the illite and the chlorite. They are composed of different structures, layers and cations and another factor that distinguishes the several groups is the basal spacing, see Figure 2.2.

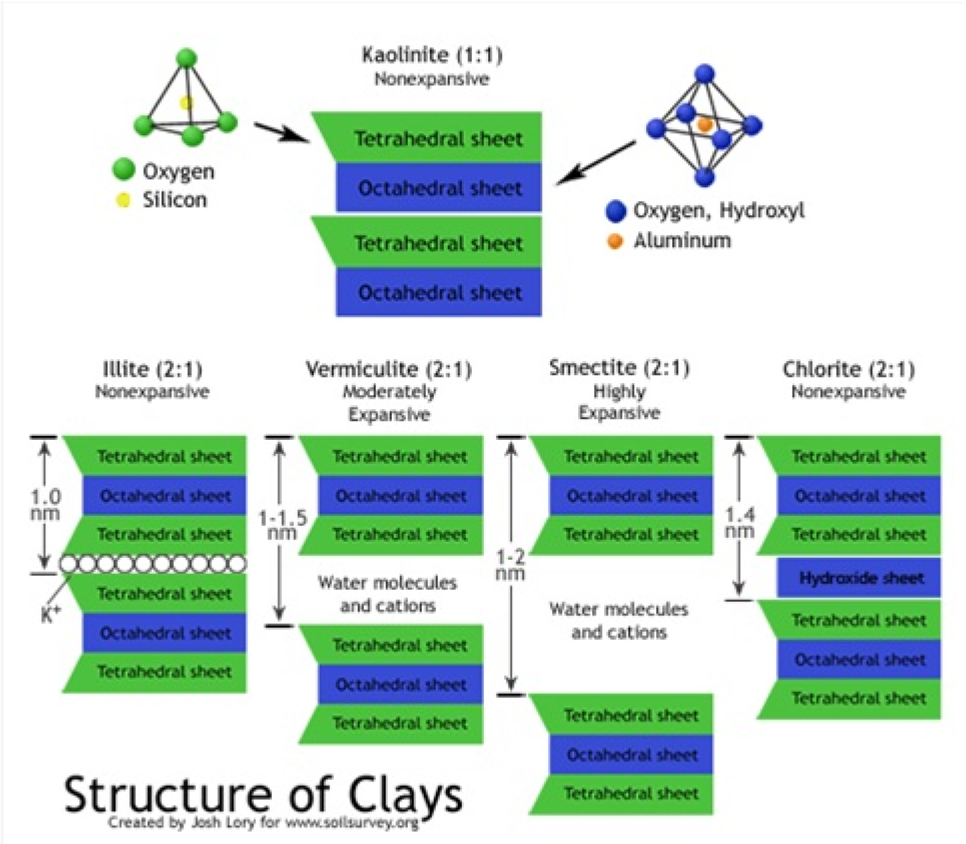


Figure 2.2: representation of the several groups of clay minerals, with the different structures (Soilsurvey.org).

The most common type of clay mineral in Sweden is the Illite (Åhnberg, 2006), which is composed of sets where tetrahedral and octahedral structures are alternated (T-O-T), and the sets are separated by potassium ions, see Figure 2.2.

The clay formation in Sweden is related to the last glaciation period, in fact the clay particles were deposited thanks to the glaciers and the ice rivers, which are very efficient means of transport for the sediments during the glacial periods. Due to the difference in energy, usually the coarser material as sand, gravel and stones is deposited close to the river mouth or the ice front, while the finer material as silt and clay is deposited above the coarser and far away from the river mouth or the ice front. For this reason, the clay sediments were deposited mainly in the sea or in lakes as post-glacial deposits.

Both the composition and the structure of the Swedish clay depend on the

ion concentration of the water where they are deposited, at the specific time of deposition. As seen before, the main chemical elements that build up the clays are silica and aluminum or magnesium. When the ice retreated the land rose above the sea level and the clay was subjected to leaching, which is a natural process in which dissolved substances as ions are removed and replaced. For this reason some of the ions in the clay were replaced by other and the surface of the clay mineral became negatively charged. After that, the clay mineral's surface attracted positive ions as the potassium we find in the illite clay (Rankka et al., 2004).
Currently all the clay we find in Sweden's underground causes settlements that's necessary to prevent, in fact the clayey soils are often problematic because of their poor resistance to deformation and low bearing capacity. Under a load which can be a road or a building, they tend to discharge the present water and to deform themselves through the process called consolidation. This process is very slow for the clays because of their low permeability, and depending by the coefficient of consolidation it can take up to several years for a complete consolidation. Obviously, thicker clay strata means bigger settlements, so a previous geotechnical model of the soil is necessary.
A sample of clay of Dalvägen area is showed in Figure 2.3.

Figure 2.3: Picture of the clay of Dalvägen area.

2.3 Dalvägen.

As already mentioned, the project of this work is located near Stockholm; the specific area of the work is in the Nacka municipality, in the eastern part of Stockholm. The project involves a big area, as showed in Figure 2.4, because it consists in the widening of all the highlighted roads.

Figure 2.4: framing of the specific area of the thesis: the main road Dalvägen (in purple), another important road Storsvängen (in blue), the minor roads in black.

In particular, the project concerns the widening of the roads to allow buses or trucks transit, the improvement of the local environment thanks to the connection with the municipal water works, the connection with the sewer system and also the construction of new residential buildings (Figure 2.5, which is reported even in the attachments.)

So, the work that will be performed in this area is the excavation of some meters of soil to permit the laying of the pipeline and the following filling to allow the construction of the roads. Pratically this results in a slope stability analysis necessary during the excavation and in a settlements calculation after the filling of the roads.

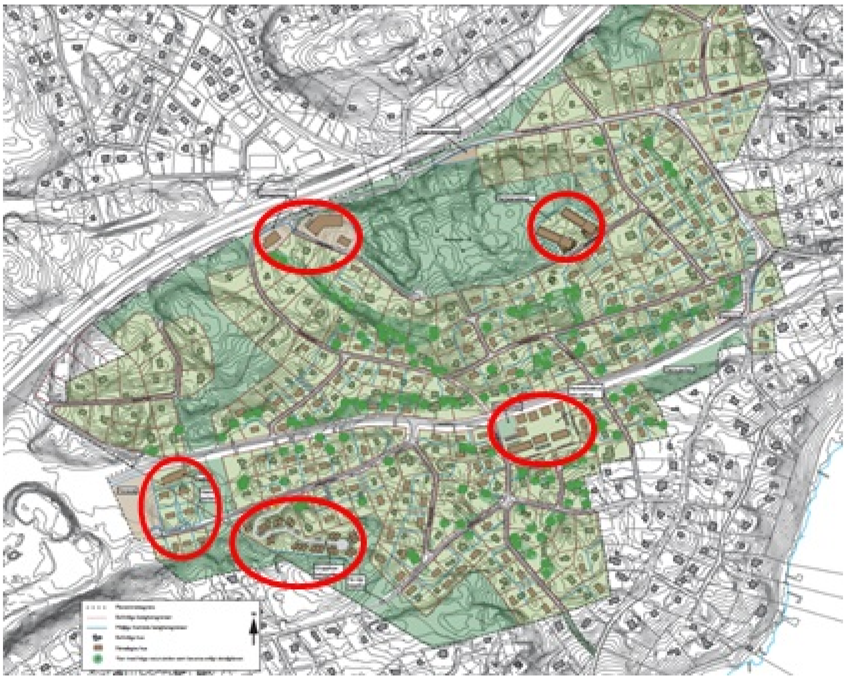


Figure 2.5: project of the area: in grey the existing houses, in brown the future houses. The major buildings are highlighted with the red circles.

Currently the area is designated as a summer residence, in fact the roads are narrow and the houses small (Figure 2.6 and Figure 2.7) but the project concerns its conversion in a permanent residential area.

Figure 2.6: picture focusing on the main road of the project, Dalvägen.

Figure 2.7: picture focusing on the road of the area and the houses as summer residences.

2.3.1 Geotechnical investigations.

The geotechnical investigations are always necessary to verify the soil conditions and properties, the presence of obstacles to the construction and the potential presence of clayey layers that are very common in Sweden. There are several type of surveys that have been conducted in the area, as listed (SGF, 2001):

- CPT or cone penetration test is a static test which consists in the penetration of a conical point in the soil at a constant rate of 20 mm/s. The results of the test are the point resistance (q_c), the frictional resistance (f_s) and sometimes the pore pressure u; In Figure 2.8 we can observe a typical result of a CPT test in the Dalvägen area: in the first graph there is the point resistance, which is measured starting from 1 m because usually the first meter is composed of made ground and/or dry crust clay. We can notice that the point resistance is quite low from 1 m to 8 m so this probably represents a clayey layer, while from 8 m to 10 m where the survey ends, the point resistance increases up to 12 MPa, so it probably represents a non-cohesive or a friction soil. In the second graph there is the frictional resistance, which results low in the clayey layer and increases when the frictional soil begins. In the third graph there is the pore pressure, which confirms the presence of the clayey layer: in fact, being the clay a cohesive soil it is usually not drained, so the pore pressure will increase during the survey. At a depth around 8 m the pore pressure falls because the presence of the friction soil that's usually drained.

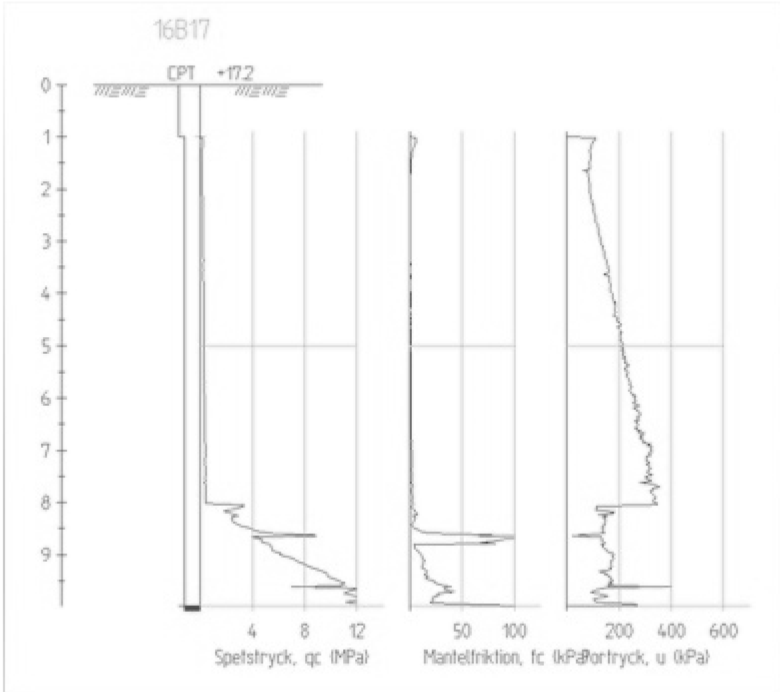


Figure 2.8: CPT results from a survey in Dalvägen.

The CPT results are useful in addition to the distinction of the several types of soil to establish some geotechnical parameters as the undrained shear strength of the cohesive soils, the friction angle and the relative density of the non-cohesive soils, the effective vertical stress, the pre-consolidation pressure and consequently the OCR degree.
The program used to elaborate the CPT results is Conrad, a software developed by the SGI (Swedish Geotechnical Institute) which is calibrated on the swedish clays. Conrad is based on the following

equation, whereby it is possible to obtain the shear strength:

$$c_u = \frac{q_t - \sigma_{\mathrm{v0}}}{13.4 + 6.65 \times w_L} \left(\frac{OCR}{1.3} \right)^{-0.2} \tag{2.3.1}$$

- Jb2 or soil/rock probing test is a drilling method (Fig. 2.9) which is executed with the rotation of the rig. This test is specific to determine the level of the bedrock.

Figure 2.9: execution of a Jb2 sounding in the Dalvägen area (picture taken in April 2016) and the detail of the button bit (indiamart.com).

The parameter registered during the test are the depth of drilling, the penetration resistance, the rate of penetration, the force input, the hammer pressure and the rotational pressure, as showed in 2.10. We can notice that the first meter is made ground because the hammer pressure (fifth graph) and the force input (sixth graph) are pretty high; from 1 m to around 1, 8 m there is dry crust clay because the penetration resistance (third graph) and the force input remain a bit high, while from 1, 8 to around 6, 5 m there is a clayey layer, in fact the previous values are now pretty low. The hammer pressure between 5 and 5, 5 m has high values, but probably it represents the presence of a boulder or stone. Under the clay there is the bedrock, marked by a specific symbol, and in fact the previous values are high.

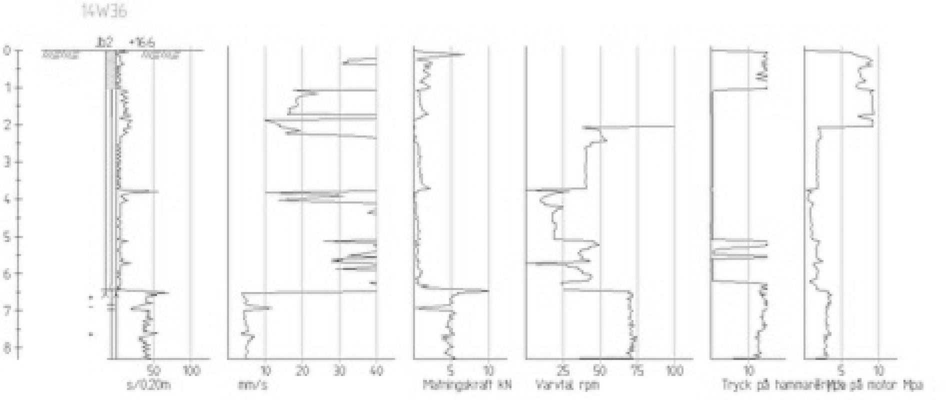


Figure 2.10: Jb2 results from a survey in Dalvägen.

- Slb or percussion sounding is a test where the penetration resistance is showed in a bar chart as the penetration time per depth interval ($sec./0,2\ m$). In Figure 2.11 we can observe a Slb test result: we can notice that the penetration resistance is low from 0 to around $2,5\ m$ so this probably represents a clayey layer, while from $2,5$ to over $3\ m$ the penetration resistance increases, representing a friction soil.

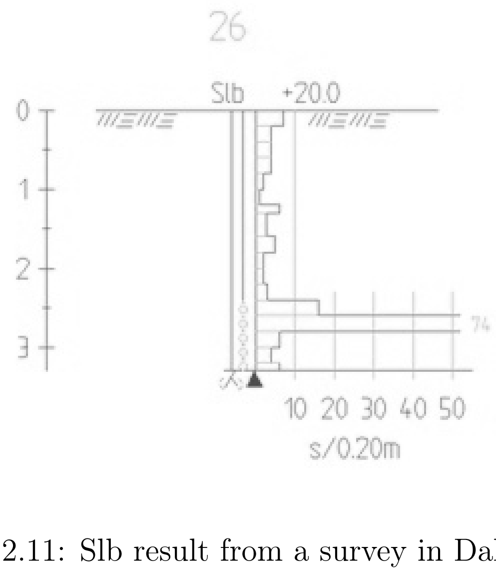


Figure 2.11: Slb result from a survey in Dalvägen.

- Tr or pyramid penetration test is conducted with a pyramid-shaped point attached to a sounding rod pushed down into the soil that measures the penetration resistance and sometimes the frictional resistance. In Figure 2.12 we can see the penetration resistance, which in the first meter it assumes a particular bell shape curve that is characteristic of the dry crust clay. From 1 to around $6,8$ m the resistance is pretty low, so this represents a clayey layer. After $6,8$ m and up to 9 m the resistance is high and there are also some shaded intervals which represent the turning of the rod, which is essential to permit the development of the survey. This high resistance and the rotation of the rod represent a friction soil.

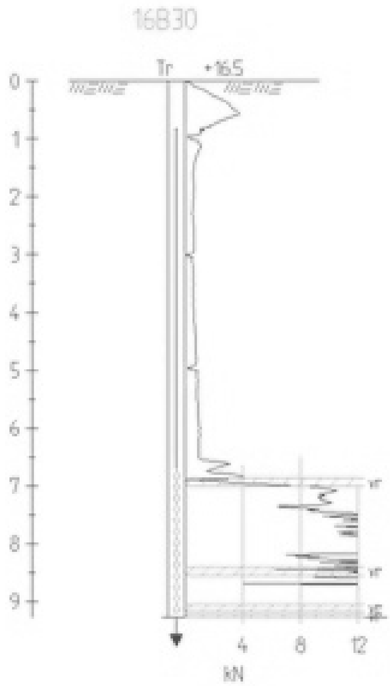


Figure 2.12: Tr result from a survey in Dalvägen.

- Vim or weight sounding is a test where the penetration resistance is registered as the applied load (where the soils are cohesive) or alternatively the number of half turns utilized measured as half-rotation/$0,2\ m$ (where the soils are non-cohesive). In Figure 2.13 we can see that from $0,5$ to around $9\ m$ the applied load is measured, which is very low because the soil is probably made of clay. After $9\ m$ the half-rotations every $0,2\ m$ are measured because the soil is a non-cohesive type.

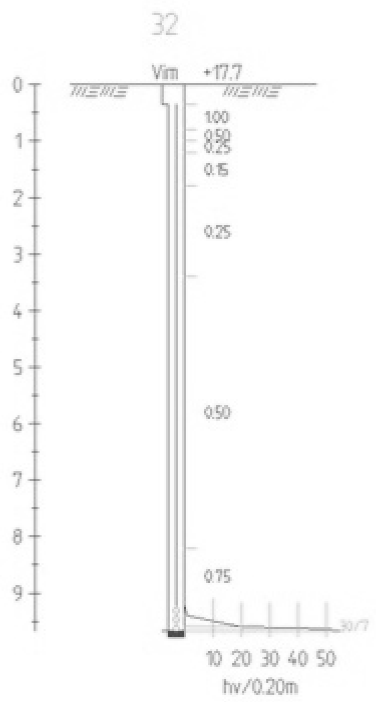


Figure 2.13: Vim result from a survey in Dalvägen.

- Vane test is a field test for estimate the undrained shear strength of the cohesive soils. The tool is composed of four vertical blades joined together to a shaft, as showed in Figure 2.14. The operating principle of the test is that the tool is forced to rotate with a specific torque moment M_t, from which it is possible to estimate the undrained shear strength, as following:

$$c_u = \frac{6 \times M_t}{7 \times \pi \times d^3} \tag{2.3.2}$$

where d is the vane width.

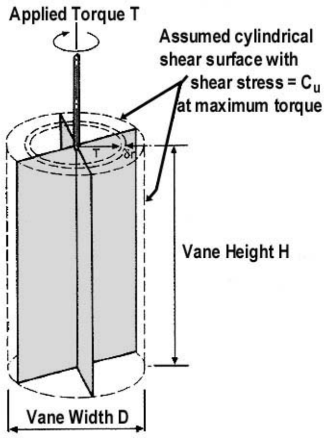


Figure 2.14: Tool of the vane test.

In the whole project more than 300 surveys have been conducted, which have been distributed between the several type of test as following:

- 113 Jb2 tests of which 106 Jb and 7 Jb2 + Vim;
- 86 Vim;
- 34 Slb;
- 29 Tr;
- 11 CPT tests;
- 11 CPT + Jb2 tests;
- 6 Vane tests;
- 4 CPT + Tr.

The Vim and Slb types have been conducted previously by another company, while the CPT, the Tr and the Jb2 have been recently conducted by the company where I had the internship. Additionally to the recent surveys, 12 piezometers have been installed to measure the level of the ground water table and soil samples were extracted from a total of 26 investigation points.

Given the huge area extension of the project, this work will now be focused only on Dalvägen, the main road, which is around 1200 m long. On this road 50 surveys have been performed recently (7 Jb2, 7 CPT, 4 CPT+Tr,

26 Tr, 6 Vane test) and 36 have been performed previously (4 Slb and 32 Vim).
Additionally, 4 piezometers have been installed, as showed in Figure 2.15.

Figure 2.15: figure representing the road Dalvägen (in green) and the position of the four piezometer (blue circles).

2.3.2 Geotechnical model of the soil.

One of the most important things when approaching a project, is the construction of the geotechnical model of the soil, which permits to have a specific framing on the characteristics of the soil and consequently on the factors that will affect significantly both the stability and the presence of settlements.
The framing of the specific area of the study has been obtained thanks to the analysis of all the surveys that have been conducted, the drawing of longitudinal and perpendicular sections to the road and the elaboration of the lab results that are all attached at the end of this work. The main parameters on which the geotechnical model is rested on are the thickness of the clay, the values of the undrained shear strength and the deformation parameters.
In particular, along Dalvägen the thickness of the clay varies from a minimum of 1 m up to a maximum of $9,5$ m, as showed in the attached sections. Having this type of clay thickness is strongly influencing the whole project, but in Sweden it is quite the normality and in fact the reinforcement with the Lime-Cement columns is very common. Anyway, the stratigraphy of the soil is usually composed of:

- $0,5-1\ m$ of made ground which is an artificial fill composed of asphalt, rubbish or plant remains;
- around $0,5-1\ m$ of dry crust clay, which is very common in Sweden;
- clay, whose thickness can vary between 1 and 9,5 m;
- few meters of friction or non-cohesive soil;
- bedrock, which is usually shallow because it often emerges up to the surface, as showed in Figure 2.16.

Figure 2.16: picture showing the emergence of the bedrock in the surface near Dalvägen.

The values of the undrained shear strength have been obtained from the elaboration of the CPT results, the vane tests and from some probe analysis, while the deformation parameters are obtained from the soil probe and the CRS results. All these files are attached at the end of this work.
It is important to notice that in the whole area the clay has a very low value of the undrained shear strength, in fact there is an average value around $12 - 15\ kPa$. In Sweden this low value is quite common and it is one of the reasons of the ordinary use of the Lime-Cement columns. More than this, in a restricted zone (see Section AA in the attachment) it has been impossible to conduct the surveys because the area was like a marsh (Figure 2.17) where the machines were sinking and in fact the value of the undrained shear strength was around $5\ kPa$.

Figure 2.17: picture showing the marsh were the machines were sinking, in Dalvägen.

These are very critical soil conditions, which didn't permit to perform the surveys, and for practical reasons I didn't considered this restricted area in this work. In fact, in special soil conditions as these, it is necessary to perform an excavation to determine if the soil conditions are so critical only near to the surface, or alternatively to assume that the geotechnical properties of that area are quite similar to the nearest areas.

Anyway, on the basis of the two parameters argued before, the thickness of the clay and the value of the undrained shear strength, I distinguished four different areas along Dalvägen, which are represented in Figure 2.18.

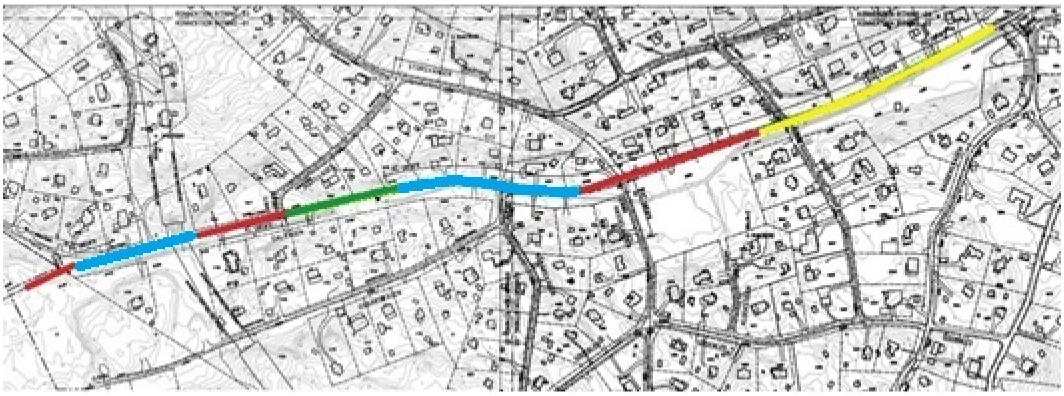


Figure 2.18: representation of the street Dalvägen divided into four zones.

The sections of the different zoned are attached. The red zone corresponds to the most critical area if considering the clay thickness ($8-10$ m), the green zone corresponds to the least critical ($1-4$ m), the blue is halfway ($3-6$ m), and the yellow is halfway too ($4-6.5$ m). The layering and the properties of the soil are then schematically represented in the figures and tables in the next pages, and are distinguished for the different areas:

- red area (Figure 2.20 and 2.21, Table 2.1 and 2.2);
- blue area (Figure 2.22 and 2.23, Table 2.3 and 2.4);
- green area (Figure 2.24 and 2.25, Table 2.5 and 2.6);
- yellow area (Figure 2.26 and 2.27, Table 2.7 and 2.8).

The soil model are represented with different symbols for each type of soil, that are represented in the legend in Figure 2.19.

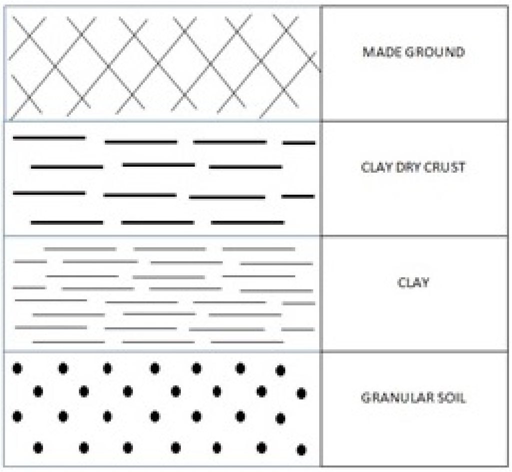


Figure 2.19: legend of the Figures 2.20, 2.22, 2.24 and 2.26

The level of the ground water table, actually, swings between 0 m (coinciding with the ground level) and around $1,5$ m. The permanence of the ground water table under the ground level permits the drying of the clay with the consequent formation of the clay dry crust, which is normally characterized by higher values of the un-drained shear strength.
It can be noticed that the blue and the yellow areas could be assembled together but differ significantly if considering the values of the undrained shear strength.

The created areas gather together different zones of Dalvägen that are characterized by similar properties. These area avoid the calculations of the slope stability in each point of survey, and permit to perform them only in four point, one for each color zones.

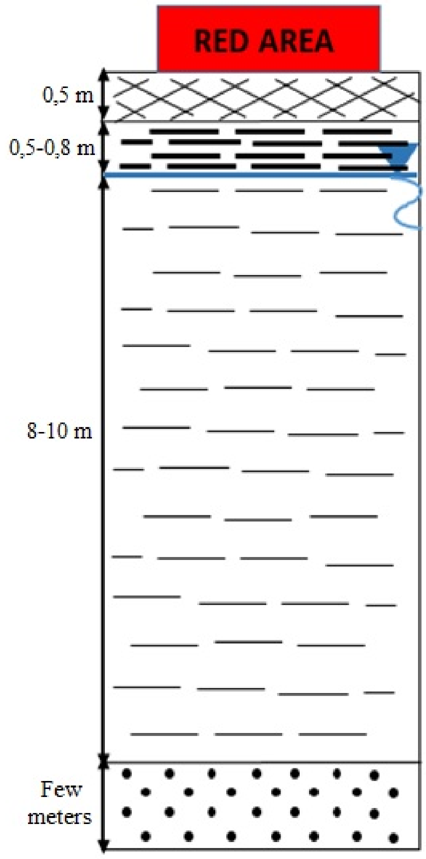


Figure 2.20: Figure representing the layering of the soil and the level of the GWT in the red area. The thickness of the clayey layer is between $8-10$ m.

The shear strength of the red area is an average value obtained from both the CPT results and the probe analysis, as showed in Figure 2.21, where the black line represents the adopted values of the shear strength. The value of the shear strength is obtained up to a depth of only 9 m because the points of the evaluations are the points with the minimum thickness of clay (8 m in this case).

The points beginning with 16 in Figure 2.21 are the points where the surveys have been conducted in 2016, the points with 14 are the points conducted in 2014 by another company and the points 30 and 14 are points where the surveys have been conducted before 2010.

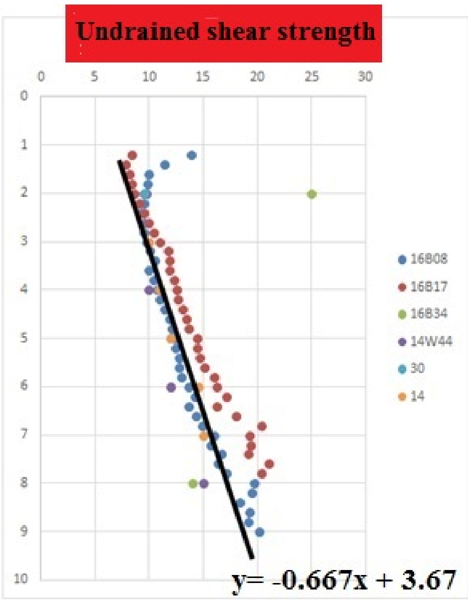


Figure 2.21: representation of the shear strength values obtained from 6 points.

RED AREA		
GRANULAR SOIL		
$\gamma(\frac{kN}{m^3})$	$c^{'}$(kPa)	$\phi^{(\circ)}$
19	0	36

Table 2.1: geotechnical properties of the granular soil in the red area.

The value of the friction angle has been obtained from the elaboration of the CPT, which are attacched at the end of this work.

RED AREA							
CLAY							
Depth (m)	γ $(\frac{kN}{m^3})$	τ_{fu} **(kPa)**	w_L (%)	M_L **(kPa)**	M_0 **(kPa)**	M'	**OCR**
1.5-3	17.5	8.9	48				
(2 m)	16.4	9.7	49	224	2401	14.9	1.52
3-6	16.8	12.3	46				
(4 m)	16.6	10	43	240	3375	18	1.03
(6 m)	16.6	12	47	230	3315	20.4	1.37
6-11.5	17.1	18.6	44				
(8 m)	16.9	15	49	544	3713	20.2	1.16

Table 2.2: geotechnical properties of the clay in the red area.

For calculating the settlements the parameters being used are the deformation properties (M_L, M' and M_0) which have been obtained from a CRS test in the point 14W44 and are referred to the specified depth.
On the other hand, to perform the slope stability analysis the parameter being used is the shear strength which referred to the depth interval.

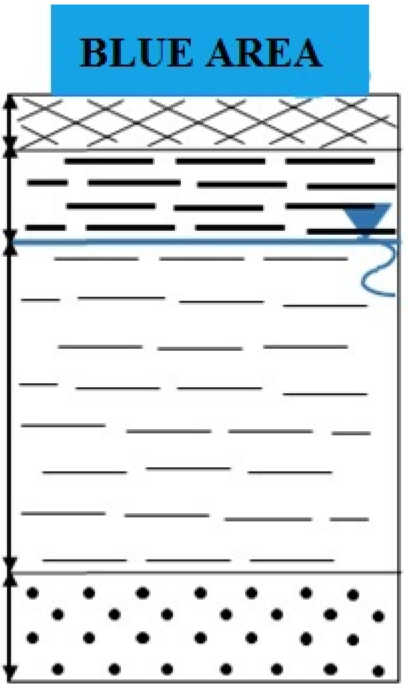


Figure 2.22: Figure representing the layering of the soil and the level of the GWT in the blue area. The thickness of the clayey layer is between $3-6$ m.

The shear strength of the blue area is an average value obtained from the CPT results conducted in 3 points, as showed in Figure 2.23, where the black line represents the adopted values of the shear strength. The value of the shear strength is obtained up to a depth of only 5 m because the points of the evaluations are the points with the minimum thickness of clay (6 m in this case). It is important to notice that the firsts points included between 1 m and $1,5$ m have a shear strength value around 20 kPa, so probably these are disturbed points due to the presence of the clay dry crust just above them. Even the first point of 14W17 at 2 m with a shear strength of around 45 kPa is probably disturbed because of the clay dry crust.

The points beginning with 16 are the points where the surveys have been conducted in 2016 while the points with 14 are the points conducted in 2014 by another company.

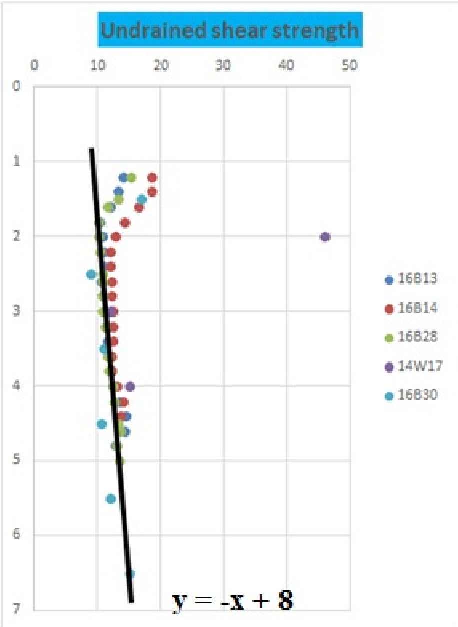


Figure 2.23: representation of the shear strength values obtained from 5 points.

BLUE AREA		
GRANULAR SOIL		
$\gamma(\frac{kN}{m^3})$	c'(kPa)	$\phi^{(\circ)}$
19	0	30

Table 2.3: geotechnical properties of the granular soil in the blue area.

The value of the friction angle has been obtained from the elaboration of the CPT, which are attacched at the end of this work.

BLUE AREA							
CLAY							
Depth (m)	$\boldsymbol{\gamma}$ $(\frac{\mathbf{kN}}{\mathbf{m^3}})$	$\boldsymbol{\tau_{fu}}$ **(kPa)**	$\boldsymbol{w_L}$ **(%)**	$\boldsymbol{M_L}$ **(kPa)**	$\boldsymbol{M_0}$ **(kPa)**	$\boldsymbol{M'}$	**OCR**
1.5-3	17.3	10.3	51				
(2 m)	18.4	46	49	4041	11386	13.2	7.67
3-7	16.9	13	49				
(3 m)	16.9	12	50	223	2803	17.7	1.52
(4 m)	17.0	15	60	760	2798	16.9	1.20

Table 2.4: geotechnical properties of the clay in the blue area.

For calculating the settlements the parameters being used are the deformation properties (M_L, M' and M_0) which have been obtained from a CRS test in the point 14W17 and are referred to the specified depth.
On the other hand, to perform the slope stability analysis the parameter being used is the shear strength which referred to the depth interval.

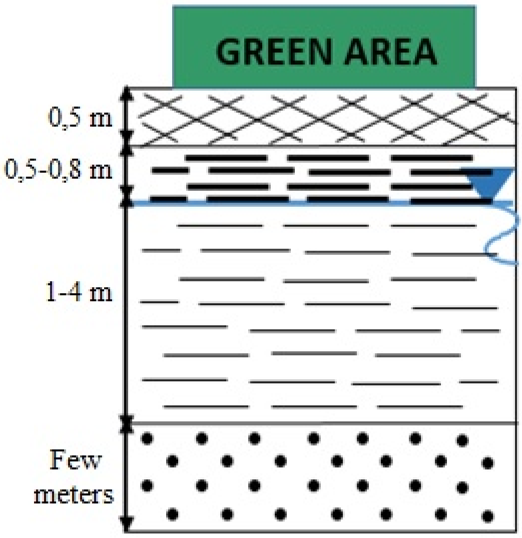


Figure 2.24: Figure representing the layering of the soil and the level of the GWT in the green area. The thickness of the clayey layer is between $1 - 4\ m$.

The shear strength of the green area is obtained from a vane test, and it is showed in Figure 2.25, where the black line represents the adopted values of the shear strength. The first point at a depth of 2 m has a shear strength value around 40 kPa, so probably this is a disturbed point due to the presence of the clay dry crust.
The deformations parameters of Table 2.6 have been obtained from a CRS test in a point in the red area which is next to the green area; it is assumed that the properties don't change.

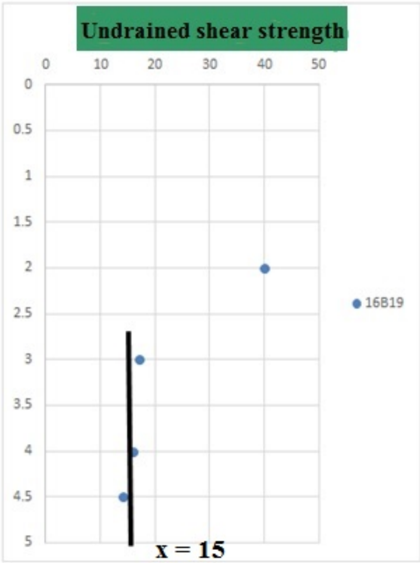


Figure 2.25: representation of the shear strength values of the green area.

GREEN AREA		
GRANULAR SOIL		
$\gamma(\frac{kN}{m^3})$	c'(kPa)	$\phi^{(o)}$
19	0	33

Table 2.5: geotechnical properties of the granular soil in the green area.

GREEN AREA							
CLAY							
Depth (m)	γ $(\frac{kN}{m^3})$	τ_{fu} (kPa)	w_L (%)	M_L (kPa)	M_0 (kPa)	M'	**OCR**
1-5	18.2	15	46				
(2 m)	17.6	9	44	339	2895	15	1.64
(3 m)	17.2	10	45	418	3062	15.6	1.30

Table 2.6: geotechnical properties of the clay in the green area.

For calculating the settlements the parameters being used are the deformation properties (M_L, M' and M_0) which have been obtained from a CRS test in the point 30 and are referred to the specified depth.

On the other hand, to perform the slope stability analysis the parameter being used is the shear strength which referred to the depth interval.

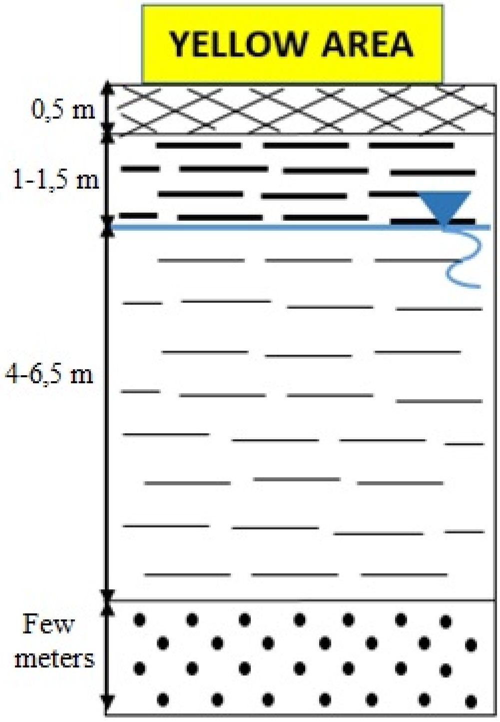


Figure 2.26: Figure representing the layering of the soil and the level of the GWT in the yellow area. The thickness of the clayey layer is between $4 - 6.5$ m.

The shear strength of the yellow area is an average value obtained from both the CPT results and the probe analysis, as showed in Figure 2.27, where the black line represents the adopted values of the shear strength. The value of the shear strength is obtained up to a depth of only $6,5$ m because the points of the evaluations are the points with the minimum thickness of clay (4 m in this case). It is important to notice that the firsts points included between 1 m and 2 m have a shear strength value of $20 - 30$ kPa, so probably these points are included in the clay dry crust.

The points beginning with 16 are the points where the surveys have been conducted in 2016, the points with 14 are the points conducted in 2014 by another company and the points 5 and 8 are points where the surveys have been conducted before 2010.

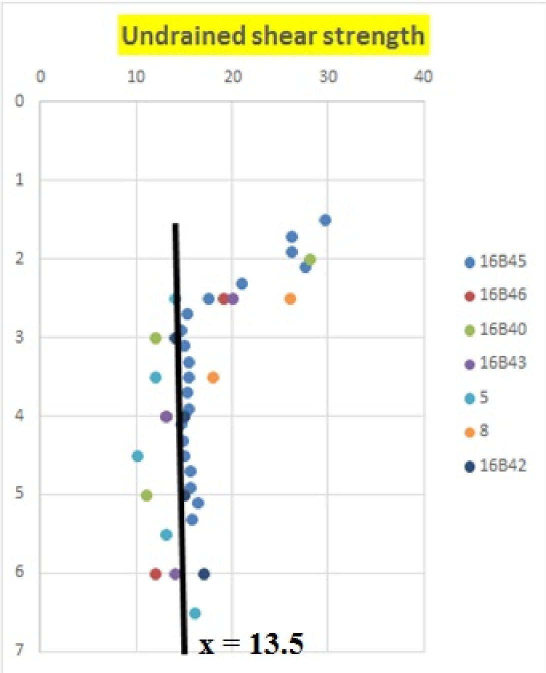


Figure 2.27: representation of the shear strength values obtained from 6 points.

YELLOW AREA		
GRANULAR SOIL		
$\gamma(\frac{kN}{m^3})$	c'(kPa)	$\phi^{(\circ)}$
19	0	34

Table 2.7: geotechnical properties of the granular soil in the yellow area.

The value of the friction angle has been obtained from the elaboration of the CPT, which are attached at the end of this work.

YELLOW AREA							
CLAY							
Depth (m)	γ $(\frac{\mathbf{kN}}{\mathbf{m^3}})$	$\boldsymbol{\tau_{fu}}$ **(kPa)**	$\boldsymbol{w_L}$ **(%)**	$\boldsymbol{M_L}$ **(kPa)**	$\boldsymbol{M_0}$ **(kPa)**	$\boldsymbol{M'}$	**OCR**
2-8.5	17.5	13.5	47				
(2.5 m)	17.5	19	48	802	4973	16.4	2.09
(4 m)	16.9	13	46	446	3783	12.4	1.17
(6 m)	17.9	12	42	349	4245	14.7	1.42

Table 2.8: geotechnical properties of the clay in the yellow area.

For calculating the settlements the parameters being used are the deformation properties (M_L, M' and M_0) which have been obtained from a CRS test in the point 16B46 and are referred to the specified depth.
On the other hand, to perform the slope stability analysis the parameter being used is the shear strength which referred to the depth interval.

Capitolo 3

SETTLEMENTS

The presence of an important quantity of clay in Sweden is the reason of the settlements. In particular in the Stockholm's area there are strong problems caused by the settlements, so a soil reinforcement is often necessary. The judgement to install or not the Lime-Cement columns depends on the amount of the settlements and on the stability of the soil, so one of the first goal to reach is the calculation of the settlements to verify if the bearing capacity of the soil is enough for the work that has to be built.

3.1 CRS test

To evaluate the settlements in a specific point, usually one or more samples are extracted from a probing. This sample is usually sent to the laboratory where it will be subjected to a CRS test. The CRS test is a sort of variant of the normal consolidation test and is currently widely used in Sweden. The main difference to the classic consolidation test is the fact that the test is done with a constant velocity of deformation. The name CRS reflects this reason, in fact it is the acronym of Constant Rate of Strain. The first people to use this test were Crawford in 1959, then Smith and Wahls in 1969 and Wissa in 1971.
The apparatus of the test (Figure 3.1) is very similar to the classic consolidation test: the specimen is 20 mm high and mounted in a Teflon ring that is inserted into a casing ring containing porous stones. The casing ring is mounted on the oedometer base where an O-ring seals the Teflon ring to make the drainage possible only at the top of the specimen. The oedo-

meter is placed in a compression machine and the specimen is deformed at a constant rate of strain (usually not exceeding a rate of $4 \times 10^{-5} mm/s$) (Larsson R., Sällfors G., 1986). At certain intervals the applied vertical pressure, the deformation and the pore pressure (measured at the bottom of the specimen) are measured.
The duration of the test depends basically on the type of clay, but normally it is around 24-30 hours.
The results of the test are represented in some graphs: $\sigma^{'} - \varepsilon$, $k - \varepsilon$, $\sigma^{'} - M$ and $\sigma^{'} - c_{\mathrm{v}}$.
$\sigma^{'}$ is calculated as

$$\sigma^{'} = \sigma - \frac{2}{3} u_{\mathrm{b}} \tag{3.1.1}$$

where u_{b} is the pore pressure at the undrained bottom of the specimen.

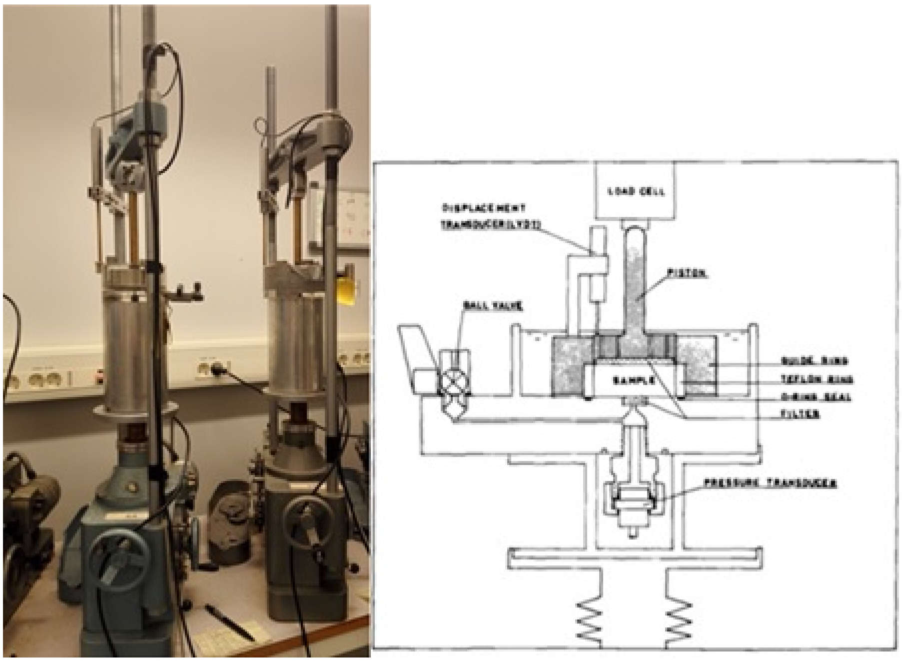


Figure 3.1: representation of the CRS apparatus (figure from Larsson R., Sällfors G., 1986, the picture is a courtesy of Sweco).

The parameters obtained from the test and needful for the settlements calculations are:

- density ρ;
- shear strength τ;
- water content w;

- permeability k;
- consolidation coefficient c_{v}: is the parameter that defines the trend of the settlements during time for the full consolidation. It is given by the equation:

$$c_{\mathrm{v}} = \frac{M_0 \times k}{g \times \rho_{\mathrm{w}}} \tag{3.1.2}$$

- pre-consolidation pressure σ_{C}': it is the highest pressure the specimen has ever been subjected to. It is obtained from the $\sigma' - \varepsilon$ graph (Figure 3.2). The two straight parts of the curve are extended until they intersect in a point. Then an isosceles triangle is inscribed between the formed lines and the curve; the intersection between the base of the triangle and the upper line is the pre-consolidation pressure, which is deviated from the previous curve of a factor called c;

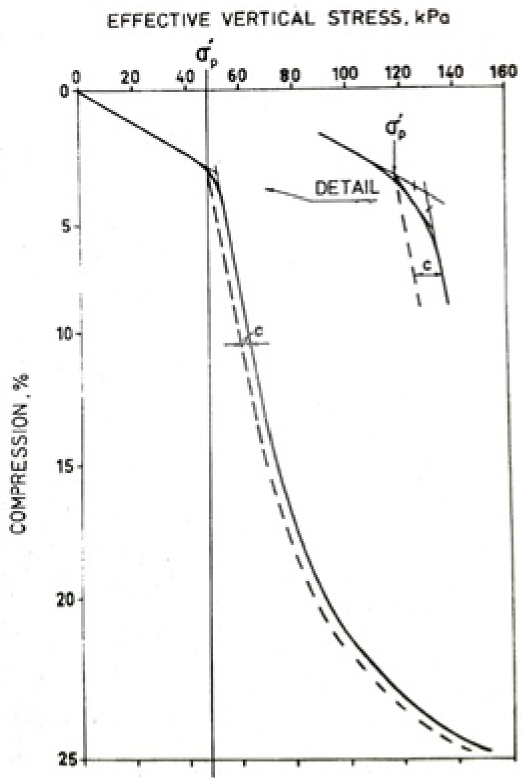


Figure 3.2: effective stress-deformation graph obtained from the CRS test (Larsson R., Sällfors G., 1986).

- modulus M_{L}: is the dominating parameter of the test (Figure 3.3). For value of the effective vertical pressure near to the pre-consolidation pressure, it tends to be constant; it depends by the previous loads and by the level of the ground water;

- limit pressure σ'_L: is the last value of pressure of the constant part of the modulus curve, just before the modulus starts to increase, see Figure 3.3;
- modulus number M': it represents the slope of the $\sigma' - M$ curve, in the part where the modulus is increasing linearly after the preconsolidation pressure.

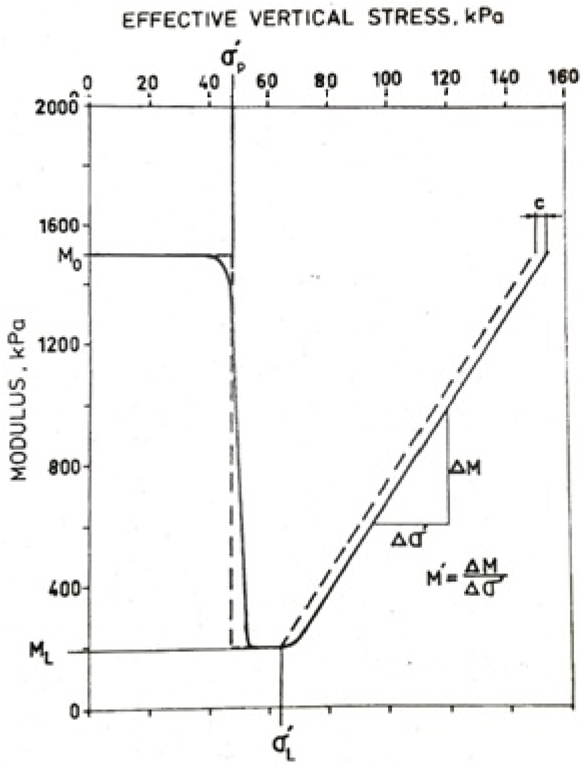


Figure 3.3: effective stress-modulus graph (Larsson R., Sällfors G., 1986).

3.2 Settlements calculation in the clay.

After the CRS test and the elaboration of the results, the settlements are calculated with the parameters obtained from the test. The equations for calculating the settlements are listed below, but the theory is basically comparable to the traditional consolidation or oedometric theory.
The following equations are utilized:

- if $\sigma^{'} < \sigma^{'}_{\mathrm{C}}$:

$$\varepsilon = \frac{\sigma^{'} - \sigma^{'}_{0}}{M_0} \tag{3.2.1}$$

- if $\sigma^{'}_{\mathrm{C}} < \sigma^{'} < \sigma^{'}_{\mathrm{L}}$:

$$\varepsilon = \frac{\sigma^{'}_{\mathrm{C}} - \sigma^{'}_{0}}{M_0} + \frac{\sigma^{'} - \sigma^{'}_{\mathrm{C}}}{M_{\mathrm{L}}} \tag{3.2.2}$$

- if $\sigma^{'} > \sigma^{'}_{\mathrm{L}}$:

$$\varepsilon = \frac{\sigma^{'}_{\mathrm{C}} - \sigma^{'}_{0}}{M_0} + \frac{\sigma^{'}_{\mathrm{L}} - \sigma^{'}_{\mathrm{C}}}{M_{\mathrm{L}}} + \frac{1}{M'} \times \ln\left(\frac{(\sigma^{'} - \sigma^{'}_{\mathrm{L}}) \times M^{'}}{M_{\mathrm{L}}} + 1\right) \tag{3.2.3}$$

where $\sigma^{'}_{0}$ is the effective vertical stress in situ and

$$\sigma^{'} = \sigma^{'}_{0} + \Delta\sigma^{'} \tag{3.2.4}$$

3.2.1 GS settlements.

The two ways used in this work to calculate the settlements of the unstabilized soil, which are based on the previous equations are:

- GeoSuite Settlements, a Novapoint software;
- An excel sheet.

An aspect to be remarked is that both the methods considers the clayey layer divided in some sub-layers and calculate the settlements for each of them. The total settlement of the entire layer of clay is then calculated by adding the contributions of each sub-layer settlement. Usually the division in sub-layers is based on the depths at which the CRSs have been performed, in fact the sub-layers are chosen in such a way that the midpoint(s) between the depths at which two consecutive CRSs have been performed

correspond to the limits between two consecutive sublayers.
As an example, if a layer of clay starts at 2 m depth, ends at 7 m and the CRS tests have been performed at a depth of 3.5, 4.5 and 5.5 m, the sub-layers will be from 2 to 4 m, from 4 to 5 $m$and from 5 to 7 m.
GS settlement starts with the stratigraphy of the soil, which can be fixed with different soil model properties. The one used in this work is the "Chalmers without creep" type, which permits to insert the fundamental properties of each layer, as the weight volume, the pre-consolidation pressure, the limit pressure, the compression modulus, the modulus before the pre-consolidation pressure, the modulus number and the consolidation coefficient. This soil model fits the oedometer curve as following (Fig. 3.4).

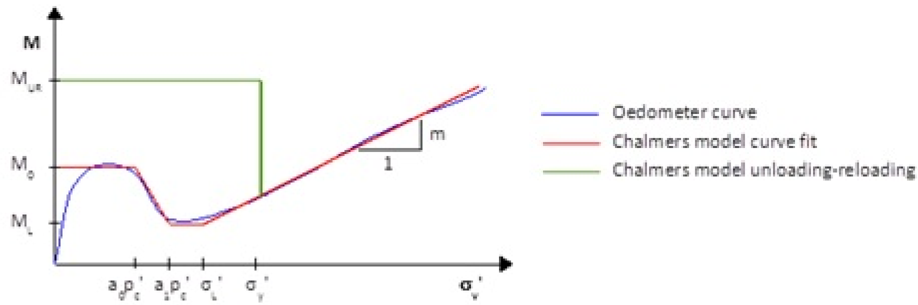


Figure 3.4: comparison between the oedometer and the Chalmers model curves.

The "Chalmers without creep" soil model is used to calculate settlements in fine grained soils such as clays and silts. After entering the soil parameters, it can be decided the level of the ground water or the pore water distribution, the loads applied on the site and the stress distribution model, which can be a "Finite Boussinesq" or an "Infinite Boussinesq". The stress distribution model adopted is the Infinite Boussinesq, which considers a strip load distribution bigger than the thickness of the clayey layer, so the total vertical stress increases by a quantity equal to the applied load.
After that some parameters as the effective vertical stress in situ and the pre-consolidation pressure are shown in graphs, in a stress chart. Finally the settlements can be calculated and a graph time-displacement is shown.
The other way to briefly calculate the settlement is an excel sheet which was developed by the company where I performed the internship. It permits to distinguish the layers that contribute to the settlements and to fill in the

grey cells with the parameters obtained from the CRS test. It calculates the OCR degree and the actual stress conditions, and shows it in a graph. At the end of the document the settlements are calculated and also a rate of them during time.

3.2.2 Resulting settlements in the clay

The settlements have been calculated in one point for each colour area, exept in the yellow area, where they have been calculated in two points, given that two CRS were avaiable. In particular the points are:

- RED AREA: point 14W44;
- BLUE AREA: point 14W17;
- GREEN AREA: point 30, which is in the red area but it is assumed that the deformation properties don't change given that the point 30 is next to the green area;
- YELLOW AREA: point 16B46 and point 16B40.

The points of calculation are represented in Figure 3.5 with the respective position and colour area.

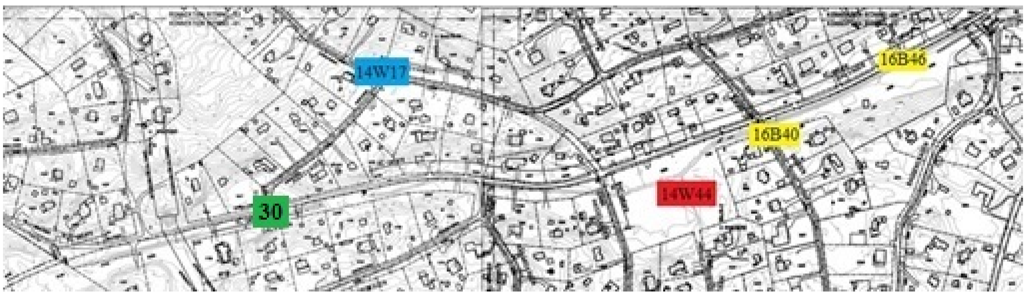


Figure 3.5: Representation of the points where the CRS tests have been conducted.

It can be observed that the points 14W44 and 14W17 are not exactly on the road Dalvägen (14W44 is 53 m far and 14W17 is 120 m far), but are a bit moved. Notwithstanding this, it can be assumed that they don't differ significantly from the points that are precisely on Dalvägen.

All the calculation with both the excel sheet and GS settlements are attached at the end of this work.

RED AREA

The red area is the most critical from every point of view because the thickness of the clay is quite high ($8-10\ m$) and at least for the firsts meters the value of the shear strength is quite low (less than 10 kPa).

The CRS tests are attached at the end of this work, but the deformation parameters influencing the settlements in the point 14W44 are reported in Table 3.1.

14W44						
Depth (m)	M_L **(kPa)**	M'	σ'_C **(kPa)**	σ'_L **(kPa)**	τ_{fu} **(kPa)**	**Settl. (Load 30 kPa) (m)**
2	224	14.9	38	52	9.7	0.148
4	240	18	39	83	10	0.229
6	230	20.4	70	96	12	0.095
8	544	20.2	75	106	15	0.083

Table 3.1: Results of the CRS test in the point 14W44.

The last column of the Table 3.1 reports the settlements for a load of 30 kPa in the sublayers $1-3\ m$, $3-5\ m$, $5-7\ m$, and $7-9.3\ m$.

The settlements have been calculated with both the excel sheet and the software Geosuite settlements and the results are reported in Table 3.2.

14W44		
	Excel	GS settlements
20 kPa	0.270	0.258
30 kPa	0.556	0.523
40 kPa	0.819	0.778
60 kPa	1.220	1.190

Table 3.2: Settlements expressed in m for in the point 14W44.

It can be notice that the results are quite similar for the two methods for each applied load. The resulting settlements are quite high, in fact for a load of 30 kPa which corresponds to a filling of 1.5 m they are around $52-55\ cm$.

In addition the rate of the settlements as a function of time is calculated and represented in Figure 3.6.

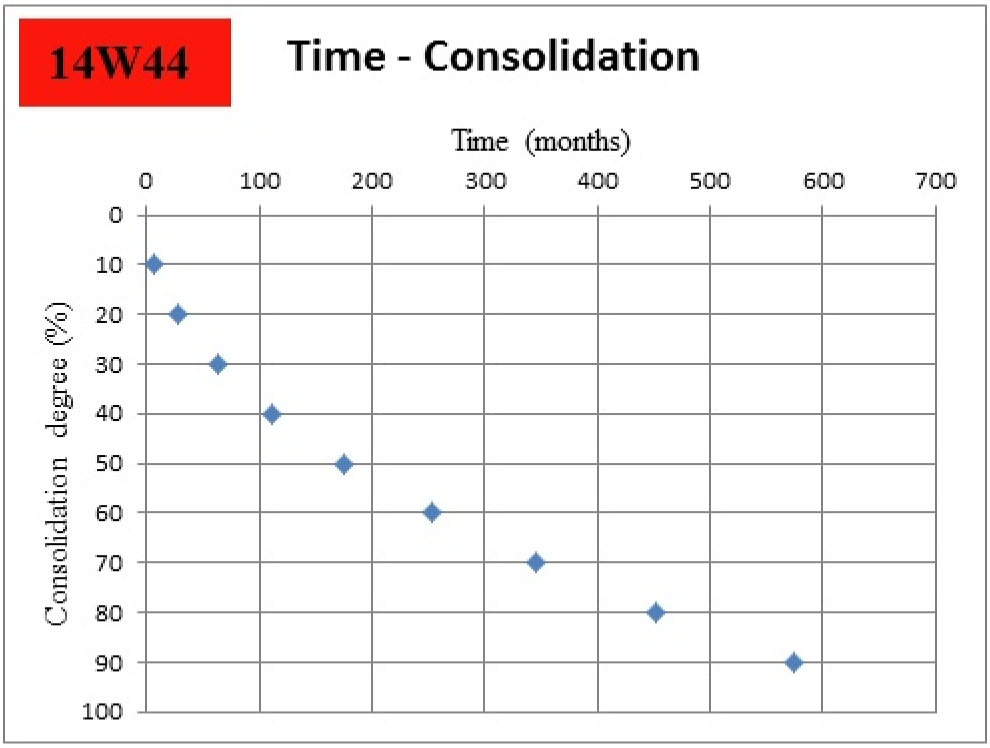


Figure 3.6: representation of the rate of the settlements with time in the point 14W44.

It can be noticed that to obtain a consolidation of 90% the required time is almost 600 months, corresponding to around 50 years.

BLUE AREA

The blue area is characterized by a thickness of the clay between $3-6\ m$ and by a value of the shear strength around $12\ kPa$.

The CRS tests are attached at the end of this work, but the deformation parameters influencing the settlements in the point 14W17 are reported in Table 3.3.

14W17						
Depth (m)	M_L **(kPa)**	M'	σ'_C **(kPa)**	σ'_L **(kPa)**	τ_{fu} **(kPa)**	**Settl. (Load 30 kPa) (m)**
2	4041	13.2	285	421	46	0.005
3	223	17.7	68	106	12	0.032
4	760	16.9	62	134	15	0.022

Table 3.3: Results of the CRS test in the point 14W17.

The last column of the Table 3.3 reports the settlements for a load of $30\ kPa$ in the sublayers $0.6-2.5\ m$, $2.5-3.5\ m$ and $3.5-4.3\ m$.
The settlements have been calculated with both the excel sheet and the software Geosuite settlements and the results are reported in Table 3.4.

14W17		
	Excel	GS settlements
$20\ kPa$	0.022	0.034
$30\ kPa$	0.059	0.076
$40\ kPa$	0.113	0.130
$60\ kPa$	0.222	0.239

Table 3.4: Settlements expressed in m for in the point 14W17.

It can be notice that the results are quite similar for the two methods for each applied load. The resulting settlements are quite low, in fact for a load of $30\ kPa$ which corresponds to a filling of $1.5\ m$ they are around $6-8\ cm$.

In addition the rate of the settlements as a function of time is calculated and represented in Figure 3.7.

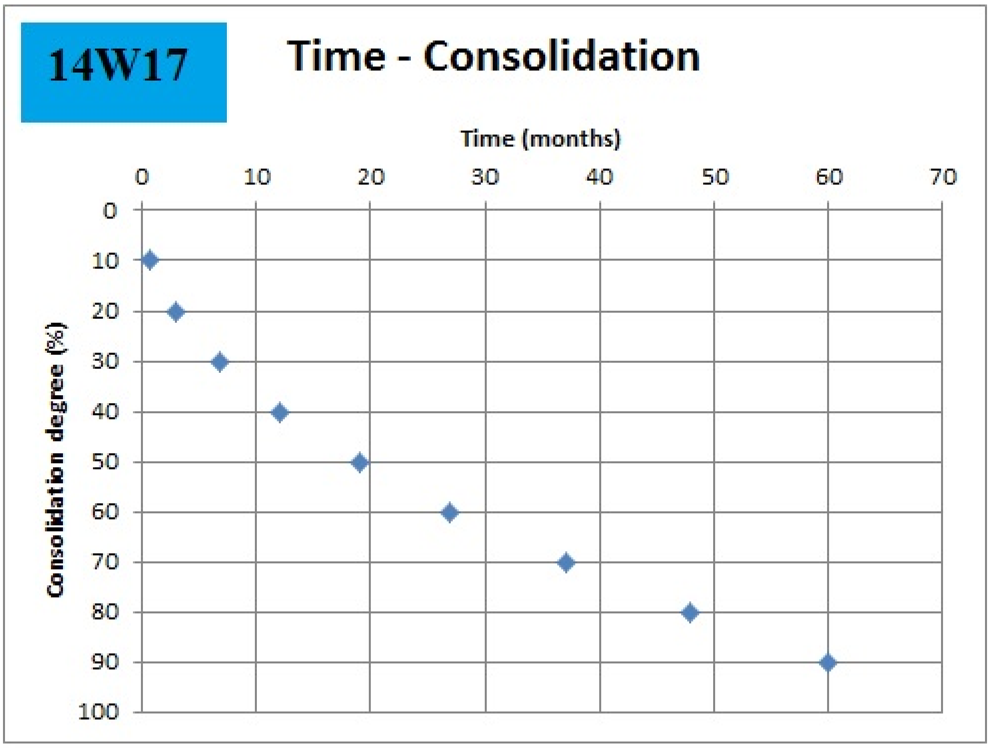


Figure 3.7: representation of the rate of the settlements with time in the point 14W17.

It can be noticed that to obtain a consolidation of 90% the required time is 60 months, corresponding to 5 years.

GREEN AREA

The green area is the less critical because the thickness of the clay is quite low ($1 - 4\ m$).
The CRS tests are attached at the end of this work, but the deformation parameters influencing the settlements in the point 30 are reported in Table 3.5.

30						
Depth (m)	**M_L (kPa)**	**M'**	**σ'_C (kPa)**	**σ'_L (kPa)**	**τ_{fu} (kPa)**	**Settl. (Load 30 kPa) (m)**
2	339	15	42	67	9	0.054
3	418	15.6	43	75	10	0.095

Table 3.5: Results of the CRS test in the point 30.

The last column of the Table 3.5 reports the settlements for a load of 30 kPa in the sublayers $1.2 - 2.5\ m$ and $2.5 - 4.5\ m$.
The settlements have been calculated with both the excel sheet and the software Geosuite settlements and the results are reported in Table 3.6.

30		
	Excel	GS settlements
20 kPa	0.068	0.097
30 kPa	0.149	0.183
40 kPa	0.231	0.269
60 kPa	0.366	0.411

Table 3.6: Settlements expressed in m for in the point 30.

It can be notice that the results are quite similar for the two methods for each applied load. The resulting settlements for a load of 30 kPa which corresponds to a filling of 1.5 m are around $15 - 18\ cm$.

In addition the rate of the settlements as a function of time is calculated and represented in Figure 3.8.

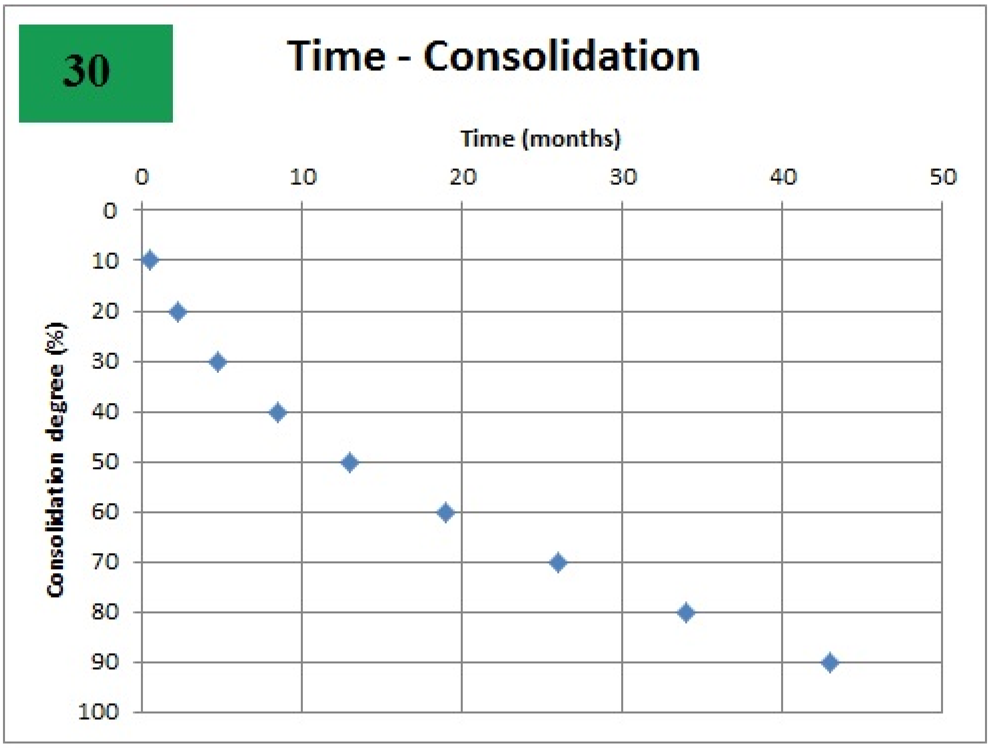


Figure 3.8: representation of the rate of the settlements with time in the point 30.

It can be noticed that to obtain a consolidation of 90% the required time is more than 40 months, corresponding to around 3.5 years.

YELLOW AREA

The yellow area is characterized by a thickness of the clay between $4-6.5\ m$ and by a value of the shear strength around $13.5\ kPa$.

The CRS tests are attached at the end of this work, but the deformation parameters influencing the settlements in the point 16B46 are reported in Table 3.7.

16B46						
Depth (m)	$\boldsymbol{M_L}$ **(kPa)**	$\boldsymbol{M'}$	$\boldsymbol{\sigma'_C}$ **(kPa)**	$\boldsymbol{\sigma'_L}$ **(kPa)**	$\boldsymbol{\tau_{fu}}$ **(kPa)**	**Settl. (Load 30 kPa) (m)**
2.5	802	16.4	83	135	19	0.007
4	446	12.4	59	94	13	0.083
6	349	14.7	93	111	12	0.027

Table 3.7: Results of the CRS test in the point 16B46.

The last column of the Table 3.7 reports the settlements for a load of $30\ kPa$ in the sublayers $2-3.25\ m$, $3.25-5\ m$ and $5-8\ m$.
The settlements have been calculated with both the excel sheet and the software Geosuite settlements and the results are reported in Table 3.8.

16B46		
	Excel	GS settlements
$20\ kPa$	0.064	0.068
$30\ kPa$	0.117	0.115
$40\ kPa$	0.238	0.228
$60\ kPa$	0.473	0.540

Table 3.8: Settlements expressed in m for in the point 16B46.

It can be notice that the results are quite similar for the two methods for each applied load. The resulting settlements are low, in fact for a load of $30\ kPa$ which corresponds to a filling of $1.5\ m$ they are around $11-12\ cm$.

In addition the rate of the settlements as a function of time is calculated and represented in Figure 3.9.

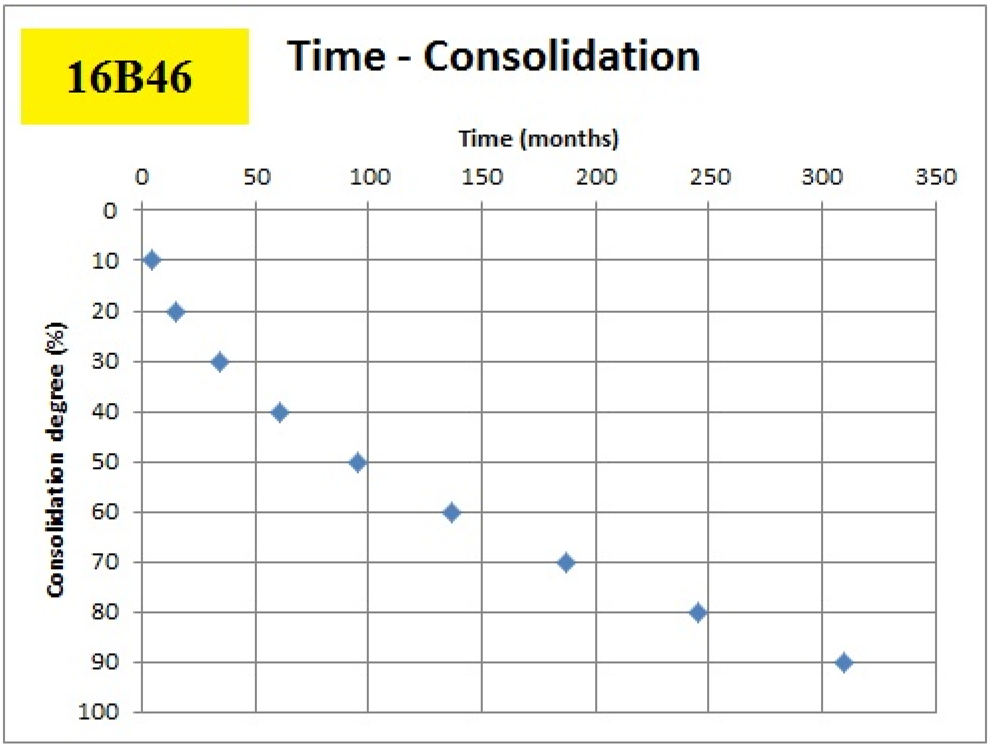


Figure 3.9: representation of the rate of the settlements with time in the point 16B46.

It can be noticed that to obtain a consolidation of 90% the required time is more than 300 months, corresponding to around 25 years.

Given that a CRS test has been performed also in the point 16B40, the settlements have been calculated for this point.
The deformation parameters influencing the settlements in the point 16B40 are reported in Table 3.9.

16B40						
Depth (m)	$\boldsymbol{M_L}$ **(kPa)**	$\boldsymbol{M'}$	$\boldsymbol{\sigma'_C}$ **(kPa)**	$\boldsymbol{\sigma'_L}$ **(kPa)**	$\boldsymbol{\tau_{fu}}$ **(kPa)**	**Settl. (Load 30 kPa) (m)**
2	3343	10.2	130	314	28	0.005
3	313	16.2	52	73	12	0.056
5	252	16.2	58	77	11	0.127

Table 3.9: Results of the CRS test in the point 16B40.

The last column of the Table 3.9 reports the settlements for a load of 30 kPa in the sublayers $1.5 - 2.5$ m, $2.5 - 4$ m, and $4 - 5.8$ m.
The settlements have been calculated with both the excel sheet and the software Geosuite settlements and the results are reported in Table 3.10.

16B40		
	Excel	GS settlements
20 kPa	0.073	0.109
30 kPa	0.188	0.208
40 kPa	0.294	0.306
60 kPa	0.432	0.462

Table 3.10: Settlements expressed in m for in the point 16B40.

It can be notice that the results are quite similar for the two methods for each applied load. The resulting settlements for a load of 30 kPa which corresponds to a filling of 1.5 m are around $18 - 20$ cm.

In addition the rate of the settlements as a function of time is calculated and represented in Figure 3.10.

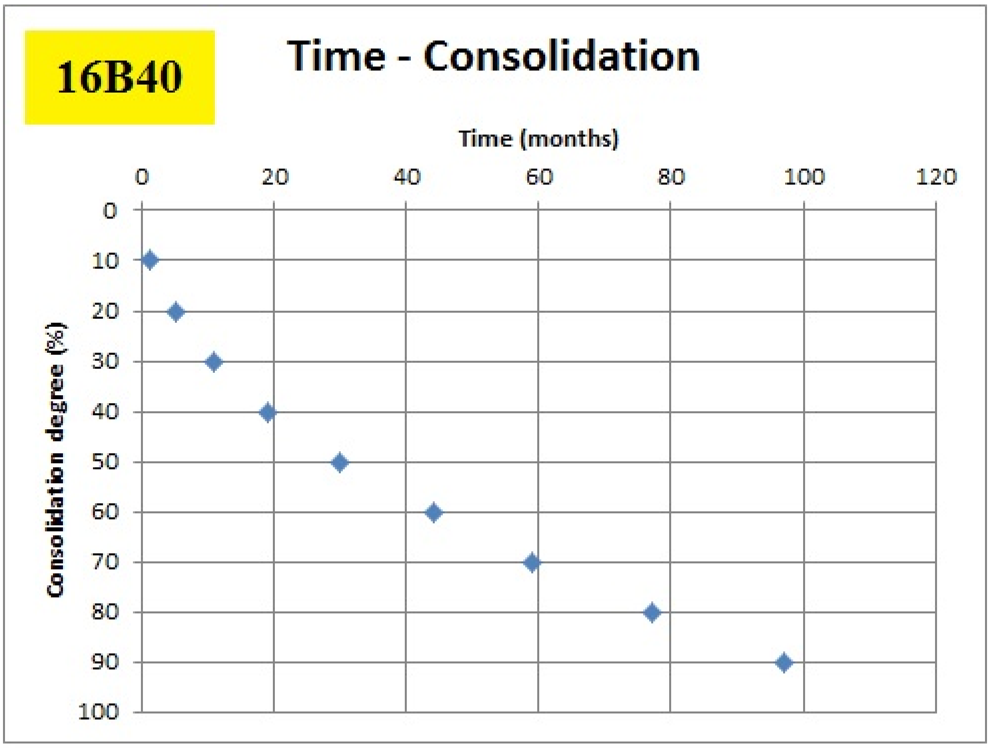


Figure 3.10: representation of the rate of the settlements with time in the point 16B40.

It can be noticed that to obtain a consolidation of 90% the required time is almost 100 months, corresponding to around 8 years.

The settlements of all the areas are then compared in Table 3.11, where it is possible to notice that the highest settlements for every load are in the red area, while the lowest are in the blue area and not in the green one. The reason of this is mainly due to the OCR degree, as will be then discussed.

14W44		
	Excel	GS settlements
20 kPa	0.270	0.258
30 kPa	0.556	0.523
40 kPa	0.819	0.778
60 kPa	1.220	1.190
14W17		
	Excel	GS settlements
20 kPa	0.022	0.034
30 kPa	0.059	0.076
40 kPa	0.113	0.130
60 kPa	0.222	0.239
30		
	Excel	GS settlements
20 kPa	0.068	0.097
30 kPa	0.149	0.183
40 kPa	0.231	0.269
60 kPa	0.366	0.411
16B46		
	Excel	GS settlements
20 kPa	0.064	0.068
30 kPa	0.117	0.115
40 kPa	0.238	0.228
60 kPa	0.473	0.540
16B40		
	Excel	GS settlements
20 kPa	0.073	0.109
30 kPa	0.188	0.208
40 kPa	0.294	0.306
60 kPa	0.432	0.462

Table 3.11: Settlements of all the areas, expressed in meters.

Some observations and remarks regarding the parameters that mostly affect the settlements are necessary. In particular:

- the thickness of the soft soils as the clay. Usually and obviously, more clay means more settlements, but this is not always true. In fact, it depends also on the other parameters.

- the OCR degree, which depends on the pre-consolidation pressure and on the actual effective stress. In fact, higher the pre-consolidation pressure, higher the OCR degree, and consequent smaller settlements. More than this also the level of the ground water table influences the OCR degree, because if the GWT rises, the water pressure rises too, the effective stress decreases because the total stress $\sigma = \sigma^{'} + u$ is constant (Terzaghi's principle). Consequently, smaller effective stress means higher OCR so less settlements. On the opposite side, if the GWT lowers, the OCR decreases because the water pressure is lower so the effective stress is higher.

- the value of the Modulus: higher M_{L} means smaller settlements, while on the opposite side lower M_{L} means higher settlements.

In the tables showing the CRS results (Table 3.1, Table 3.3, Table 3.5, Table 3.7 and Table 3.9), the last column reports the settlements for each sub-layer for a load of 30 kPa, so it is possible to better compare the parameters influencing the settlements.
Looking at the Table 3.1 we can see that for the firsts two layers the settlements are quite high, while for the other two the settlements are very low. The reason of this difference is basically due to the higher value of the pre-consolidation pressure (which implicates an higher OCR) and to the higher value of the Modulus for the last layer.
Looking at the Table 3.3 we can now see that for the first layer the settlement is very low while for the other two layers the settlements are higher. The reason of this difference is basically due to the fact that the Modulus, the pre-consolidation pressure and the limit pressure of the first layer are very high, and they are probably due to the presence of the clay dry crust. This is confirmed from the value of the shear strength at that depth, which is 46 kPa.

So, the two main parameters that influence the settlements are the pre-consolidation pressure and the Modulus. But which of the two has more influence? For this purpose a sensitivity analysis is necessary.
Considering the three equations of the settlement calculations, equation 3.2.1, 3.2.2 and 3.2.3, we can notice that the pre-consolidation pressure is always in the numerator while the Modulus is always in the denominator. This imply that if considering for example the equation 3.2.2, the partial derivative of the deformations ε with respect to the pre-consolidation pressure $\sigma_{\mathrm{C}}^{'}$ is equal to:

$$\frac{\partial \varepsilon}{\partial \sigma_{\mathrm{C}}^{'}} = \frac{1}{M_0} - \frac{1}{M_{\mathrm{L}}} \tag{3.2.5}$$

that means that fixed M_{L} and M_0 and changing $\sigma_{\mathrm{C}}^{'}$, ε changes linearly.
On the other hand, the partial derivative of the deformations ε with respect to the Modulud M_{L} is equal to:

$$\frac{\partial \varepsilon}{\partial M_{\mathrm{L}}} = \frac{\sigma_{\mathrm{C}}^{'} - \sigma^{'}}{M_{\mathrm{L}}^2} \tag{3.2.6}$$

that means that fixed $\sigma_{\mathrm{C}}^{'}$ and $\sigma^{'}$ and changing M_{L}, ε changes according to a inverse quadratic function that means that the deformation ε changes more as much as the modulus is small.

Another important consideration is the evaluation of the rate of the settlements as a function of time:

- 30: after 12 months the consolidation is at 48%;
- 14W17: after 12 months the consolidation is at 40%;
- 16B40: after 12 months the consolidation is at 32%;
- 16B46: after 12 months the consolidation is at 18%;
- 14W44: after 12 months the consolidation is at 13%.

The difference is mainly due to the distinct values of the consolidation coefficient, which is higher for the point 30 ($c_v = 1.56 \times 10^{-08} \frac{m^2}{s}$), lower for the point 14W44 ($c_v = 7.3 \times 10^{-09} \frac{m^2}{s}$) and mean for the others:

14W17 ($c_v = 1.39 \times 10^{-08} \frac{m^2}{s}$), 16B40 ($c_v = 1.16 \times 10^{-08} \frac{m^2}{s}$) and 16B46 ($c_v = 7.06 \times 10^{-09} \frac{m^2}{s}$). The consolidation coefficients have been obtained by the CRS tests.

In fact, higher consolidation coefficient means higher T according to:

$$T = \frac{c_v \times t}{H^2} \tag{3.2.7}$$

where t is the time we want to know the consolidation degree and H is the drainage length, considered as half of the thickness of the clayey layer. Finally, T is a factor depending on U, the consolidation degree: higher T means higher U and vice versa.

To obtain a consolidation of 90% the required time is:

- 30 (green): 3,5 years.
- 14W17 (blue): 5 years;
- 16B40 (yellow): 8 years;
- 16B46 (yellow): 26 years;
- 14W44 (red): 48 years.

It can be noticed that the required time for the whole consolidation is very high especially for the points 16B46 and 14W44, so a stabilization is quite necessary.

Capitolo 4

SLOPE STABILITY

In a geotechnical project, the slope stability is one of the main factors to check, because a soil reinforcement is often necessary. For this reason and next to the settlement calculations, the slope stability is one of the most important examination.
The slope stability analysis essentially deals with the formation of landslides, which are connected with both geological and geotechnical reasons. The formation of a landslide happens when the stabilising forces are smaller than the destabilising forces that act on the soil. The parameter that describes this relationship is the safety factor, which is the ratio between the stabilising and the destabilising forces, as showed in equation 4.0.1.

$$SF = \frac{stabilisingforces}{destabilisingforces} \tag{4.0.1}$$

An increase of the destabilising forces can be due for example to the erosion of the slope, to a raising of the ground water table or to the increase of the load on the slope. On the other hand, the main parameter that influences the stabilising forces is the shear strength, which obviously depends on the type of soil. The clay is the soil that has the lowest values of shear strength, so the slope stability analysis in Dalvägen area is quite necessary.

For definite slope as the embankments or the excavations, the theory that is usually applied is the the global limit equilibrium. The assumptions of the method are:

- circular slip surface;

- plain strain;
- plastic behaviour of the soil;
- Mohr Coloumb theory.

More than this, it supposes that the soil doesn't deform until the break which corresponds to the formation of a landslide. For this reason, the break will be a clear-cut because it will separate the stable soil and the unstable soil.
A more simplification is given by the method of slices which divides the unstable area in several slices, as showed in Figure 4.1 and calculates the stability for each one of them.

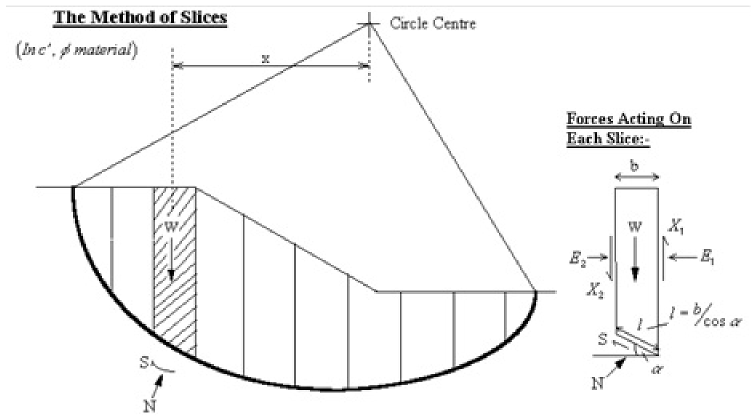


Figure 4.1: representation of a slope divided in slices (Connolly, 1997). On the right there's the representation of the force acting on each slice.

4.1 Bishop and Fellenius.

The Fellenius or Swedish method considers equal to zero all the normal forces to the sliding surface acting on the lateral sides, so considering the figure above, $E_1 = E_2 = X_1 = X_2 = 0$. It results that the destabilising force S_s is equal to:

$$S_\mathrm{s} = W_\mathrm{s} \times \sin\alpha \tag{4.1.1}$$

while the stabilising force N_s is equal to:

$$N_\mathrm{s} = W_\mathrm{s} \times \cos\alpha \tag{4.1.2}$$

The stabilizing moment will be:

$$M_{\text{stab}} = \sum_{i=1}^{N} N_{\text{si}} \times R_{\text{s}} \tag{4.1.3}$$

where R_s is the radius of the sliding surface.
Depending the stabilising force by the shear strength of the soil at the failure surface and by the weight of the slice in the perpendiculat direction to the sliding surface, it results:

$$M_{\text{stab}} = \sum_{i=1}^{N} [c_{\text{si}} l_{\text{si}} + N_{\text{si}} \times \tan \phi_{\text{i}}] \tag{4.1.4}$$

while depending the un-stabilizing moment by the weight of the slice in the tangent direction to the sliding surface, it results:

$$M_{\text{un-stab}} = \sum_{i=1}^{N} S_{\text{si}} \times R_{\text{s}} = \sum_{i=1}^{N} [W_{\text{si}} \times \sin \alpha_{\text{i}} \times R_{\text{s}}] \tag{4.1.5}$$

The safety factor will then result:

$$SF = \frac{M_{\text{stab}}}{M_{\text{un-stab}}} = \frac{\sum_{i=1}^{N} [c_{\text{si}} l_{\text{si}} + (N_{\text{si}} - u_{\text{i}}) \tan \phi_{\text{i}}]}{\sum_{i=1}^{N} W_{\text{si}} \times \sin \alpha_{\text{i}}} \tag{4.1.6}$$

On the other hand, the main hypothesis of the Bishop simplified method is to considers equal to zero all the vertical forces to the sliding surface acting on the lateral sides, so considering the figure above, $E_{1\text{s}} = E_{2\text{s}} = 0$.
The equilibrium of a slice in the vertical direction is equal to:

$$W_{\text{si}} - u_{\text{i}} \Delta x_{\text{i}} - N_{\text{si}} \times \cos \alpha_{\text{i}} - \frac{1}{SF} (c_{\text{i}}^{'} \Delta x_{\text{i}} + N_{\text{si}} \tan \phi_{\text{i}} \times \cos \alpha_{\text{i}}) \times \tan \alpha_{\text{i}} \tag{4.1.7}$$

where through simplifications results that:

$$N_{\text{si}} = \frac{W_{\text{si}} - u_{\text{i}} \Delta x_{\text{i}} - \frac{1}{SF} c_{\text{i}}^{'} \Delta x_{\text{i}} \times \tan \alpha_{\text{i}}}{\cos \alpha_{\text{i}} [1 + \frac{\tan \phi' \times \tan \alpha_{\text{i}}}{SF}]} \tag{4.1.8}$$

which has to be replaced in the following equation:

$$SF = \frac{M_{\mathrm{stab}}}{M_{\mathrm{un-stab}}} = \frac{\sum_{i=1}^{N} [c_{\mathrm{si}} l_{\mathrm{si}} + (N_{\mathrm{si}} - U_{\mathrm{si}}) \tan \phi_{\mathrm{i}}]}{\sum_{i=1}^{N} W_{\mathrm{si}} \times \sin \alpha_{\mathrm{i}}} \tag{4.1.9}$$

So, in the Bishop simplified method we obtain the safety factor depending on itself, and consequently an iterative process is necessary.
Usually, the most used method is the Bishop semplified, even if the most conservative is the Fellenius method, given that it provides the lowest safety factors.

4.1.1 GS stability.

The software used in this work to calculate the slope stability, which is based on the previous theory is GeoSuite Stability, a Novapoint software. It allows to start from a CAD section with a definite excavation or embankments, and then let to choose the stratigraphy of the soil. The value of the shear strength is the most important parameter, in fact the software permits also to create a shear strength profile with depth. The values of shear strength which have been used to calculate the slope stability in each area have been previously reported in the tables 2.2, 2.4, 2.6 and 2.8 and correspond to the shear strengths referred to the depth interval. Other important parameters to choose in the software are the level of the ground water table and the applied loads, which can be a point load, a distributed load or a shear load. The points where the load is acting can be decided, and also if the calculation is on a right or left slope. Finally it is possible to choose between some calculation methods (Bishop, Bishop semplified, Force Equilibrium, Beast) and also between some calculation strategies. The calculation strategies differ substantially in the shape of the searching area of slip, as showed in Fig. 4.2.
In the end the software calculates the most probable slip surface, with the corrisponding safety factor. It is also possible to force the slip surface to pass through a definite point, which is very useful in the cases where the slip surface given by the program is not realistic or insignificant.

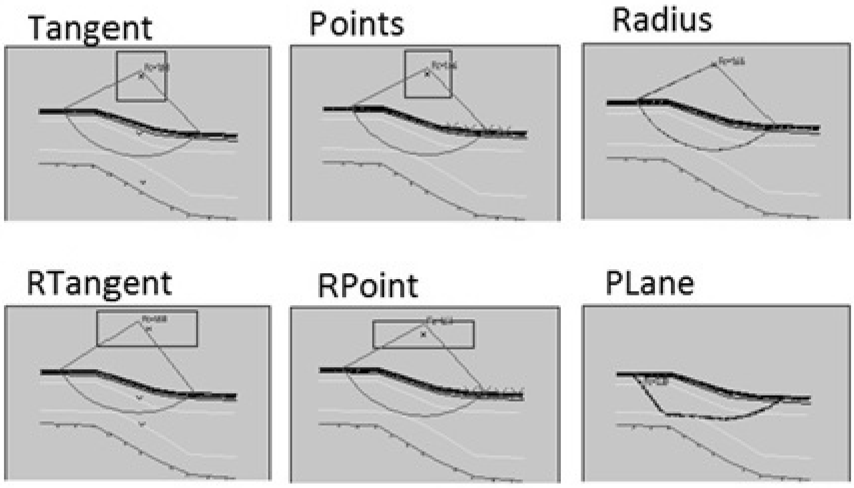


Figure 4.2: representation of the different calculation strategies.

4.1.2 Resulting slope stability

The slope stability analysis have been conducted in 4 points, one for each colour area. More than this, the analysis have been conducted in the most critical points of each colour area, and in particular in the points with the highest thickness of clay:

- red area: point 16B34;
- blue area: point 16B12;
- green area: point 16B19;
- yellow area: point 16B46;

The main parameter of the slope stability calculations is the shear strength of the clay, whose values are reported in the tables 2.2, 2.4, 2.6 and 2.8. The calculations have been performed in the case of a 3 and 4.5 m of excavation with 3 different slope, as showed in Figure 4.3.

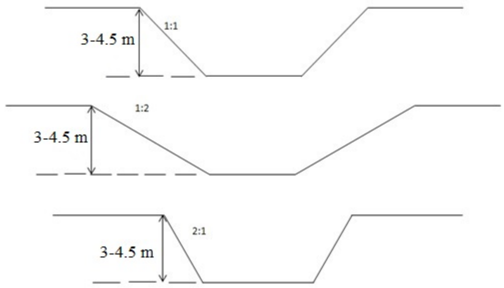


Figure 4.3: representation of the different slope configurations.

The slip surfaces have been calculated with the software Geosuite Stability, with a distributed load and with the Rtangent calculation strategy, which requires a centre point, a rectangular searching area, and the upper and lower tangential levels in which searching the slip surface. Obviously, being the clay the weak layer the slip surfaces have been searched in this layer.
The desired safety factor for reaching the stability is 1.3, as established by the swedish regulations. In fact, notwithstanding the geotechnical eurocode establishes a SF of 1.0, the current swedish regulation requires 1.3, so this will be the desired safety factor in this work.
The slip surfaces have been calculated with the software GeoSuite Stability, which permits also to visualize the contour lines of the different safety factors, as showed in Figure 4.4, to better determine the slip surface that gives the lowest safety factor.

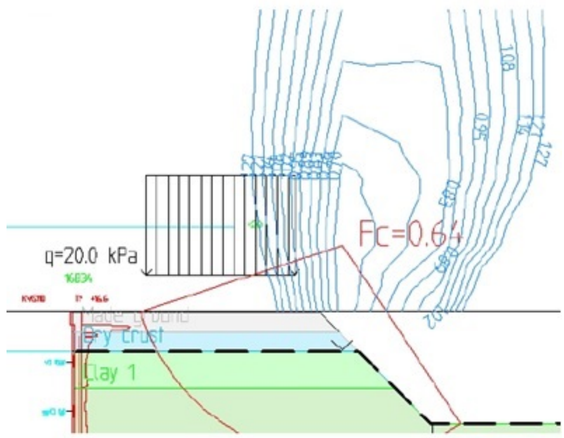


Figure 4.4: example of the contour lines of the safety factor.

RED AREA
3 m excavation

SLOPE 1:1

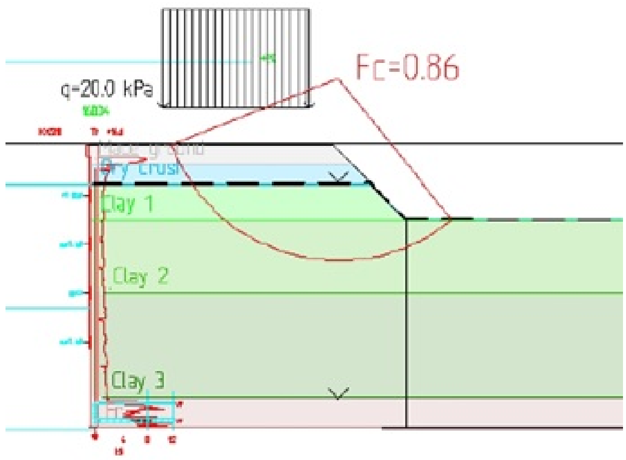


Figure 4.5: representation of the slip surface and the safety factor in 16B34 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

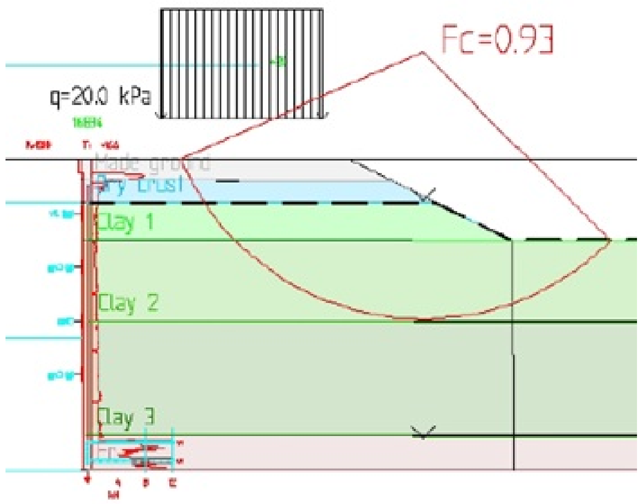


Figure 4.6: representation of the slip surface and the safety factor in 16B34 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

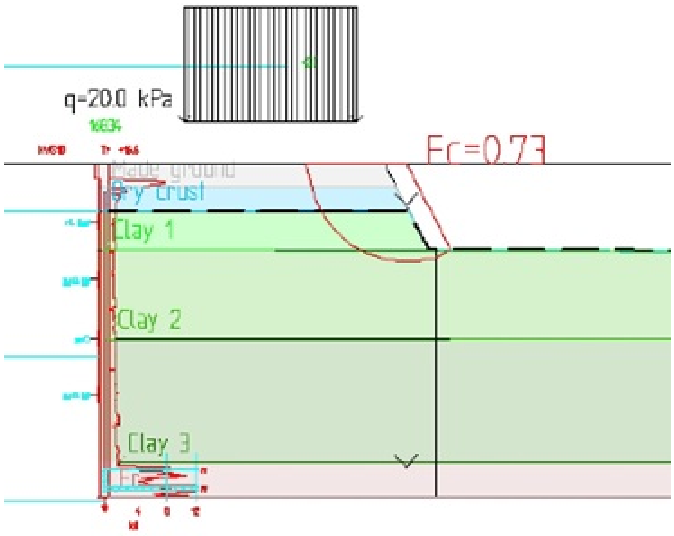


Figure 4.7: representation of the slip surface and the safety factor in 16B34 with slope 2:1 and 3 m of excavation.

The results are then summarized in Table 4.1.

RED AREA - 3 m	
Slope	**SF**
2:1	0.73
1:1	0.86
1:2	0.93

Table 4.1: Resulting safety factor for the different slope for the point 16B34 and for an excavation of 3 m.

In every case the safety factor is smaller than the minimum required. Obviously, the minimum SF is obtained by the slope 2:1 which is the steepest, while the highest SF is obtained by the slope 1:2 which is the less steep. Another important thing to notice it that the resulting slip surface never extends in the third layer of clay; this is due to the higher value of the shear strength in this layer, as showed previously in Table 2.2.

RED AREA

4.5 m excavation

SLOPE 1:1

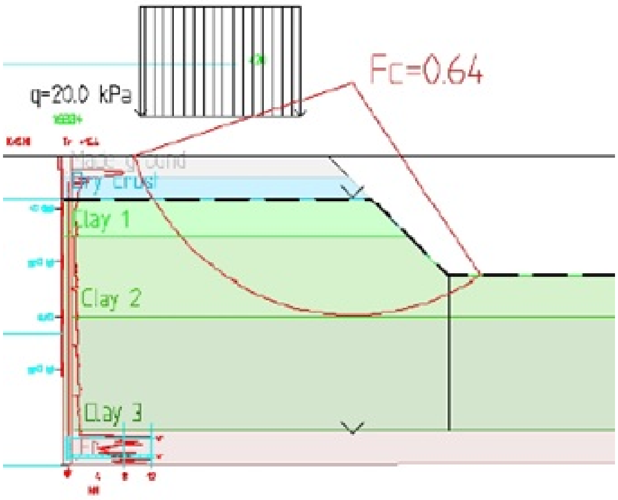


Figure 4.8: representation of the slip surface and the safety factor in 16B34 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

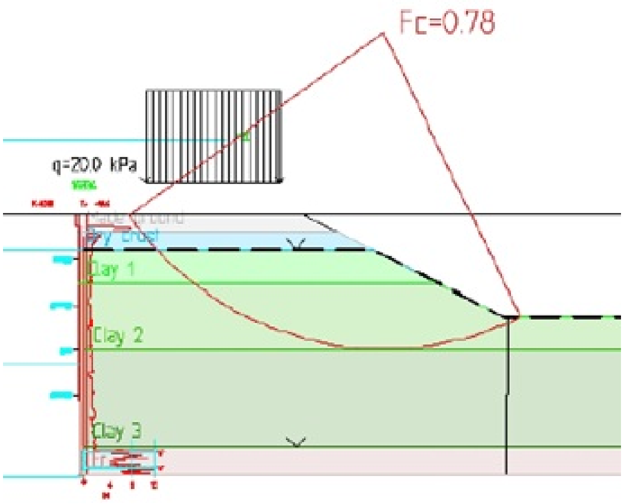


Figure 4.9: representation of the slip surface and the safety factor in 16B34 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

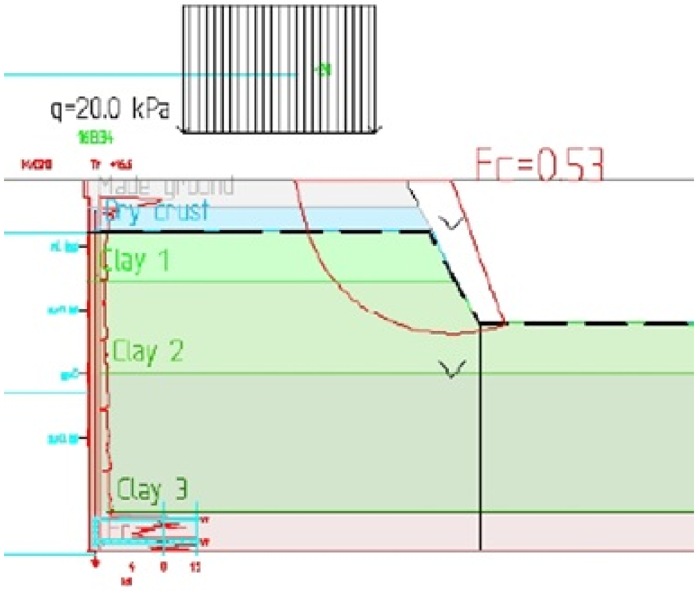


Figure 4.10: representation of the slip surface and the safety factor in 16B34 with slope 2:1 and 4.5 m of excavation.

The results are then summarized in Table 4.2.

RED AREA - 4.5 m	
Slope	**SF**
2:1	0.53
1:1	0.64
1:2	0.78

Table 4.2: Resulting safety factor for the different slope for the point 16B34 and for an excavation of 4.5 m.

In every case the safety factor is smaller than the minimum required.

BLUE AREA
3 m excavation

SLOPE 1:1

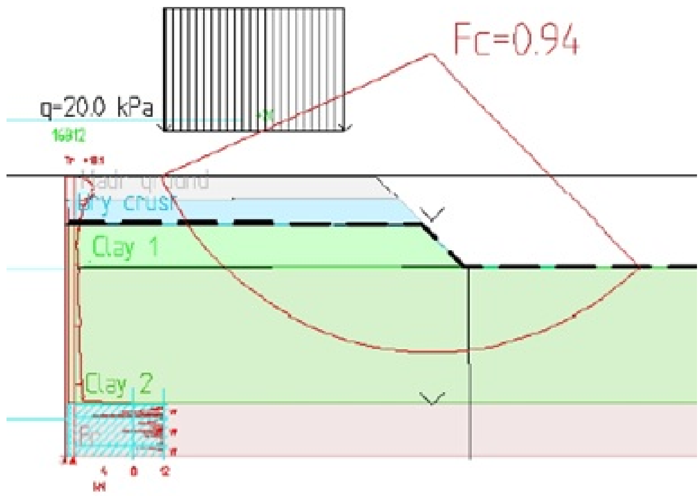


Figure 4.11: representation of the real slip surface and the safety factor in 16B12 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

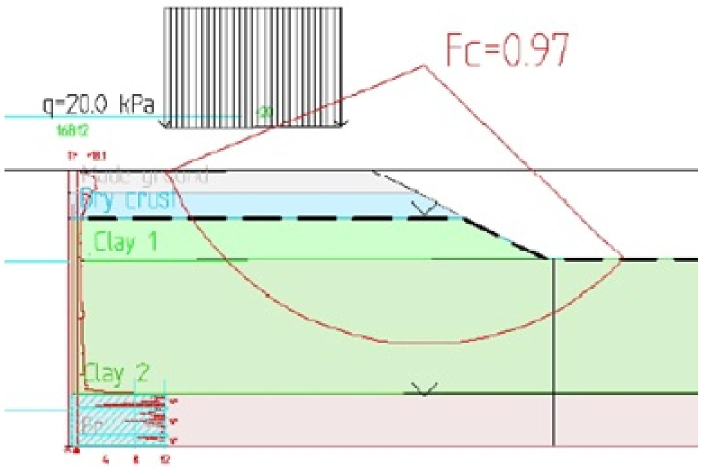


Figure 4.12: representation of the slip surface and the safety factor in 16B12 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

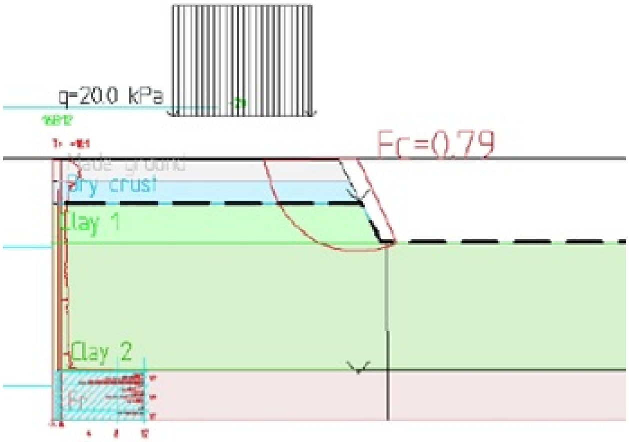


Figure 4.13: representation of the slip surface and the safety factor in 16B12 with slope 2:1 and 3 m of excavation.

The results are then summarized in Table 4.3.

BLUE AREA - 3 m	
Slope	**SF**
2:1	0.79
1:1	0.94
1:2	0.97

Table 4.3: Resulting safety factor for the different slope for the point 16B12 and for an excavation of 3 m.

Even in this point and in every case the safety factor is smaller than the minimum required.

BLUE AREA
4.5 m excavation

SLOPE 1:1

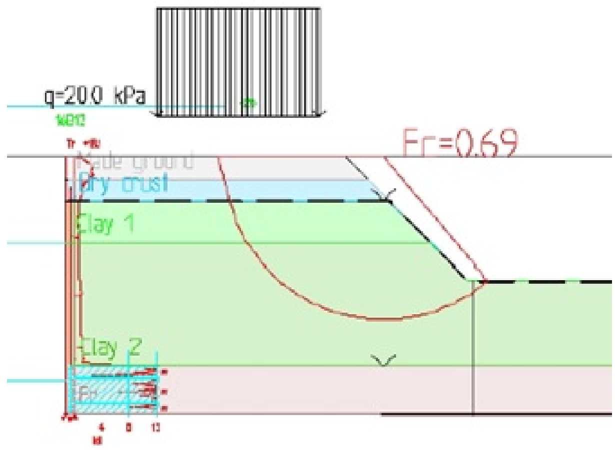


Figure 4.14: representation of the real slip surface and the safety factor in 16B12 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

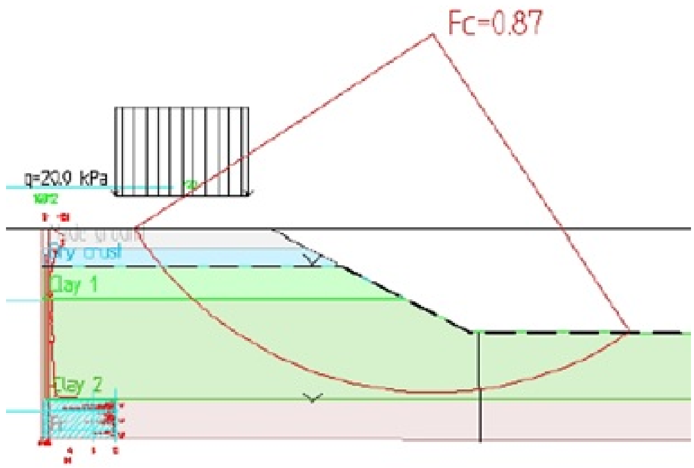


Figure 4.15: representation of the slip surface and the safety factor in 16B12 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

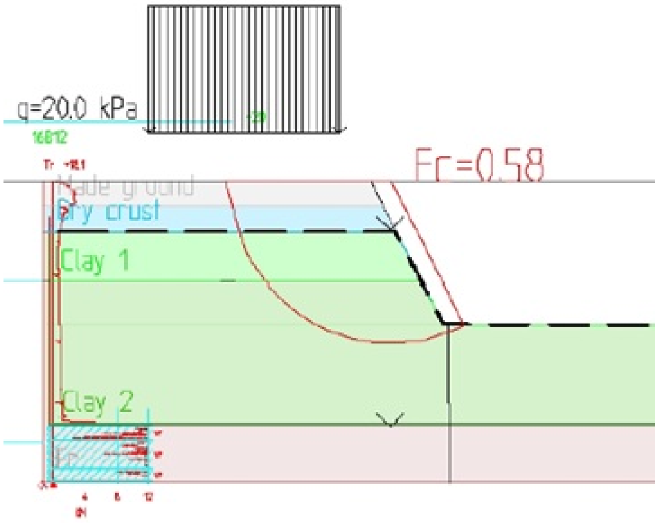


Figure 4.16: representation of the slip surface and the safety factor in 16B12 with slope 2:1 and 4.5 m of excavation.

The results are then summarized in Table 4.4.

BLUE AREA - 4.5 m	
Slope	**SF**
2:1	0.58
1:1	0.69
1:2	0.87

Table 4.4: Resulting safety factor for the different slope for the point 16B12 and for an excavation of 4.5 m.

Even in this point and in every case the safety factor is smaller than the minimum required.

GREEN AREA

3 m excavation

SLOPE 1:1

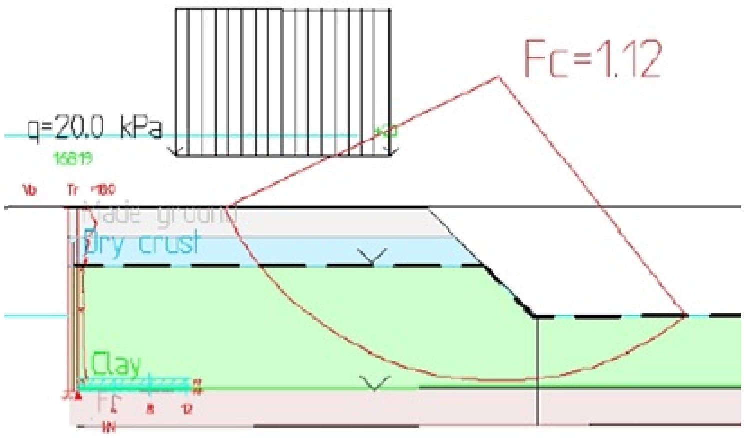


Figure 4.17: representation of the slip surface and the safety factor in 16B19 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

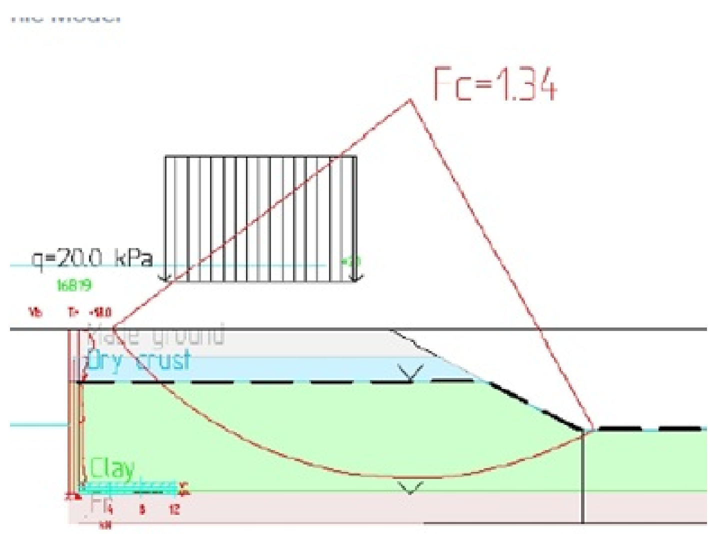


Figure 4.18: representation of the slip surface and the safety factor in 16B19 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

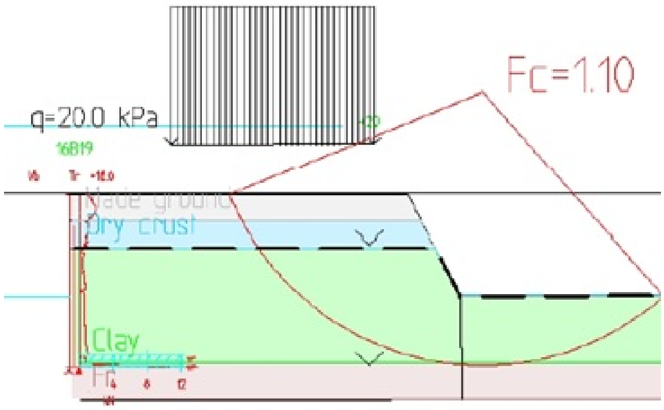


Figure 4.19: representation of the slip surface and the safety factor in 16B19 with slope 2:1 and 3 m of excavation.

The results are then summarized in Table 4.5.

GREEN AREA - 3 m	
Slope	**SF**
2:1	1.10
1:1	1.12
1:2	1.34

Table 4.5: Resulting safety factor for the different slope for the point 16B19 and for an excavation of 3 m.

It is important to notice that the safety factor of 1.3 is reached in the green area and for the configuration of a 1:2 slope. In other words, the minimum SF it is obtained in the less critical configuration and in the less critical area, as expected.

GREEN AREA
4.5 m excavation

SLOPE 1:1

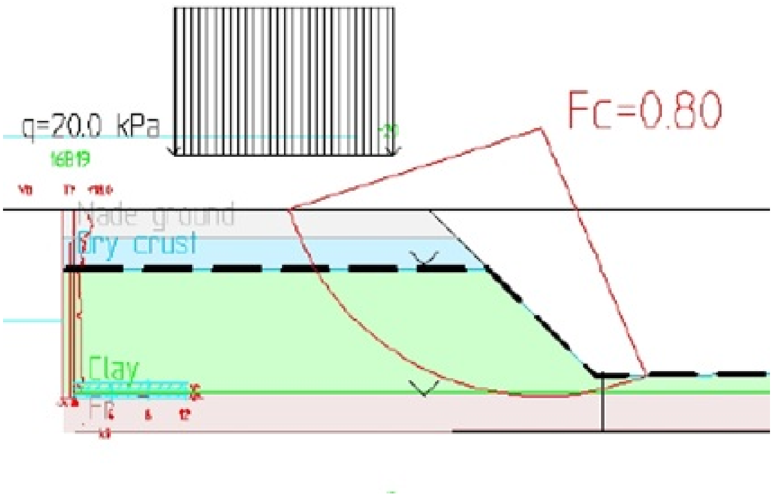


Figure 4.20: representation of the slip surface and the safety factor in 16B19 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

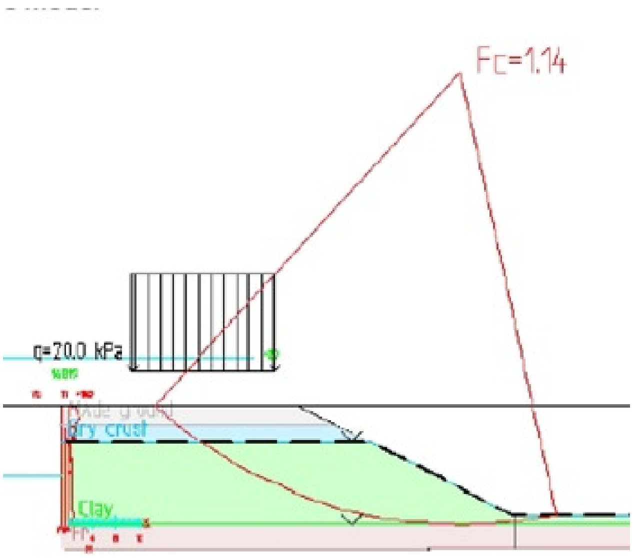


Figure 4.21: representation of the slip surface and the safety factor in 16B19 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

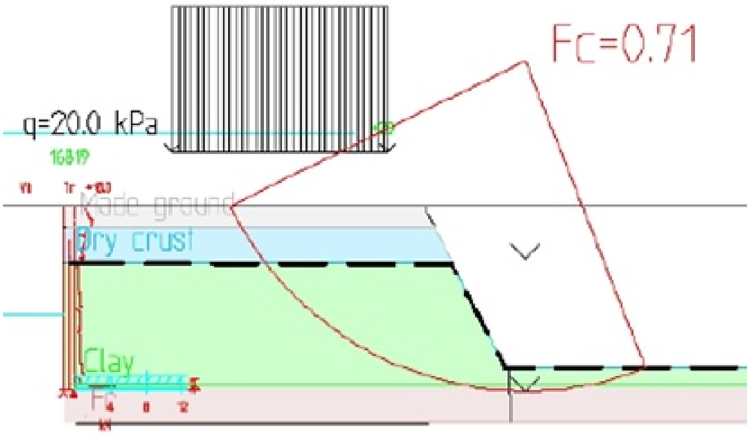


Figure 4.22: representation of the slip surface and the safety factor in 16B19 with slope 2:1 and 4.5 m of excavation.

The results are then summarized in Table 4.6.

GREEN AREA - 4.5 m	
Slope	**SF**
2:1	0.71
1:1	0.80
1:2	1.14

Table 4.6: Resulting safety factor for the different slope for the point 16B19 and for an excavation of 4.5 m.

Even in this point and in every case the safety factor is smaller than the minimum required.

YELLOW AREA
3 m excavation

SLOPE 1:1

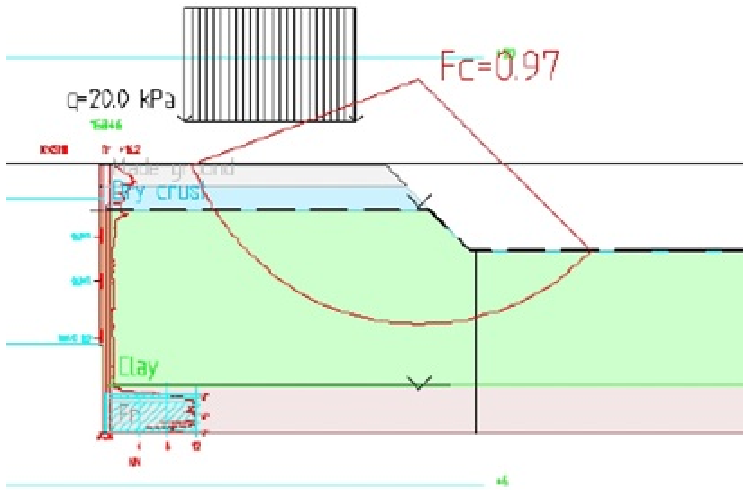


Figure 4.23: representation of the real slip surface and the safety factor in 16B46 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

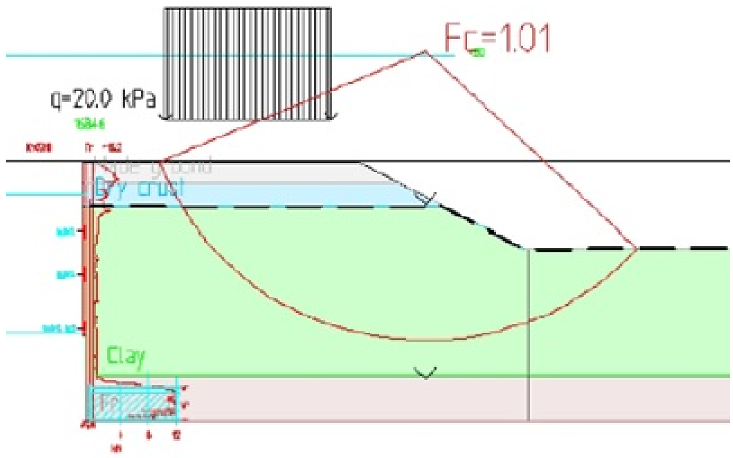


Figure 4.24: representation of the slip surface and the safety factor in 16B46 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

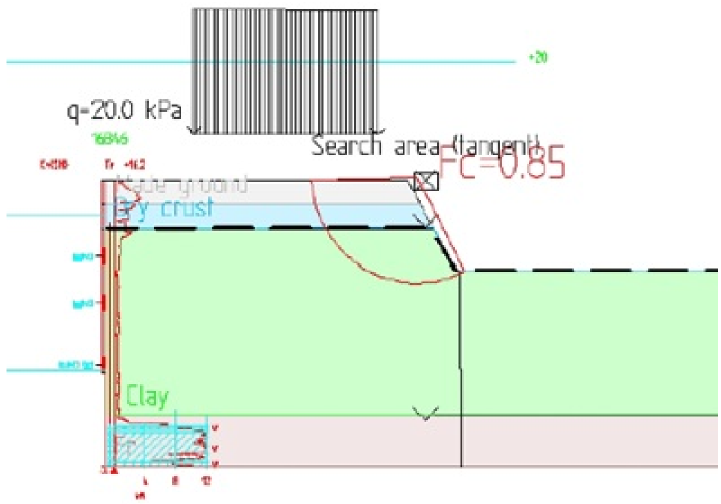


Figure 4.25: representation of the slip surface and the safety factor in 16B46 with slope 2:1 and 3 m of excavation.

The results are then summarized in Table 4.7.

YELLOW AREA - 3 m	
Slope	**SF**
2:1	0.85
1:1	0.97
1:2	1.01

Table 4.7: Resulting safety factor for the different slope for the point 16B46 and for an excavation of 3 m.

Even in this point and in every case the safety factor is smaller than the minimum required.

YELLOW AREA

4.5 m excavation

SLOPE 1:1

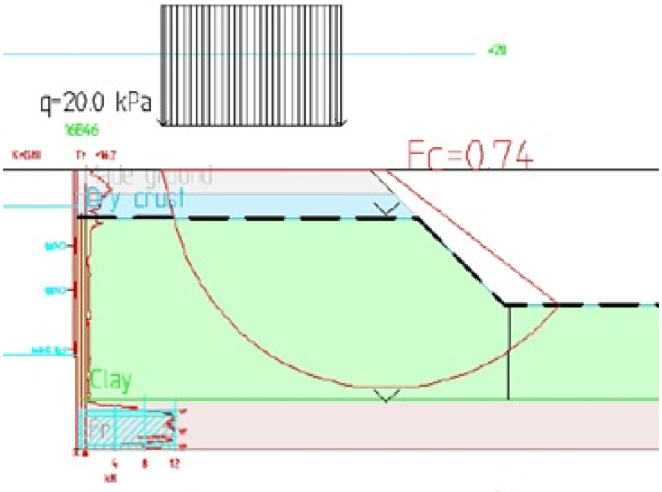


Figure 4.26: representation of the real slip surface and the safety factor in 16B46 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

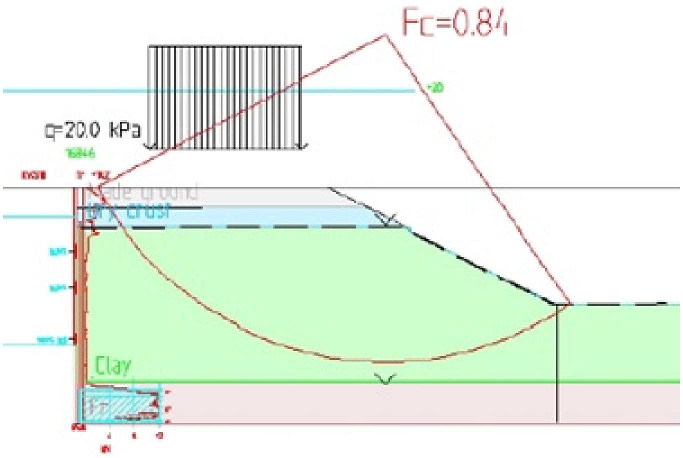


Figure 4.27: representation of the slip surface and the safety factor in 16B46 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

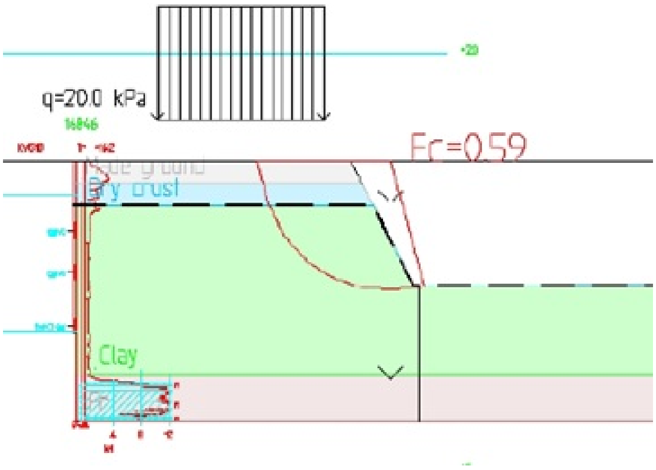


Figure 4.28: representation of the slip surface and the safety factor in 16B46 with slope 2:1 and 4.5 m of excavation.

The results are then summarized in Table 4.8.

YELLOW AREA - 4.5 m	
Slope	**SF**
2:1	0.59
1:1	0.74
1:2	0.84

Table 4.8: Resulting safety factor for the different slope for the point 16B46 and for an excavation of 4.5 m.

Even in this point and in every case the safety factor is smaller than the minimum required.

In conclusion, the safety factor is reached only in the green area, only for a 3 m excavation and only in the less critical slope configuration, which is also not very significant, because usually the excavations have a slope of at least 1:1.
For this reason a stabilization intervention is quite necessary. The results of all the areas are reported in Table 4.9.

RED AREA		
Slope	**SF (3 m)**	**SF (4.5 m)**
2:1	0.73	0.53
1:1	0.86	0.64
1:2	0.93	0.78
YELLOW AREA		
Slope	**SF (3 m)**	**SF (4.5 m)**
2:1	0.85	0.59
1:1	0.97	0.74
1:2	1.01	0.84
BLUE AREA		
Slope	**SF (3 m)**	**SF (4.5 m)**
2:1	0.79	0.58
1:1	0.94	0.69
1:2	0.97	0.87
GREEN AREA		
Slope	**SF (3 m)**	**SF (4.5 m)**
2:1	1.10	0.71
1:1	1.12	0.80
1:2	1.34	1.14

Table 4.9: Results of all the areas.

It is possible to notice that for the red area, which is the most critical, the safety factors are the lowest, and in fact in the point of the calculation (16B34) the thickness of the clay is 9.5 m and the values of the shear strength are quite low (Table 2.2). On the other hand, for the green area which is the less critical (less thickness of clay, high shear strength (15 kPa, see Table 2.6) the safety factors are the highest.
The safety factors of the blue and yellow areas are very similar, but in general the values of the yellow area are slightly higher; this reflects the values of the shear strength, which are a bit higher in the yellow area (13.5 kPa, see Table 2.8) than in the blue area ($10.3 - 13$ kPa, see Table 2.4).

Capitolo 5

STABILIZATION WITH THE LCC

In this chapter will be first explained the reason of the use of the Lime-Cement columns instead of other methods.
After that some calculations of the settlements after the stabilization with the columns will be performed and they will highlight the improvement of the settlements given by this method of stabilization. Then it will be calculated the necessary shear strength that permit the reaching of a safety factor of 1.3 and the corresponding slip surfaces will be shown.
Finally, both the calculations of the settlements and the slope stability will led to a columns planning, especially defining the distance between the colunmns center.

5.1 Why the Lime-Cement columns?

First of all, in Dalvägen area the problem is double because not only important settlements are present, but also the safety factor is not sufficient to ensure a slope stability. For this reasion many other methods that would act only to improve the settlements are excluded, as the installation of vertical drains.
Another method would be the installation of light materials, which performing the excavation of the superficial granular material and the following filling with light materials characterized by a very low value of density, led to a final load of 0 kPa. Notwithstanding this, the method is excluded

because a) it doesn't have any effect on the slope stability and b) it isn't totally safe because a stagional oscillation of the ground water table would implicate a load different than 0 kPa.

From an economical point of view the Lime-Cement columns are one of the cheapest method, given that the price of 1 m of a standard columns with a diameter of 0.6 m swings between 80 and 100 krona, which corresponds to $8-10$ euros. Obviously, beyond this risible price it has to be considered also the transport and the installation of the Lime-Cement column machines and the previous planning of the soil stabilization. Given that the Lime-Cement columns machines are very huge so their transportation is not very simple, the price of the transport is around 100000 krona, which corresponds to around 10000 euros. Even the previous planning of the stabilization requiring a lot of work has a high price of 10000 euros.
Considering these two prices, the columns don't seem anymore cheap, but it has also to be considered the huge size of Dalvägen project. In fact, another method could be the excavation of the clay but a) the price of 1 m^3 of excavation is around 500 krona (50 euros), 2) it is quite impossible to dig $8-10$ m of clay and 3) only Dalvägen street is around 1200 m long, so the excavations are not convenient.
In fact, if considering that the columns stabilizes only themselves we have that 1 m of columns that corresponds to 0.283 m^3 costs 10 euros, while as already mentioned an excavation of 1 m^3 costs 50 euros. The total cost is then represented in Figure 5.1, where it is possible to notice that the excavation is economically convenient only if smaller than 1400 m^3.
On the other hand it is more correct to consider that the columns doesn't stabilize only themselves, but all an area around them. In the case of single columns with a distance between the centres of 1.2 m they stabilize a square area of 1.44 m^2, so 10 euros is the cost of 1.44 m^3. In this case the total cost is represented in Figure 5.2, where it is possible to notice that the excavation is economically convenient only if smaller than 500 m^3.

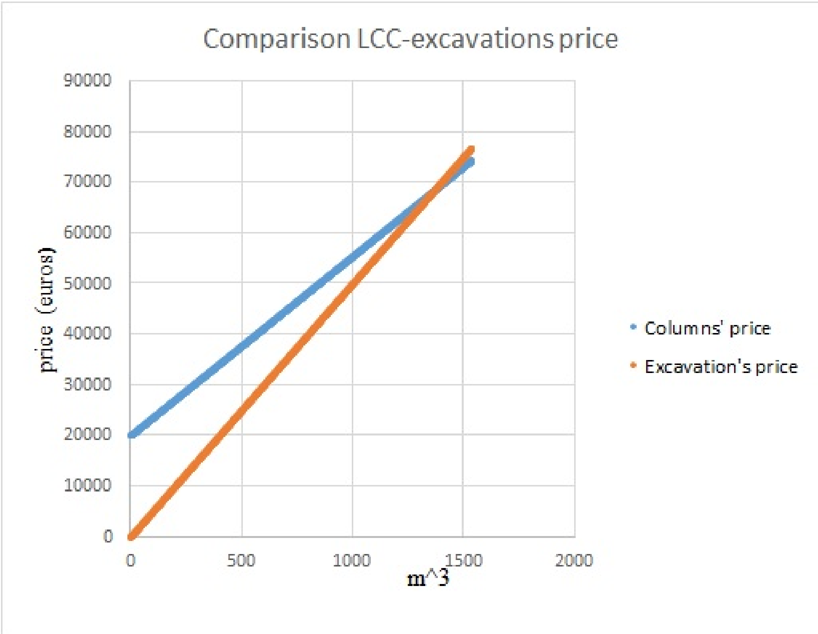


Figure 5.1: representation of the price of the Lime-Cement columns compared to the excavations if the columns stabilize only themselves.

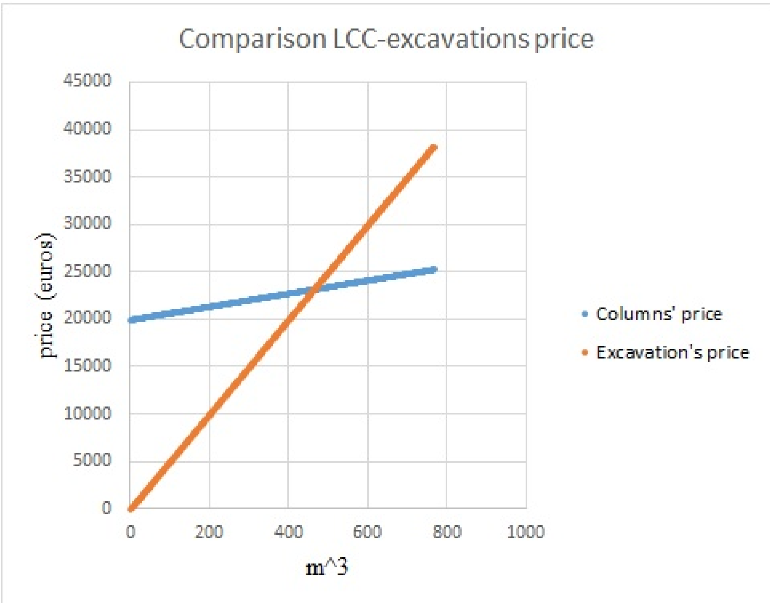


Figure 5.2: representation of the price of the Lime-Cement columns compared to the excavations if the columns stabilize an area around them.

Anyway, in all the two cases, the quantity of excavation that results economically convenient is quite small, in fact both 1400 and 500 m^3 are not comparable with Dalvägen area. The only way where the excavations would be used instead of the columns is in the few points where the clay thickness is low (around $1 - 3$ m), so only in the places where the columns can't be installed.

Another possible method would be the installation of the sheet pile walls, but 1) it is suitable only for the slope stability and not for the settlements and 2) it has a very high cost: 5000 kr/m^2 which corresponds to around 500 $euros/m^2$.If considering that the sheet pile walls should be installed with a depth under the excavation of at least 2/3 of the height of the excavation, we obtain that the total lenght of a sheet pile wall is 5 m for an excavation of 3 m and a total lenght of 7.5 m for an excavation of 4.5 m. Considering a lenght of the excavation of 50 m, it results that the total area is respectively 250 and 375 m^2, so the total cost for the installation of the sheet pile walls is respectively 125000 and 187500 euros.
To have a comparison with the Lime-Cement columns, we know that in the case of excavations the pattern has to be a slab. Considering a slab distance of 2.5 m and the lenght of the excavation of 50 m, it results that 20 slabs have to be arranged. Considering a perpendicular lenght to the excavation of 6 m and that the columns have to overlap for 10 cm, we obtain that 12 columns are necessary in the perpendicular direction to the excavation. So, the total number of columns that have to be installed along the 50 m is 240. As seen before the price for the columns is 10 euros per 0.28 m^3, so which will be the length of the columns necessary to obtain the same price of the sheet pile walls? The solution is represented in Figure 5.3, where it is assumed that the columns stabilize only themselves and it is considered also the price of the transport and the planning of the columns (20000 euros in total). It is possible to notice that the lenght of the columns necessary to equal the price of the sheet pile walls in both the two cases are very high, around 22 m and 37 m, which are surely not comparable with the columns that will be installed along Dalvägen, which are at most around 10 m.

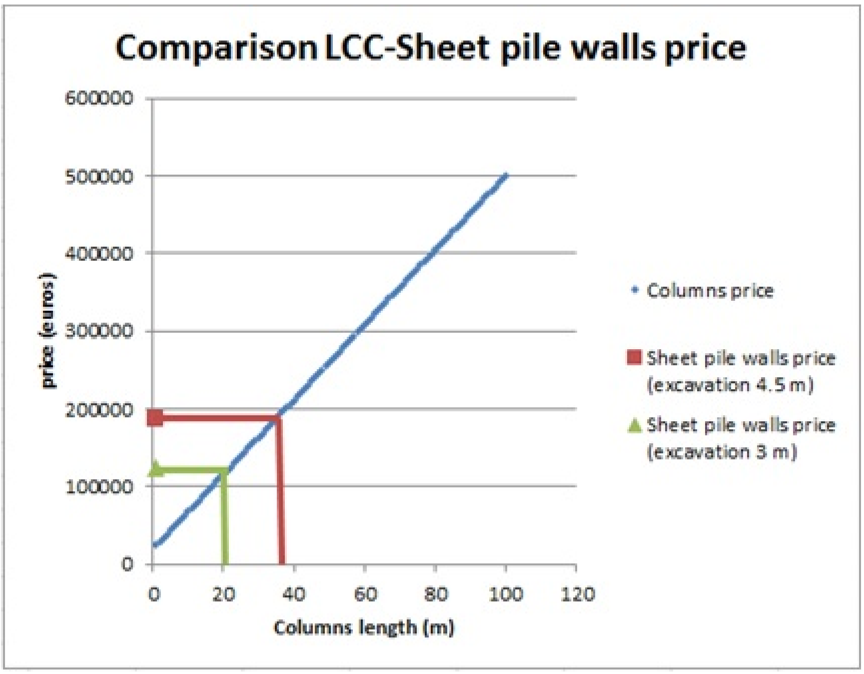


Figure 5.3: representation of the price of the Lime-Cement columns compared to the sheet pile walls.

5.2 Settlements calculation after the LCC installation.

The method of calculation of the settlements after the installation of the Lime-Cement columns is specified by some swedish regulations which are presented in the Report 4:95 of the Swedish Geotechnical Society.

The method is based on the hypothesis that the columns and the un-stabilized clay support the same compression. This implies that the load on the un-stabilized clay is transferred to the columns (q_2), while the load on the columns is transferred at their base (q_1), as showed in Figure 5.4.

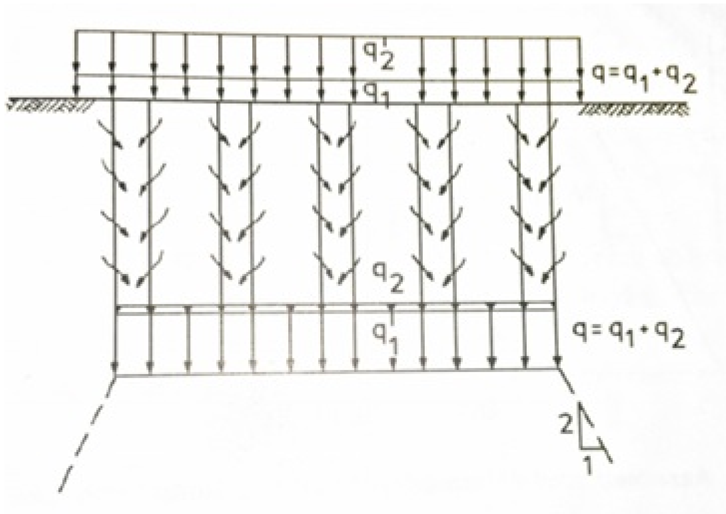


Figure 5.4: representation of the load distribution in the column reinforcement (SGF report 4:95, 2000).

To calculate the different parts q_1 and q_2 of the total load, it is necessary to consider the load-deformation curve in Figure 5.5.

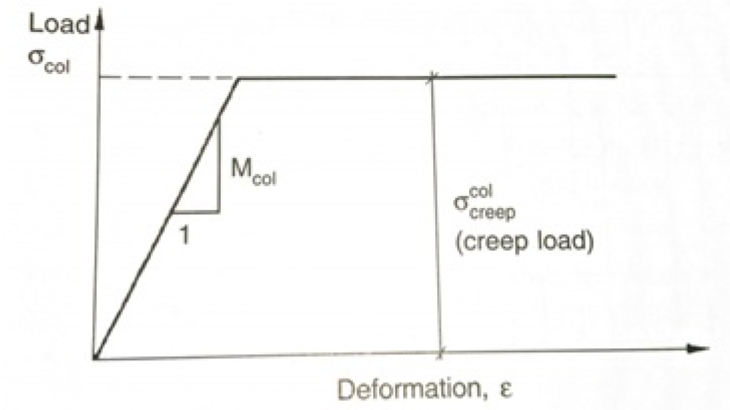


Figure 5.5: load-deformation curve (SGF report 4:95, 2000).

The curve is linear until the creep strength is reached and the slope of the curve represents the compression modulus of the columns M_{col}. After the creep strength has been exceeded, the load on the columns is constant. The creep strength of clay and Lime-Cement columns can be put between 65 and 80% of the ultimate strength σ_{ult}, which is function of the characteristic undrained shear strength and of the horizontal pressure σ_{h} on the columns. Considering a creep strength of 65% (most conservative case), the calculation starts with a maximum load on the single columns that is:

$$q_{1\max} = 0.65 \times a \times \sigma_{\mathrm{ult}} \tag{5.2.1}$$

where a is calculated as seen before in equation 1.2.3.
σ_{ult} is function of the shear strength τ of the columns and of the horizontal pressure σ_{h}, accordin to the empirical equation:

$$\sigma_{\mathrm{ult}} = 2 \times \tau + 3 \times \sigma_{\mathrm{h}} \tag{5.2.2}$$

Where σ_{h} can be assumed equal to σ_{v} or as $\frac{q}{2}$. Consequently,

$$q_2 = q - q_1 \tag{5.2.3}$$

since the total load is made up of the two parts, the load on the columns and the load on the un-stabilized clay. Having the different loads for the columns and for the un-stabilized clay, it is possible to calculate the settlements for these two different parts:

- the settlements for the columns:

$$S_1 = \sum \frac{\Delta h}{a} \times \frac{q_1}{M_{\mathrm{col}}} \tag{5.2.4}$$

  where h is the single stratum thickness (the model considers the clay profile divided into some strata);

- the settlements for the un-stabilized clay:

  1. if $\sigma_0^{'} + \frac{q_2}{1-a} \leq \sigma_{\mathrm{C}}^{'}$:

$$S_2 = \sum \frac{\Delta h}{1-a} \times \frac{q_2}{M_{\mathrm{clay}}} \tag{5.2.5}$$

  2. if $\sigma_{\mathrm{C}}^{'} \leq \sigma_0^{'} + \frac{q_2}{1-a} \leq \sigma_{\mathrm{L}}^{'}$:

$$S_2 = \sum \Delta h \times \left[\frac{\sigma_{\mathrm{C}}^{'} - \sigma_0^{'}}{M_0} + \left(\frac{\sigma_0^{'} + \frac{q_2}{1-a} - \sigma_{\mathrm{C}}^{'}}{M_{\mathrm{L}}} \right) \right] \tag{5.2.6}$$

3. if $\sigma_0^{'} + \frac{q_2}{1-a} \geq \sigma_L^{'}$:

$$\begin{aligned} S_2 &= \sum \Delta h \times \left[\frac{\sigma_C^{'} - \sigma_0^{'}}{M_0} + \frac{\sigma_L^{'} - \sigma_C^{'}}{M_L} \right. \\ &+ \left. \frac{1}{M'} \times \ln \left(\frac{M' \times (\sigma_0^{'} + \frac{q_2}{1-a} - \sigma_L^{'})}{M_L} + 1 \right) \right] \end{aligned} \tag{5.2.7}$$

The total settlements will be different in two cases:

- If $S_1 > S_2$, a load transfer begins reducing q_1 and increasing q_2. An iterative process is necessary, because for each new load q_1 and q_2 new settlements result, and the equation for S_2 can vary (Equation 5.2.5 or 5.2.6 or 5.2.7). This iterative process continues until $S_1 = S_2$.

- If $S_1 < S_2$, the columns can't sustain any more load, so the settlements will be equal to S_2.

It is important to notice that the total settlements are function of several parameters, as:

- the ratio between M_{col} and M_{clay};

- the extension of the stabilized soil with the columns;

- the consolidation properties of the clay;

- the creep load of the columns;

- the time of load application;

- the permeability of the columns and of the un-stabilized soil; usually the permeability of the columns is much higher (1000 times) than the one of the un-stabilized clay, so the columns may be considered as drains.

One of the most important things to consider when stabilizing a soil is the time trend of the settlements. This is obviously dependent on the degree of consolidation because until the consolidation has not completely occurred, the settlements would be present. The consolidation process can be accelerated with some techniques as the use of pre-charge, but when using the Lime-Cement columns, these can be considered as vertical drains accelerating the process. The degree of consolidation is calculated with the following equation:

$$U = 1 - \exp\left[\frac{-2c_{\mathrm{h}} \times t}{R^2 \times f(n)}\right] \tag{5.2.8}$$

where c_{h} is the coefficient of consolidation in the un-stabilized clay, normally considered as $2c_{\mathrm{v}}$, t is the period of consolidation and R is the influence radius of columns.
In the end, the parameter $f(n)$ is dependent on the geometrical configuration of the columns, which act as radial drains so they have an influencing equivalent diameter, and in fact the consolidation degree in Eq. 5.2.8 is a radial degree. $f(n)$ is calculated as:

$$\begin{aligned} f(n) &= \frac{n^2}{n^2-1} \times \left[\ln(n) - 0.75 + \frac{1}{n^2} \times \left(1 - \frac{1}{4n^2}\right)\right] \\ &+ \left[\frac{n^2-1}{n^2} \times \frac{1}{r^2} \times \frac{k_{\mathrm{clay}}}{k_{\mathrm{col}}} \times L_{\mathrm{D}}^2\right] \end{aligned} \tag{5.2.9}$$

where $n = \frac{R}{r}$ with r column radius and L_{D} the drainage length.

5.2.1 Limeset.

The two ways used in this work to calculate the settlements of the stabilized soil, which use the previous equations are:

- Limeset;
- An excel sheet.

Limeset is a software developed by the Swedish Geotechnical Institute in 1989 which is based on the theory seen before. It starts from the parameters of each layer as its thickness, its effective density, the pre-consolidation pressure, the limit pressure, the compression modulus, the modulus number, the modulus before the pre-consolidation pressure, the compression modulus of the columns and the shear strength of the columns. Other important needed parameters are the ground water level, the column diameter and length, the consolidation coefficient, the value of the creep strength, the ratio between the permeability of the columns and the one of the clay and the type of the drainage (single or double). In the end it asks for the

applied load and the centre distances between the columns. The output of the program is a txt file in which the settlements for the several loads and centre distances are reported, and also the time needed for a specific degree of consolidation.
During my internship period, I developed an excel workbook which calculate the settlements after the reinforcement using the same theoretical equations of Limeset. The blue cells are the cells that have to be filled in, while the white cells are parameters obtained from the calculations that have not to be modified and the green cells are the final settlements. It is possible to choose three different center distances and three loads for each one of these. It calculates also the rate of the settlements with time. At the end two graphs are showed: one with the settlements as a function of the loads and one with the settlements as a function of the time.

5.2.2 Resulting settlements after the LCC installation.

All the settlements after the reinforcement have been calculated with both Limeset and the excel workbook which are attached at the end of this work, while in this chapter the main results are presented.
The settlements have been calculated for three different loads (20, 30 and 40 kPa) and for three different centre distance between the columns (0.8, 1 and 1.2 m).

RED AREA

14W44						
	Excel			**Limeset**		
Load (kPa)	c_1**(m)**	c_2**(m)**	c_3**(m)**	c_1**(m)**	c_2**(m)**	c_3**(m)**
20	0.039	0.050	0.063	0.040	0.052	0.064
30	0.059	0.082	0.110	0.062	0.086	0.112
40	0.083	0.118	0.157	0.087	0.122	0.161

Table 5.1: Settlements calculation after the stabilization with the Lime-Cement columns in the point 14W44. c_1 corresponds to 0.8 m, c_2 to 1 m and c_3 to 1.2 m.

The rate of the settlements with time is represented in Figure 5.6.

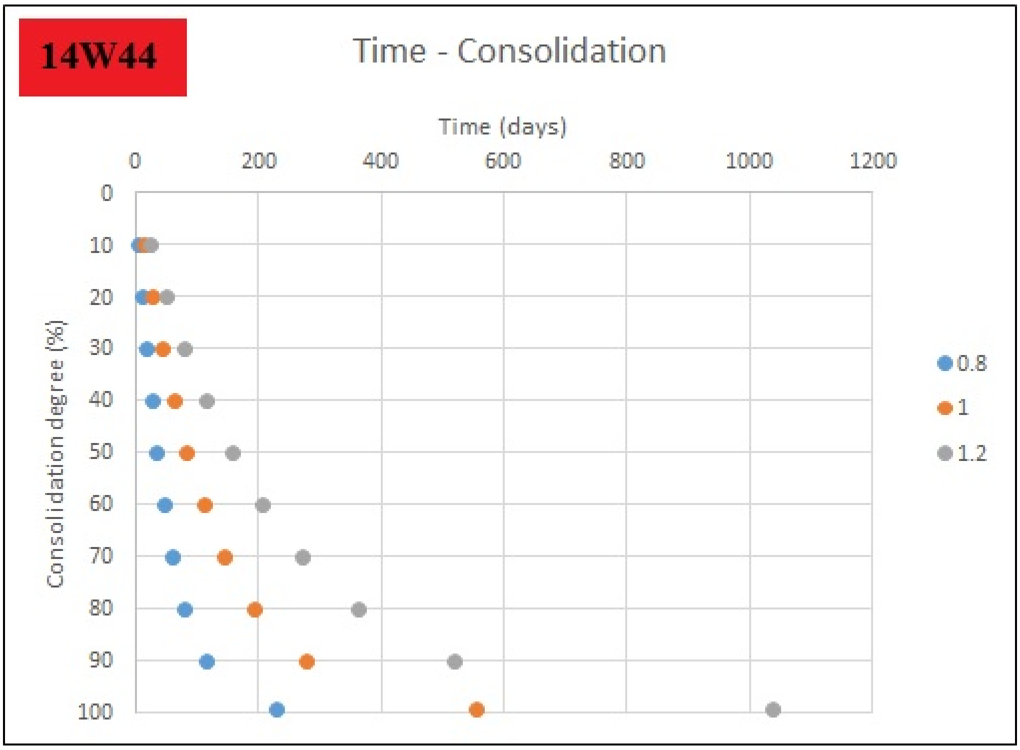


Figure 5.6: representation of the rate of the settlements with time in the point 14W44 after the LCC for three different centre distance c_1, in particular 0.8 m, 1 m and 1.2 m.

We can notice that the 90% of the consolidation is already reached after 500-550 days with a centre distance between the columns of 1.2 m which is the worst configuration. Compared to the required time before the soil stabilization (48 years) it is possible to notice the big difference.

BLUE AREA

14W17						
	Excel			**Limeset**		
Load (kPa)	c_1**(m)**	c_2**(m)**	c_3**(m)**	c_1**(m)**	c_2**(m)**	c_3**(m)**
20	0.007	0.009	0.010	0.011	0.013	0.014
30	0.011	0.014	0.017	0.018	0.020	0.022
40	0.016	0.020	0.025	0.024	0.028	0.032

Table 5.2: Settlements calculation after the stabilization with the Lime-Cement columns in the point 14W17. c_1 corresponds to 0.8 m, c_2 to 1 m and c_3 to 1.2 m.

The rate of the settlements with time is represented in Figure 5.7.

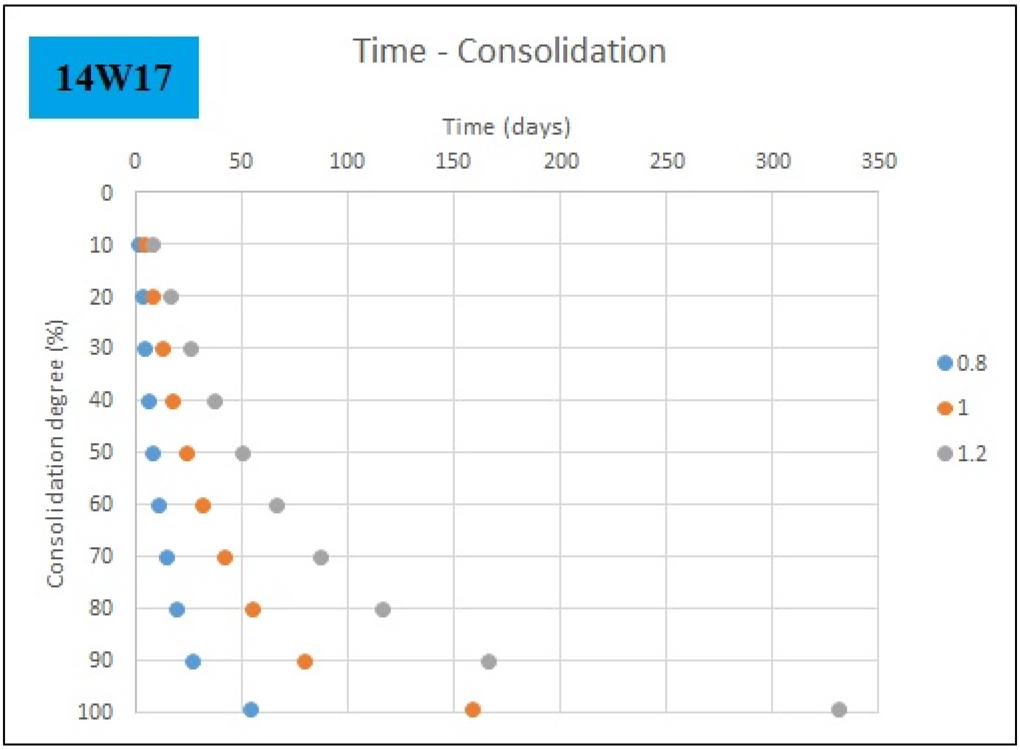


Figure 5.7: representation of the rate of the settlements with time in the point 14W17 after the LCC for three different centre distance c_1, in particular 0.8 m, 1 m and 1.2 m.

We can notice that the 90% of the consolidation is already reached after 150-170 days with a centre distance between the columns of 1.2 m which is the worst configuration, while before the reinforcement the required time was 5 years.

GREEN AREA

	30					
	Excel			Limeset		
Load (kPa)	c_1(m)	c_2(m)	c_3(m)	c_1(m)	c_2(m)	c_3(m)
20	0.015	0.021	0.028	0.018	0.024	0.030
30	0.024	0.035	0.046	0.029	0.038	0.049
40	0.035	0.049	0.065	0.039	0.053	0.069

Table 5.3: Settlements calculation after the stabilization with the Lime-Cement columns in the point 30. c_1 corresponds to 0.8 m, c_2 to 1 m and c_3 to 1.2 m.

The rate of the settlements with time is represented in Figure 5.8.

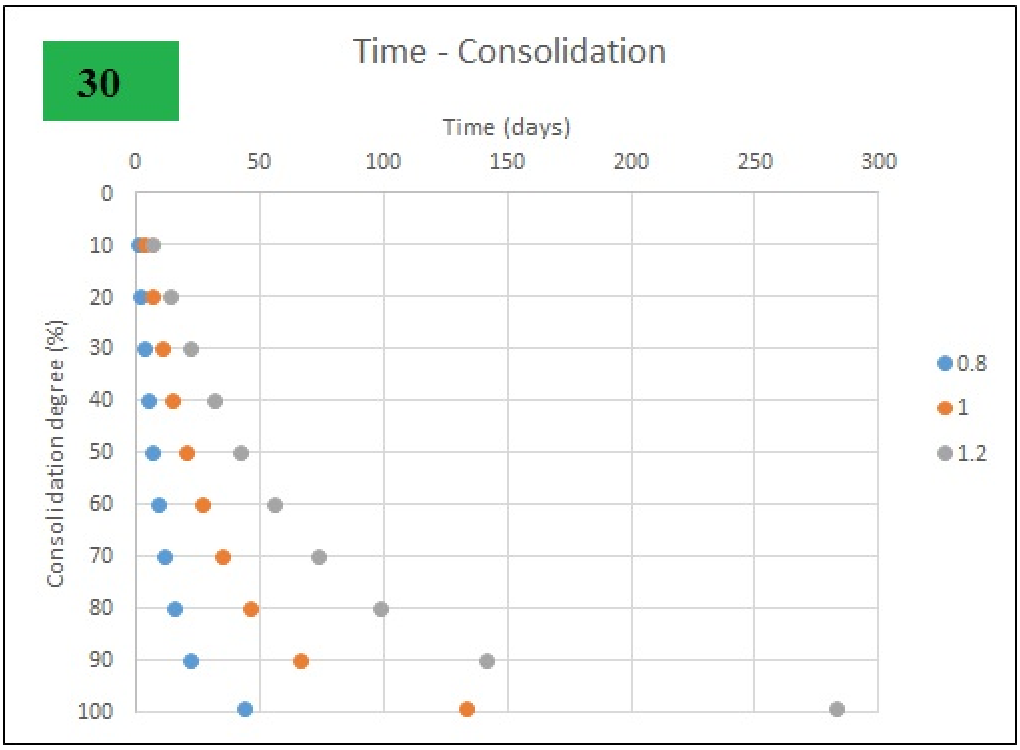


Figure 5.8: representation of the rate of the settlements with time in the point 30 after the LCC for three different centre distance c_1, in particular 0.8 m, 1 m and 1.2 m.

We can notice that the 90% of the consolidation is already reached after around 140 days with a centre distance between the columns of 1.2 m which is the worst configuration. Compared to the required time before the soil stabilization (3.5 years)it is possible to notice the difference.

YELLOW AREA

16B46						
	Excel			Limeset		
Load (kPa)	c_1(m)	c_2(m)	c_3(m)	c_1(m)	c_2(m)	c_3(m)
20	0.024	0.030	0.035	0.028	0.034	0.039
30	0.038	0.049	0.060	0.043	0.054	0.066
40	0.052	0.070	0.088	0.061	0.078	0.095

Table 5.4: Settlements calculation after the stabilization with the Lime-Cement columns in the point 16B46. c_1 corresponds to 0.8 m, c_2 to 1 m and c_3 to 1.2 m.

The rate of the settlements with time is represented in Figure 5.9.

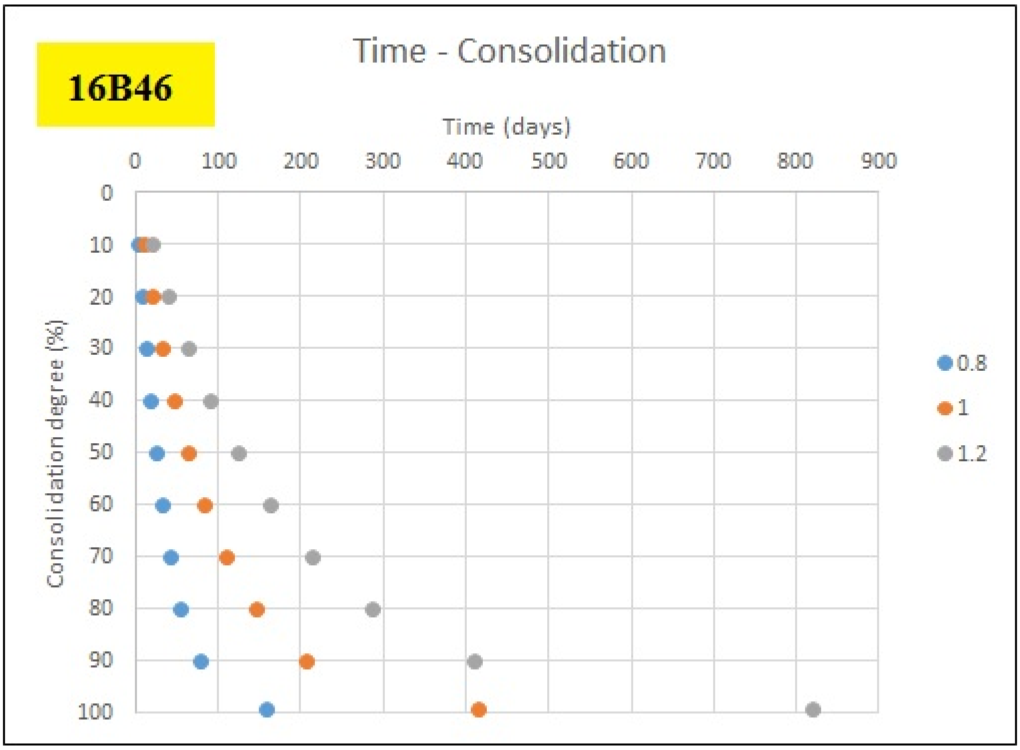


Figure 5.9: representation of the rate of the settlements with time in the point 16B46 after the LCC for three different centre distance c_1, in particular 0.8 m, 1 m and 1.2 m.

We can notice that the 90% of the consolidation is already reached after 400-420 days with a centre distance between the columns of 1.2 m which is the worst configuration, while before the reinforcement the required time was 26 years.

16B40						
	Excel			Limeset		
Load (kPa)	c_1**(m)**	c_2**(m)**	c_3**(m)**	c_1**(m)**	c_2**(m)**	c_3**(m)**
20	0.019	0.024	0.031	0.021	0.028	0.033
30	0.030	0.041	0.052	0.034	0.045	0.056
40	0.041	0.056	0.073	0.047	0.062	0.079

Table 5.5: Settlements calculation after the stabilization with the Lime-Cement columns in the point 16B40. c_1 corresponds to 0.8 m, c_2 to 1 m and c_3 to 1.2 m.

The rate of the settlements with time is represented in Figure 5.10.

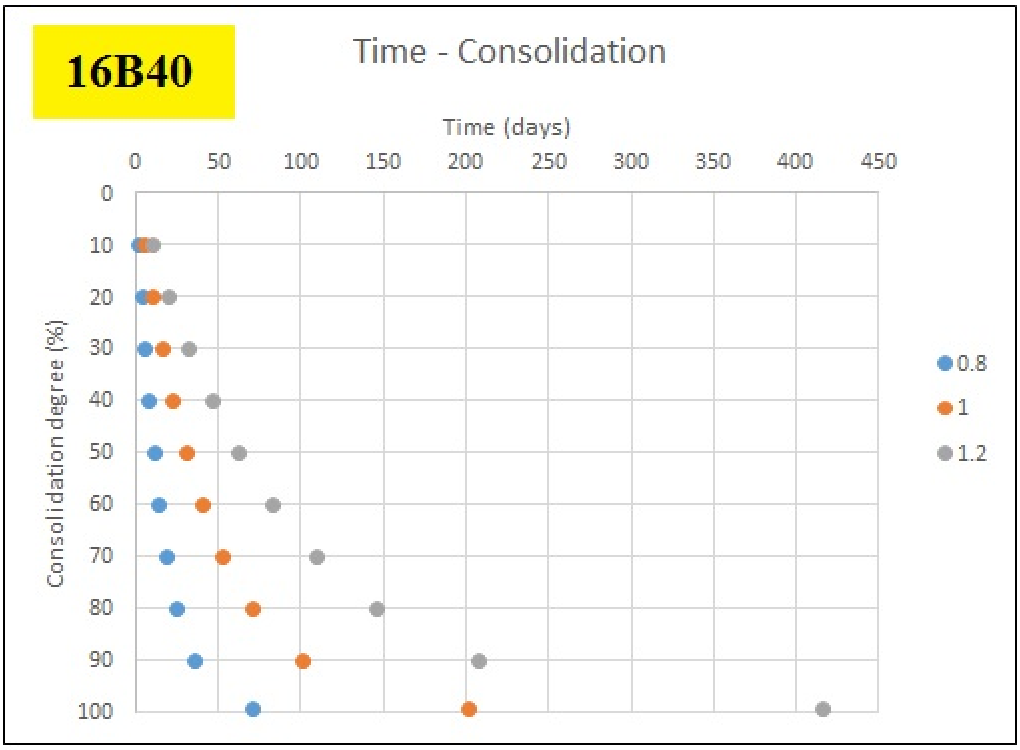


Figure 5.10: representation of the rate of the settlements with time in the point 16B40 after the LCC for three different centre distance c_1, in particular 0.8 m, 1 m and 1.2 m.

We can notice that the 90% of the consolidation is already reached after 200-220 days with a centre distance between the columns of 1.2 m which is the worst configuration, while before the reinforcement the required time was 8 years.

Looking at the resulting settlements before and after the reiforcement of the soil with the Lime-Cement columns it can be noticed the improvement given by the columns, which can both significantly reduce the settlements and the time for a complete consolidation. This confirms that the columns act also as a drain.

5.3 Slope stability after the LCC installation.

The stabilization of the soil with the Lime-Cement columns is useful not only for the settlements improvement, but also for the slope stability, especially where the excavations are necessary. In Dalvägen the excavations are necessary because the placing of the municipal water works and of the sewer system.
The effect of the LCC on the slope stability is to increase the values of the shear strength, which as seen before is very low for the clay we find in the area. As seen previously in equation 1.2.2, we have that:

$$c_{\mathrm{uk}} = a \times c_{\mathrm{uk(col)}} + (1 - a) \times c_{\mathrm{uk(clay)}}$$

Using GS stability it is possible to change the shear strength of the clay $c_{\mathrm{uk(clay)}}$ until a safety factor of 1.3 is reached. When these safety factors are reached, the inserted value of shear strength is equal to c_{uk}, the final value we want to reach after the stabilization. After that, knowing c_{uk}, $c_{\mathrm{uk(col)}}$ which is usually 100 kPa and $c_{\mathrm{uk(clay)}}$ which is the initial value of shear strength of the clay is it possible to calculate the factor a which is the ratio of the total column area to the total area of reinforced soil:

$$a = \frac{c_{\mathrm{uk}} - c_{\mathrm{uk(clay)}}}{c_{\mathrm{uk(col)}} - c_{\mathrm{uk(clay)}}} \tag{5.3.1}$$

Knowing a is then possible to make the columns planning, in particular to determine c, the distance between the column centres which guarantees a safety factor of 1.3.
This method is simple and applicable for the yellow and the green areas, where we have only one layer of clay, so we have a unique value of the shear strength. On the other hand, when dealing with the red and the blue areas, which are composed of 3 and 2 layers respectively, the adopted procedure

is a bit different.
Taking as an example the red area here we have three sub-layers of clay with a starting shear strength of 8.5, 12.5 and 17.5 kPa respectively. The possible methods I tried for calculating the required shear strength for reaching a SF of 1.3 are two:

- the first is to set a unique value of shear strength for all the three sub-layers and to change it in the software GS stability until a safety factor of 1.3 is reached. This is the fastest method, but the procedure is not totally correct, because the sub-layers are characterized by three different starting values of shear strength and the columns don't homogenize.

- the second is to decide a value of a and to calculate the resulting shear strength for the three sub-layers with equation 1.2.2, starting from each different shear strength value. Then these value of shear strength are inserted in the program and the corresponding slip surface and safety factor are calculated. This is a sort of trial and error method because the procedure is the same until a SF of 1.3 is reached. This method is more correct given that it implicates that the columns improve proportionally the different values of shear strength, and for this reason it is the one adopted in the calculations for the slope stability after the LCC installation.

5.3.1 Resulting slope stability after the LCC installation.

The figures representing the slip surface with a safety factor of 1.3 are here reported.

RED AREA
3 m excavation

SLOPE 1:1

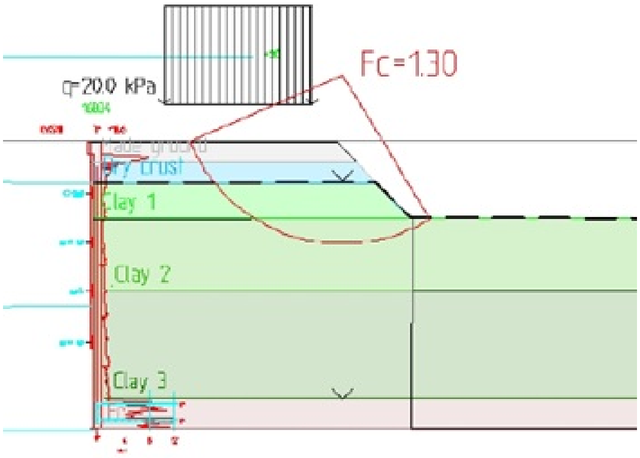


Figure 5.11: representation of the slip surface for a safety factor of 1.3 in 16B34 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

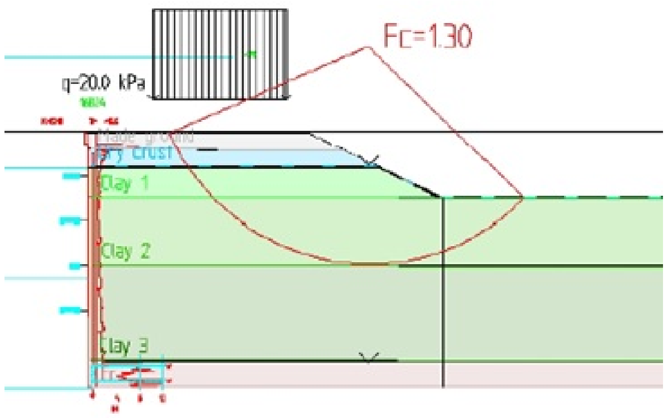


Figure 5.12: representation of the slip surface for a safety factor of 1.3 in 16B34 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

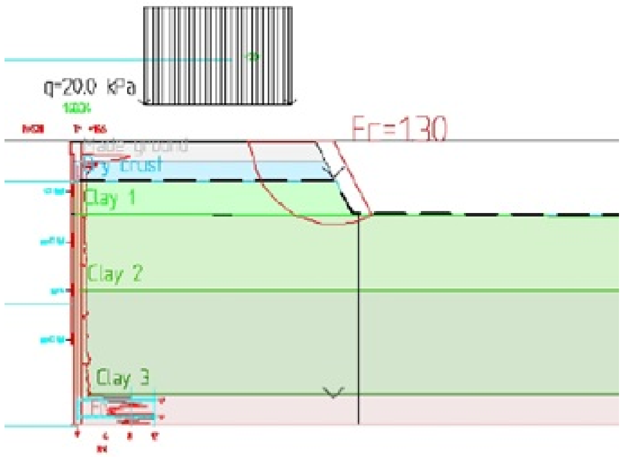


Figure 5.13: representation of the slip surface for a safety factor of 1.3 in 16B34 with slope 2:1 and 3 m of excavation.

RED AREA
4.5 m excavation

SLOPE 1:1

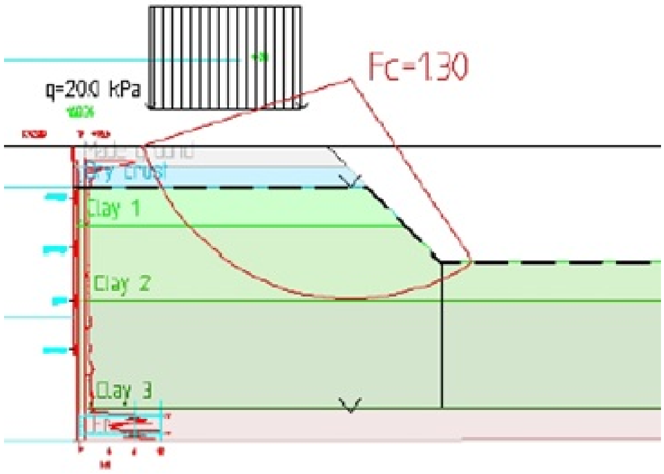


Figure 5.14: representation of the slip surface for a safety factor of 1.3 in 16B34 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

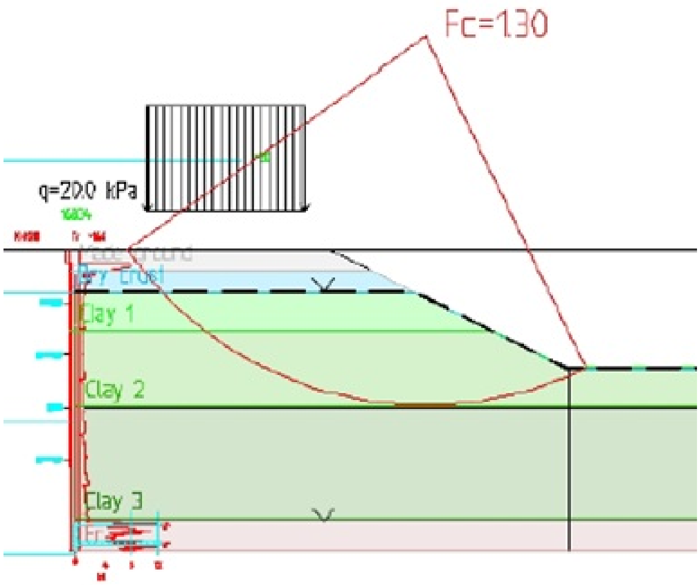


Figure 5.15: representation of the slip surface for a safety factor of 1.3 in 16B34 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

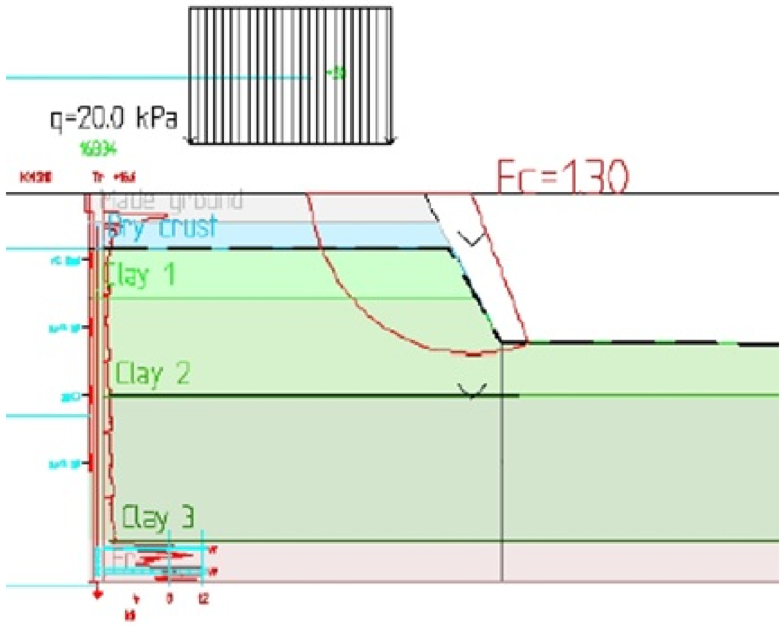


Figure 5.16: representation of the slip surface for a safety factor of 1.3 in 16B34 with slope 2:1 and 4.5 m of excavation.

The required shear strength for reaching a safety factor of 1.3 in the red area are reported in Table 5.6.

It is important to notice that the highest shear strength is required for reaching the safety factor of 1.3 where the slope is 2:1, which is the steepest, while the lowest shear strength is where the slope is 1:2 which is the less steep. More than this, the shear strength is higher for the deepest layer, while it is lower for the shallowest because it reflects the initial value of shear strength.
It can also be noticed that the values of shear strenth for reaching the safety factor in the example of 4.5 m of excavation are quite higher compared to the 3 m of excavation.

RED AREA					
Slope		**τ (3m) (kPa)**	**τ (4.5m) (kPa)**	**a (3m)**	**a (4.5m)**
2:1	1^{st} sublayer	18.5	25.6		
	2^{nd} sublayer	21.5	28.3	0.105	0.183
	3^{rd} sublayer	27.1	33.5		
1:1	1^{st} sublayer	15.7	22.1		
	2^{nd} sublayer	18.9	25.0	0.075	0.145
	3^{rd} sublayer	24.7	30.4		
1:2	1^{st} sublayer	14.1	17.7		
	2^{nd} sublayer	17.3	20.8	0.057	0.097
	3^{rd} sublayer	23.2	26.5		

Table 5.6: Values of shear strength and a necessary fo reaching a SF of 1.3 for the different slope for the point 16B34 and for the two different excavations.

BLUE AREA
3 m excavation

SLOPE 1:1

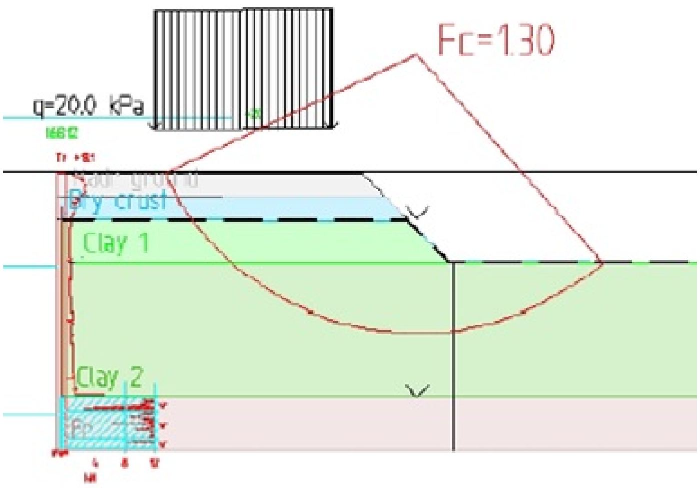


Figure 5.17: representation of the real slip surface for a safety factor of 1.3 in 16B12 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

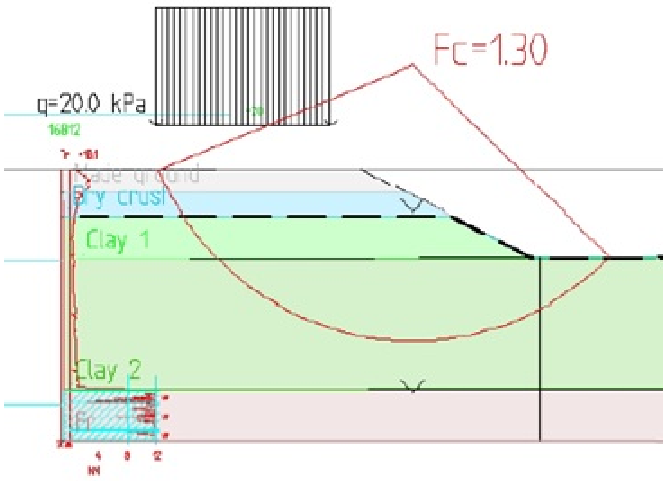


Figure 5.18: representation of the slip surface for a safety factor of 1.3 in 16B12 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

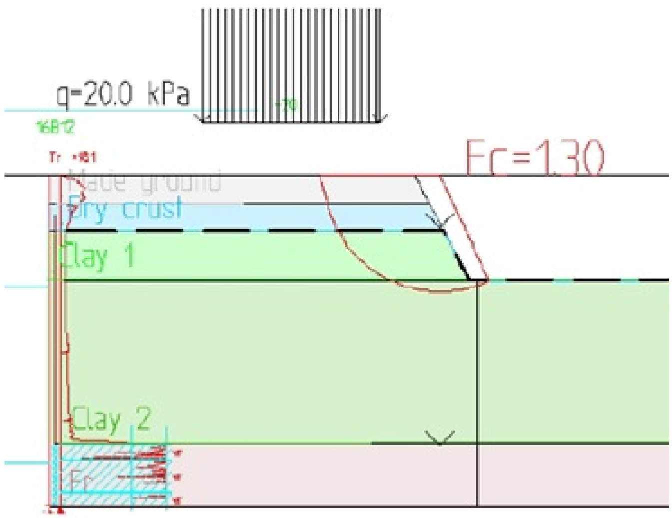


Figure 5.19: representation of the slip surface for a safety factor of 1.3 in 16B12 with slope 2:1 and 3 m of excavation.

BLUE AREA
4.5 m excavation

SLOPE 1:1

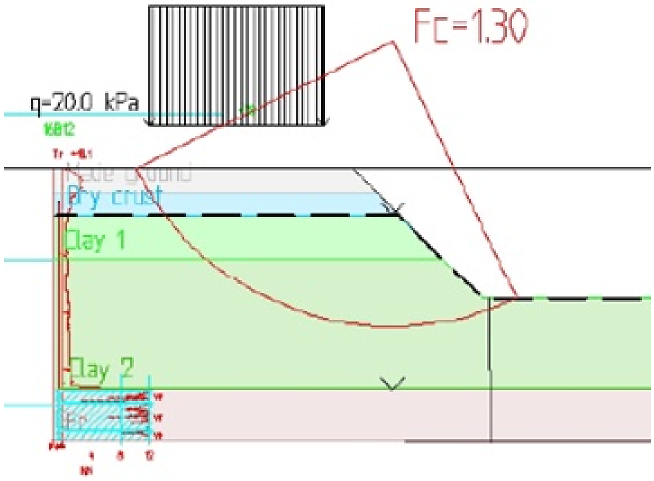


Figure 5.20: representation of the real slip surface for a safety factor of 1.3 in 16B12 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

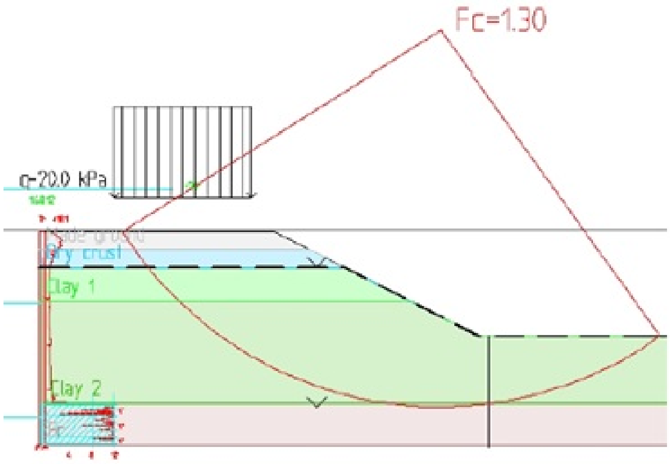


Figure 5.21: representation of the slip surface for a safety factor of 1.3 in 16B12 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

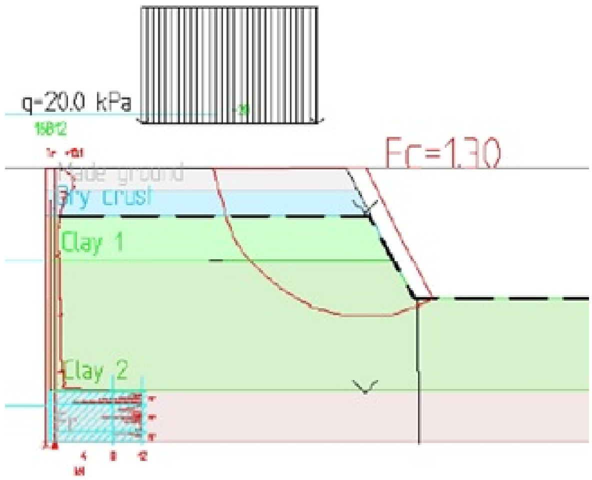


Figure 5.22: representation of the slip surface for a safety factor of 1.3 in 16B12 with slope 2:1 and 4.5 m of excavation.

The required shear strength for reaching a safety factor of 1.3 in the blue area are reported in Table 5.7.

BLUE AREA					
Slope		**τ (3m) (kPa)**	**τ (4.5m) (kPa)**	**a (3m)**	**a (4.5m)**
2:1	1^{st} sublayer	19.2	25.8	0.099	0.173
	2^{nd} sublayer	21.6	28		
1:1	1^{st} sublayer	15.6	22.3	0.059	0.134
	2^{nd} sublayer	18.1	24.7		
1:2	1^{st} sublayer	15.0	18.2	0.052	0.088
	2^{nd} sublayer	17.5	20.7		

Table 5.7: Values of shear strength and a necessary for reaching a SF of 1.3 for the different slope for the point 16B12 and for the two different excavations.

Even in this case the highest shear strength is required for reaching the safety factor of 1.3 where the slope is 2:1, which is the steepest, while the lowest shear strength is where the slope is 1:2 which is the less steep. More than this, the shear strength is higher for the deepest layer, while it is lower for the shallowest because it reflects the initial value of shear strength. Even in this case the values of shear strenth for reaching the safety factor in the example of 4.5 m of excavation are quite higher compared to the 3 m of excavation.

GREEN AREA
3 m excavation

SLOPE 1:1

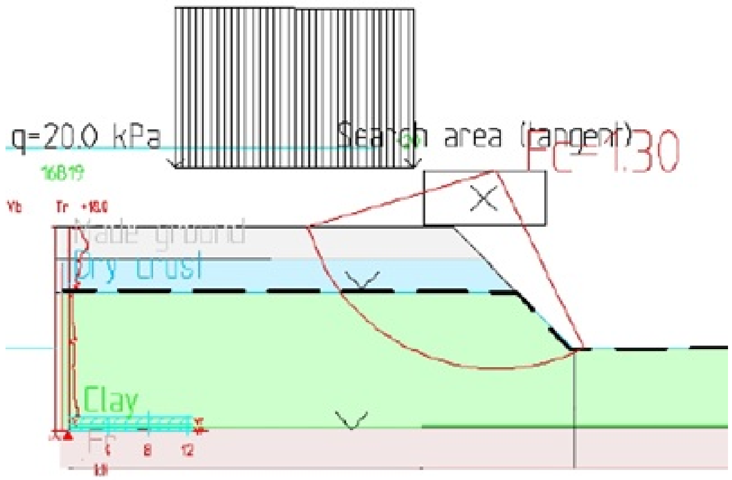


Figure 5.23: representation of the slip surface for a safety factor of 1.3 in 16B19 with slope 1:1 and 3 m of excavation.

SLOPE 2:1

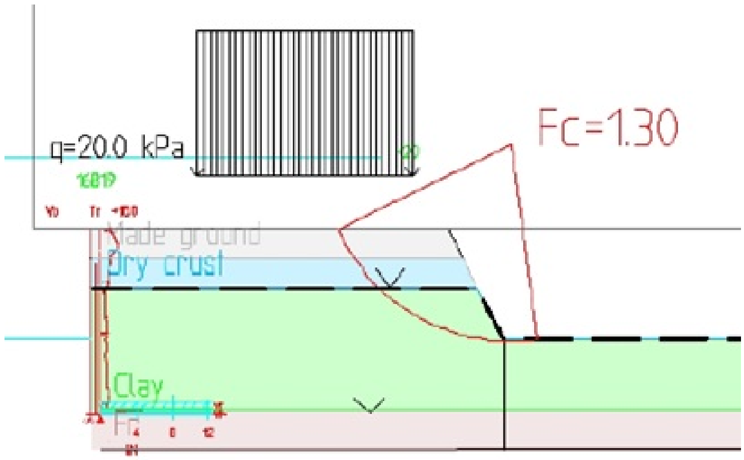


Figure 5.24: representation of the slip surface for a safety factor of 1.3 in 16B19 with slope 2:1 and 3 m of excavation.

GREEN AREA
4.5 m excavation

SLOPE 1:1

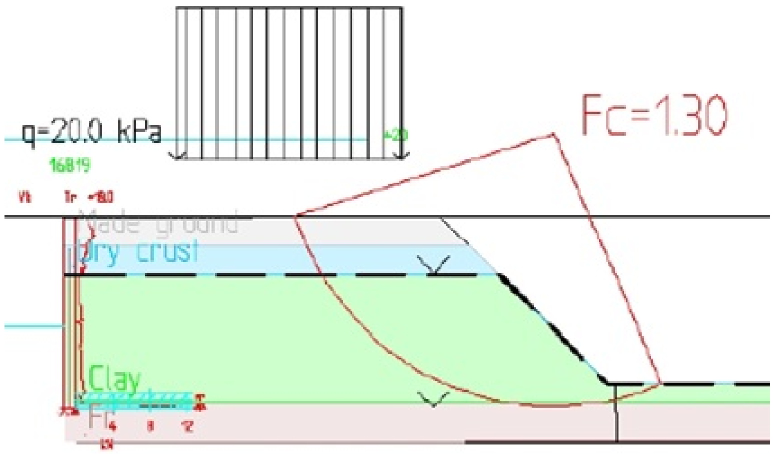


Figure 5.25: representation of the slip surface for a safety factor of 1.3 in 16B19 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

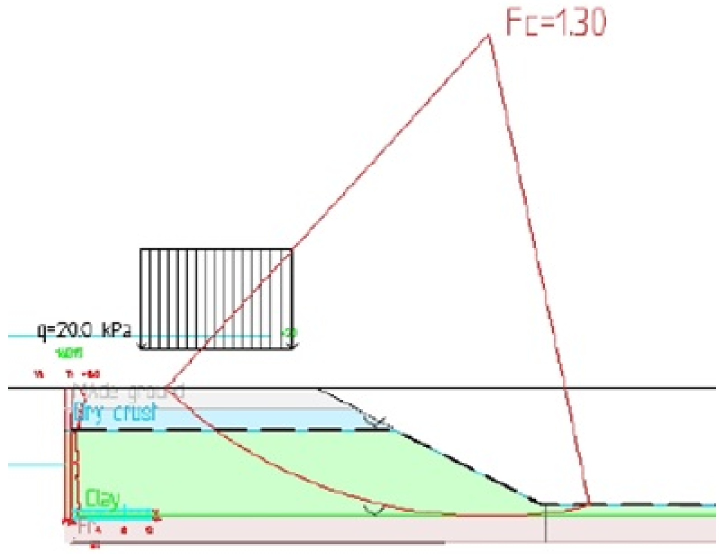


Figure 5.26: representation of the slip surface for a safety factor of 1.3 in 16B19 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

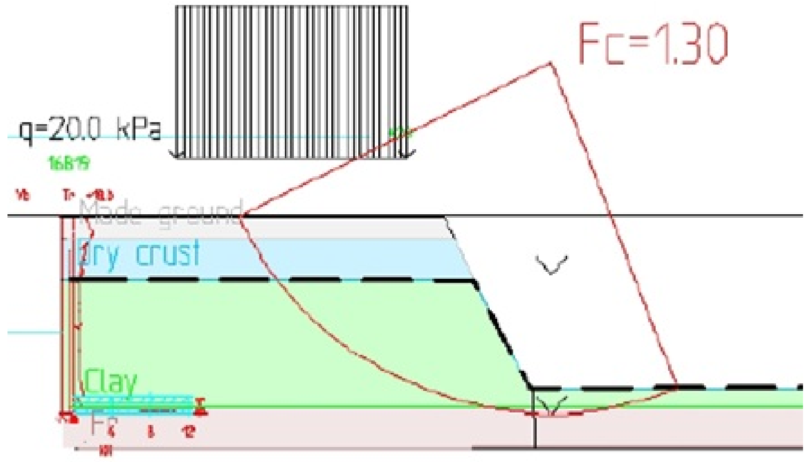


Figure 5.27: representation of the slip surface for a safety factor of 1.3 in 16B19 with slope 2:1 and 4.5 m of excavation.

The required shear strength for reaching a safety factor of 1.3 in the green area are reported in Table 5.8.

GREEN AREA					
Slope		**τ (3m) (kPa)**	**τ (4.5m) (kPa)**	**a (3m)**	**a (4.5m)**
2:1	1^{st} sublayer	19.2	31.2	0.049	0.190
1:1	1^{st} sublayer	19	27.3	0.047	0.145
1:2	1^{st} sublayer		17.7		0.032

Table 5.8: Values of shear strength and a necessary fo reaching a SF of 1.3 for the different slope for the point 16B19 and for the two different excavations.

Even in this case the highest shear strength is required for reaching the safety factor of 1.3 where the slope is 2:1, which is the steepest, while the lowest shear strength is where the slope is 1:2 which is the less steep.
Even in this case the values of shear strenth for reaching the safety factor in the example of 4.5 m of excavation are quite higher compared to the 3 m of excavation.
The shear strength for the 3 m excavation and for the slope configuration 1:2 has not been calculated because the safety factor was 1.34 even without stabilization.
The values of shear strength for the 4.5 m excavation and for the slope 2:1 and 1:1 are quite high, and they reflect the shape of the slip surface which for geometrical reasons has a very high center and involves a bigger quantity of soil, if compared to all the other examples.

YELLOW AREA
3 m excavation

SLOPE 1:1

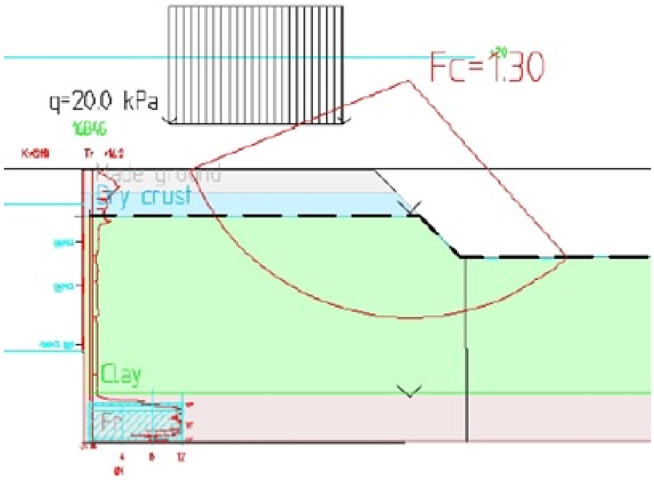


Figure 5.28: representation of the real slip surface for a safety factor of 1.3 in 16B46 with slope 1:1 and 3 m of excavation.

SLOPE 1:2

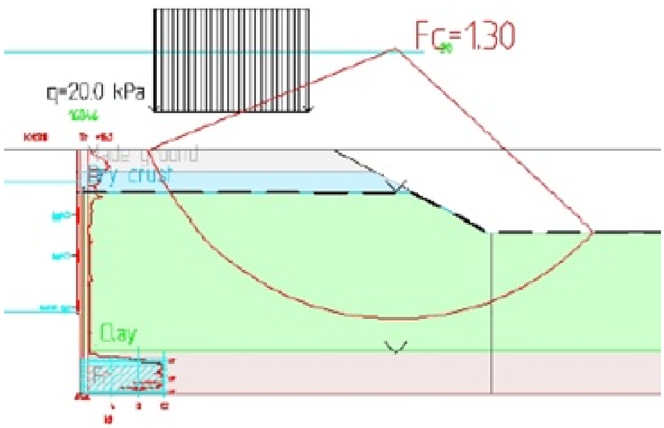


Figure 5.29: representation of the slip surface for a safety factor of 1.3 in 16B46 with slope 1:2 and 3 m of excavation.

SLOPE 2:1

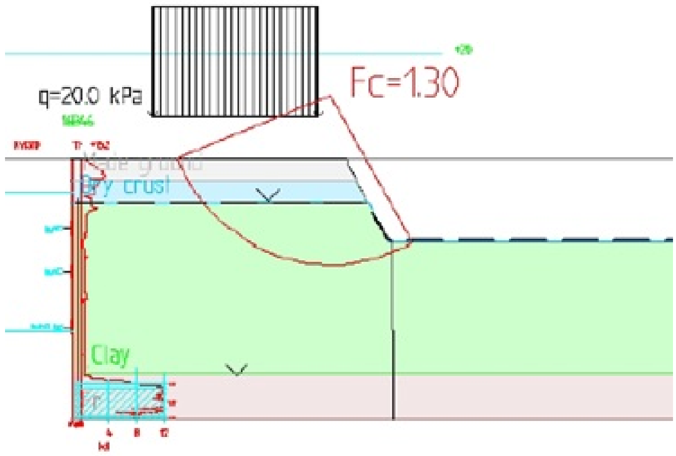


Figure 5.30: representation of the slip surface for a safety factor of 1.3 in 16B46 with slope 2:1 and 3 m of excavation.

YELLOW AREA
4.5 m excavation

SLOPE 1:1

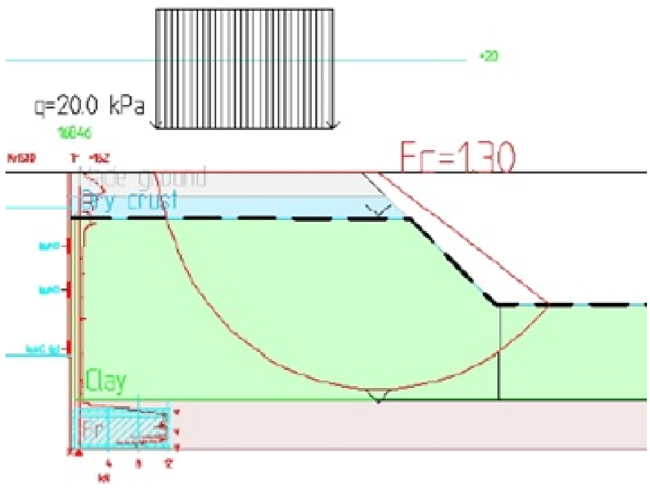


Figure 5.31: representation of the real slip surface for a safety factor of 1.3 in 16B46 with slope 1:1 and 4.5 m of excavation.

SLOPE 1:2

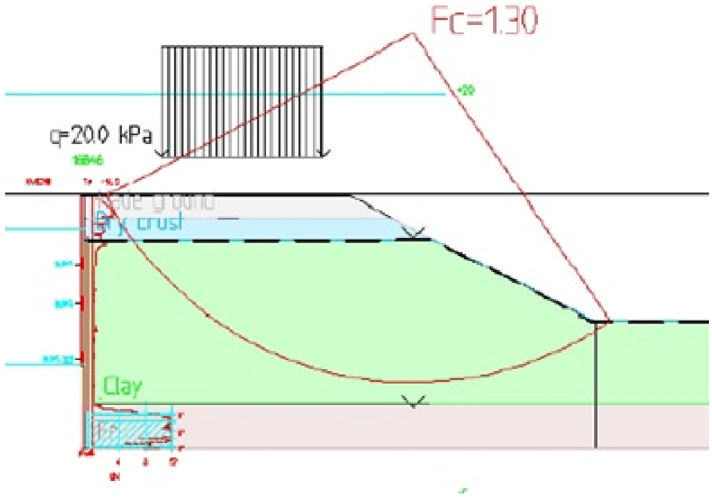


Figure 5.32: representation of the slip surface for a safety factor of 1.3 in 16B46 with slope 1:2 and 4.5 m of excavation.

SLOPE 2:1

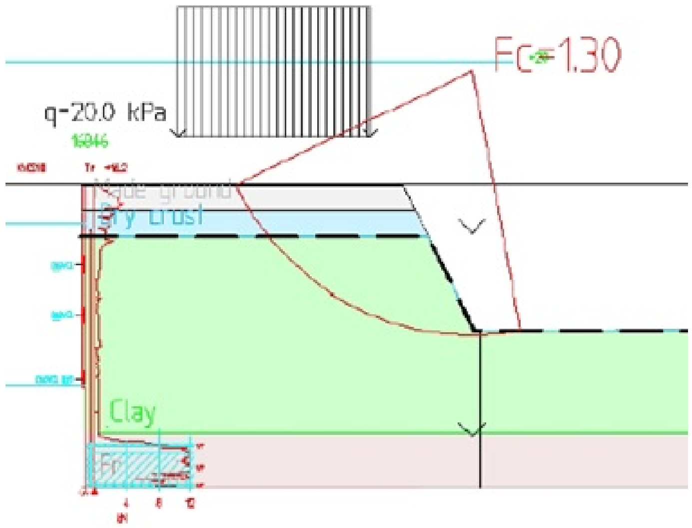


Figure 5.33: representation of the slip surface for a safety factor of 1.3 in 16B46 with slope 2:1 and 4.5 m of excavation.

The required shear strength for reaching a safety factor of 1.3 in the yellow area are reported in Table 5.9.

Even in this case the highest shear strength is required for reaching the safety factor of 1.3 where the slope is 2:1, which is the steepest, while the

YELLOW AREA					
Slope		**τ (3m) (kPa)**	**τ (4.5m) (kPa)**	**a (3m)**	**a (4.5m)**
2:1	1^{st} sublayer	18.6	27.2	0.059	0.158
1:1	1^{st} sublayer	18.4	23.8	0.057	0.119
1:2	1^{st} sublayer	17.6	21	0.047	0.087

Table 5.9: Values of shear strength and a necessary fo reaching a SF of 1.3 for the different slope for the point 16B46 and for the two different excavations.

lowest shear strength is where the slope is 1:2 which is the less steep.
The values of shear strenth for reaching the safety factor in the example of 4.5 m of excavation are quite higher compared to the 3 m of excavation.

5.4 Columns planning

When dealing with both settlements and slope stability, the arrangement of the columns that has to be adopted is represented in Figure 5.34.

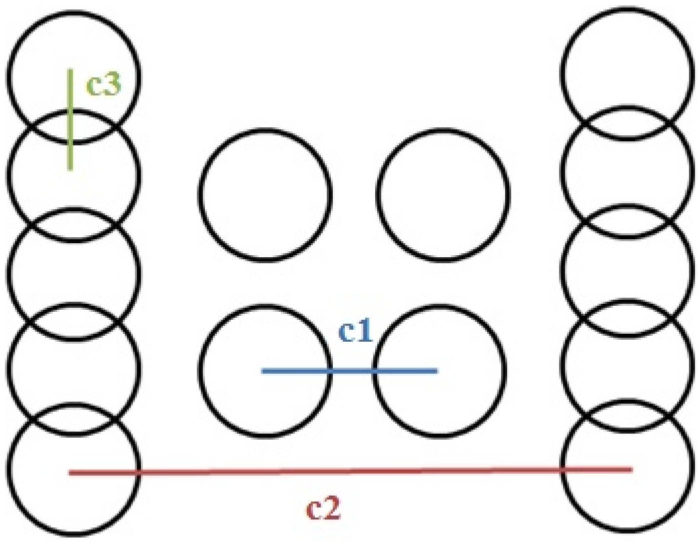


Figure 5.34: Representation of the pattern of the columns.

The arrangement in single columns is suitable for improving the settlements, and the parameter that is given by the calculations is c_1. On the other hand, the arrangement in slab is necessary when dealing with the excavations, and the parameter given by the calculations is c_2, while c_3 is usually 0.5 m because the columns overlap for at least 10 cm.

As mentioned before, the value a of the slab arrangement, which is the ratio between the total column area to the total area of the reinforced soil can be obtained knowing the c_{uk} for reaching the desired safety factor. The calculated values of a which guarantee a safety factor of 1.3 have been previously reported in the Tables 5.6, 5.7, 5.8 and 5.9.
From these values it is then possible to determine the sevral distances c_2, given Figure 5.35 and the following equations.

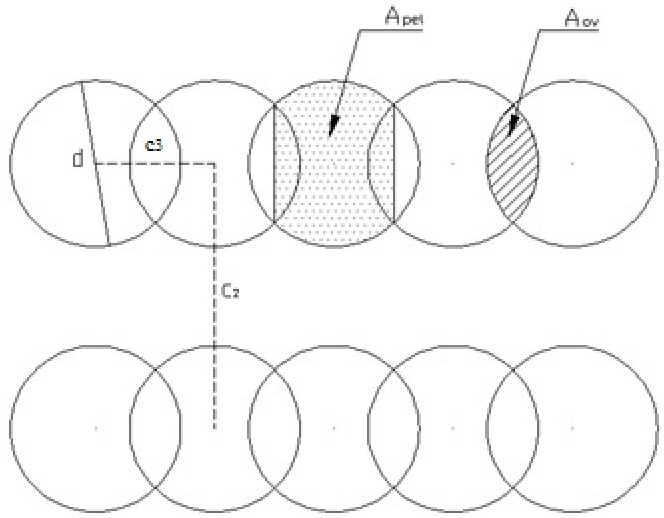


Figure 5.35: Representation of slab pattern for the columns.

As previously mentioned in Equation 1.2.3 we have:

$$a = \frac{A}{c^2}$$

but in the case of slab we have:

$$a = \frac{A_{\text{pel}}}{c_3 \times c_2} \tag{5.4.1}$$

where A_{pel}, c_3 and c_2 are represented in Figure 5.35.

A_{pel} is equal to:

$$A_{\text{pel}} = \frac{\pi d^2}{4} - A_{\text{ov}} \tag{5.4.2}$$

where A_{ov} which is the area of overlapping of the columns is equal to:

$$A_{\text{ov}} = \frac{d^2}{2} \times \left[\arccos\left(\frac{c_3}{d}\right) - \frac{c_3}{d}\sqrt{1 - \left(\frac{c_3}{d}\right)^2} \right] \tag{5.4.3}$$

The values of c_2 is obtained from the equation above, and the results for the different excavations and areas are here reported:

RED AREA		
Slope	**c_2 (m) excav. 3 m**	**c_2 (m) excav. 4.5 m**
2:1	4.96	2.84
1:1	6.94	3.59
1:2	9.13	5.37

Table 5.10: Values of c_2 necessary for reaching the shear strength of Table 5.6 that ensure a SF of 1.3, for the different slope for the point 16B46 and for the two different excavations.

BLUE AREA		
Slope	**c_2 (m) excav. 3 m**	**c_2 (m) excav. 4.5 m**
2:1	5.25	3.00
1:1	8.82	3.88
1:2	10.00	5.91

Table 5.11: Values of c_2 necessary for reaching the shear strength of Table 5.7 that ensure a SF of 1.3, for the different slope for the point 16B12 and for the two different excavations.

GREEN AREA		
Slope	**c_2 (m) excav. 3 m**	**c_2 (m) excav. 4.5 m**
2:1	10.54	2.73
1:1	11.06	3.59
1:2		16.39

Table 5.12: Values of c_2 necessary for reaching the shear strength of Table 5.8 that ensure a SF of 1.3, for the different slope for the point 16B19 and for the two different excavations.

As previously discussed, notwithstanding the green area is the less critical from every point of view, for the excavation of 4.5 m and for the configurations 2:1 and 1:1, the green area presents very high value of the shear strength necessary to reach the safety factor of 1.3 with consequent very low value of c_2. This is essentially due to geometrial reasons.

YELLOW AREA		
Slope	**c_2 (m) excav. 3 m**	**c_2 (m) excav. 4.5 m**
2:1	8.83	3.29
1:1	9.19	4.37
1:2	10.98	6.00

Table 5.13: Values of c_2 necessary for reaching the shear strength of Table 5.9 that ensure a SF of 1.3, for the different slope for the point 16B46 and for the two different excavations.

Observing the results it is possible to notice the difference of the value c_2 for the two different excavations. The excavation of 4.5 m gives habitual values of c_2, while the other are less ordinary. Obviously, the big difference in the columns distance influences the total price and demonstrates that a correct and efficient operation of the columns depends on the correct planning of they.

For determine the distance c_1 it is necessary to consider the settlement calculations after the LCC stabilization. Considering the load of 20 kPa which is the same used for the stability calculations, the distance c_1 which

allows to have settlements lower than 5 cm in every area is equal to 1 m, so this could be the c_1 adopted.
Notwithstanding this, in some cases where the pipeline that has to be layed is particularly breakable, settlements lower than 1 cm are necessary. For this reason in some cases as in the red area where in no-one case (different load or c_1) the settlements are less than 1 cm, the use of the pre-loads is quite necessary.

5.4.1 Pre-loads

This method consists in the application of an additional load to force the consolidation to happen during a smaller period of time. After this time the settlements will be totally finished and this will ensure a total safety.
The application of the pre-loads is very useful in some cases as the case of this work, because of the laying of the pipeline and of the sewer system. The dissipation of the settlements in a small period of time will in fact guarantee a correct and totally operation of the tubes.

RED AREA-14W44

Considering the red area, for a centre distance of 1 m and for a load of 20 kPa the settlements are 0.05 m while for a load of 40 kPa the settlements are 0.118 m. Considering the coefficient of consolidation of the red area ($7.3 \times 10^{-}9\frac{m^2}{s}$), a columns length of 8.3 m and a permeability of the columns equal to 500 times the permeability of the clay (standard parameter for the swedish regulations), the required times for the different degree of consolidation are reported in Table 5.36.

RED AREA			
U (%)	time (days)	Settl. 20 kPa (m)	Settl. 40 kPa (m)
10	12.66	0.005	0.012
20	26.81	0.010	0.024
30	42.86	0.015	0.036
40	61.39	0.020	0.047
42	65.46	0.021	0.050
50	83.29	0.025	0.059
60	110.11	0.030	0.071
70	144.68	0.035	0.083
80	193.40	0.040	0.095
90	276.70	0.045	0.107
99	553.40	0.050	0.117

Table 5.14: Time necessary for the specified consolidation degree in the red area. In the last two columns the settlements correponding to the consolidation degree for a load of 20 and 40 kPa.

The development of the settlements during time is showed in Figure 5.36.

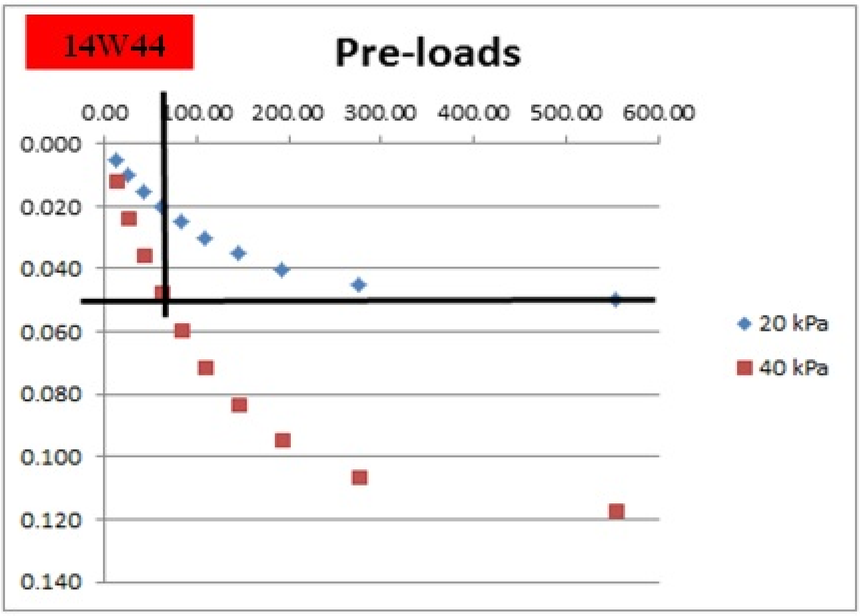


Figure 5.36: Development of the settlements during time for a load of 20 and 40 kPa in the red area.

The black lines shows that if a pre-load of 20 kPa (corresponding to the difference between 20 and 40) is applied, all the settlements we would have with a load of 20 kPa in more than 500 days are now spended in less than 100 days, in particular around 42 days.

BLUE AREA-14W17

Considering the blue area, for a centre distance of 1 m and for a load of 20 kPa the settlements are 0.009 m, which is already smaller than 1 cm. For this reason the pre-loads are not necessary.

GREEN AREA-30

Considering the green area, for a centre distance of 1 m and for a load of 20 kPa the settlements are 0.021 m while for a load of 40 kPa the settlements are 0.049 m. Considering the coefficient of consolidation of the green area ($1.56 \times 10^{-}8\frac{m^2}{s}$), a columns length of 3.3 m and a permeability of the columns equal to 500 times the permeability of the clay (standard parameter for the swedish regulations), the required times for the different degree of consolidation are reported in Table 5.15.

GREEN AREA			
U (%)	**time (days)**	**Settl. 20 kPa (m)**	**Settl. 40 kPa (m)**
10	3.05	0.002	0.005
20	6.45	0.004	0.010
30	10.32	0.006	0.015
40	14.77	0.008	0.020
50	20.05	0.011	0.025
60	26.50	0.013	0.030
70	34.82	0.015	0.034
80	46.55	0.017	0.039
90	66.60	0.019	0.044
99	133.20	0.021	0.049

Table 5.15: Time necessary for the specified consolidation degree in the green area. In the last two columns the settlements correponding to the consolidation degree for a load of 20 and 40 kPa.

The development of the settlements during time is showed in Figure 5.37.

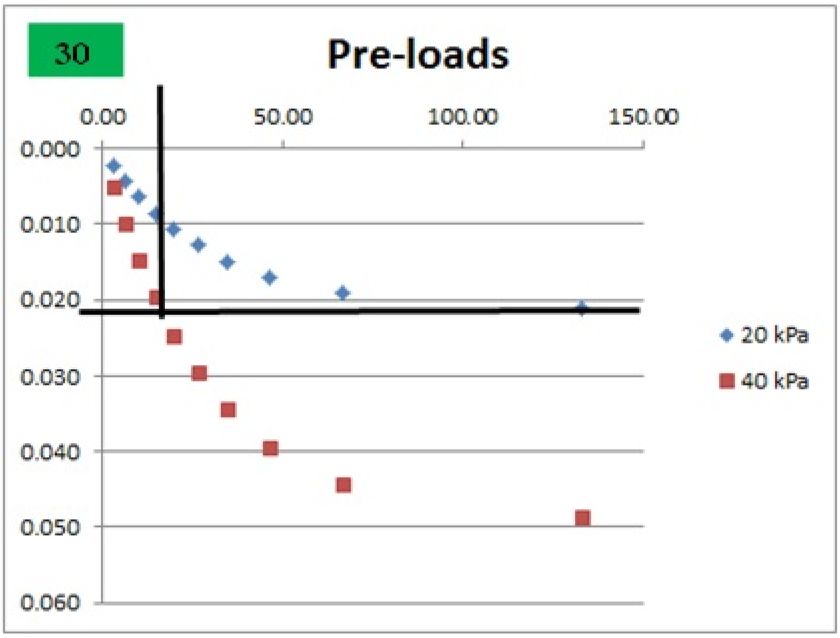


Figure 5.37: Development of the settlements during time for a load of 20 and 40 kPa in the green area.

The black lines shows that if a pre-load of 20 kPa (corresponding to the difference between 20 and 40) is applied, all the settlements we would have with a load of 20 kPa in less than 150 days are now spended in less than 50 days, in particular around 40 days.

YELLOW AREA-16B46

Considering the yellow area (point 16B46), for a centre distance of 1 m and for a load of 20 kPa the settlements are 0.030 m while for a load of 40 kPa the settlements are 0.070 m. Considering the coefficient of consolidation of the yellow area in the point 16B46 ($7.06 \times 10^-9 \frac{m^2}{s}$), a columns length of 6 m and a permeability of the columns equal to 500 times the permeability of the clay (standard parameter for the swedish regulations), the required times for the different degree of consolidation are reported in Table 5.16.

YELLOW AREA			
U (%)	time (days)	Settl. 20 kPa (m)	Settl. 40 kPa (m)
10	9.49	0.003	0.007
20	20.09	0.006	0.014
30	32.11	0.009	0.021
40	45.99	0.012	0.028
43	50.33	0.013	0.030
50	62.41	0.015	0.035
60	82.50	0.018	0.042
70	108.40	0.021	0.049
80	144.91	0.024	0.056
90	207.31	0.027	0.063
99	414.63	0.030	0.069

Table 5.16: Time necessary for the specified consolidation degree in the yellow area point 16B46. In the last two columns the settlements correponding to the consolidation degree for a load of 20 and 40 kPa.

The development of the settlements during time is showed in Figure 5.38.

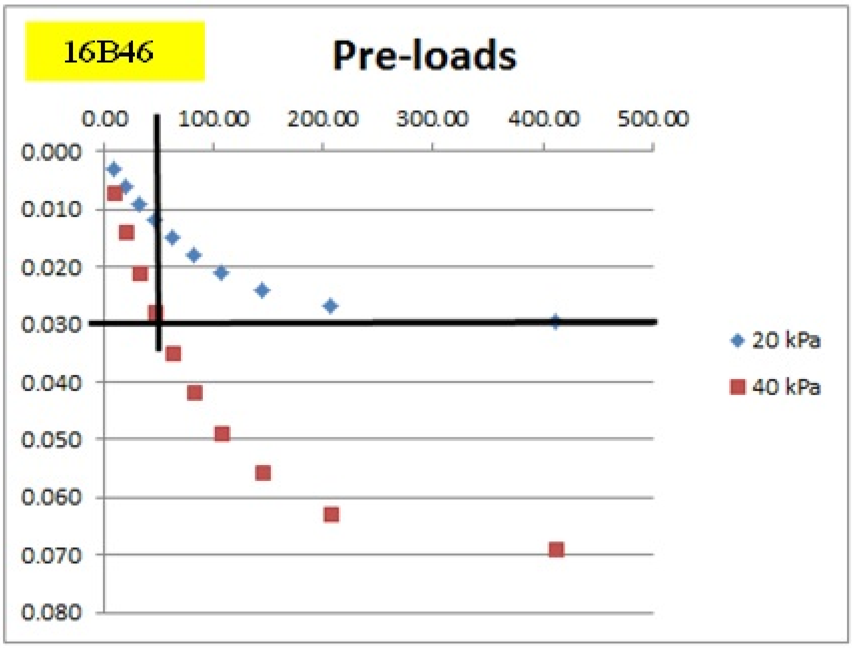


Figure 5.38: Development of the settlements during time for a load of 20 and 40 kPa in the yellow area point 16B46.

The black lines shows that if a pre-load of 20 kPa (corresponding to the difference between 20 and 40) is applied, all the settlements we would have with a load of 20 kPa in more than 400 days are now spended in less than 100 days, in particular around 43 days.

YELLOW AREA-16B40

Considering the yellow area (point 16B40), for a centre distance of 1 m and for a load of 20 kPa the settlements are 0.024 m while for a load of 40 kPa the settlements are 0.056 m. Considering the coefficient of consolidation of the yellow area in the point 16B40 ($1.16 \times 10^{-}8\frac{m^2}{s}$), a columns length of 4.3 m and a permeability of the columns equal to 500 times the permeability of the clay (standard parameter for the swedish regulations), the required times for the different degree of consolidation are reported in Table 5.39.

YELLOW AREA			
U (%)	**time (days)**	**Settl. 20 kPa (m)**	**Settl. 40 kPa (m)**
10	4.61	0.002	0.006
20	9.75	0.005	0.011
30	15.59	0.007	0.017
40	22.33	0.010	0.022
42	23.81	0.010	0.024
50	30.30	0.012	0.028
60	40.05	0.015	0.034
70	52.62	0.017	0.039
80	70.35	0.019	0.045
90	100.64	0.022	0.051
99	201.29	0.024	0.056

Table 5.17: Time necessary for the specified consolidation degree in the yellow area point 16B40. In the last two columns the settlements correponding to the consolidation degree for a load of 20 and 40 kPa.

The development of the settlements during time is showed in Figure 5.17.

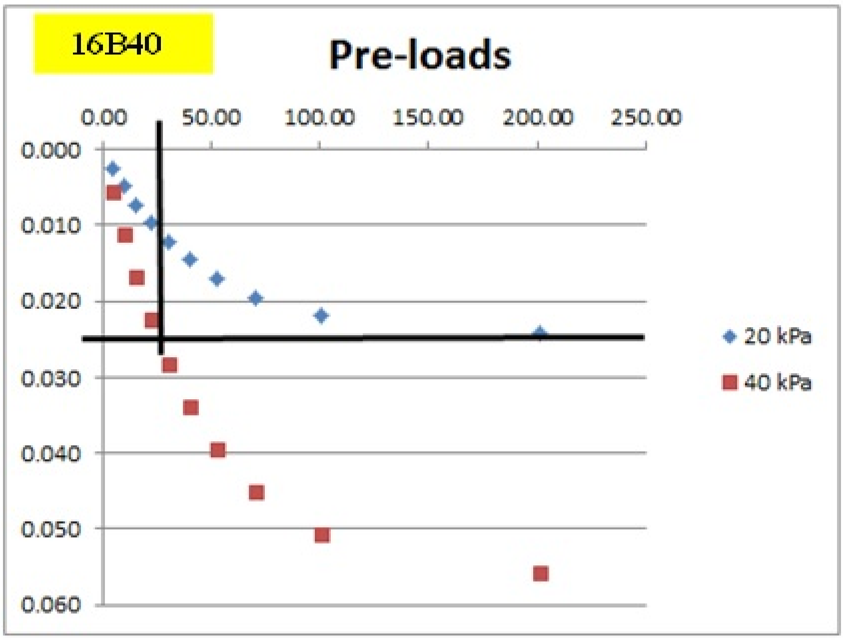


Figure 5.39: Development of the settlements during time for a load of 20 and 40 kPa in the yellow area point 16B40.

The black lines shows that if a pre-load of 20 kPa (corresponding to the difference between 20 and 40) is applied, all the settlements we would have with a load of 20 kPa in around 200 days are now spended in less than 50 days, in particular around 42 days.

Capitolo 6

CONCLUSIONS.

The purpose of this work was to analyse a real case of soil reinforcement, passing through a first analysis of the geotechnical conditions of the soil, a analysis of the two major problems when dealing with clay, settlements and slope stability and a final analysis of the motivations that led to the choice of the Lime-Cement columns method.

The analysis of the soil conditions led to the creation of 4 different geotechnical models of the soil (Chapter 2, page 64-79) and showed that the red area is characterized by the worst soil conditions, more clay (around 10 m) and smaller shear strength. On the opposite side, the green area is characterized by the best soil conditions, in fact in this area the thickness of clay is the lowest (less than 4 m) and the shear strength is the highest (15 kPa). The blue and the yellow area are similar, but the yellow is characterzied by a slightly higher thickness of clay and shear strength.

The analysis of the settlements has been conducted thanks to the CRS result, which provided the main deformation properties as the Modulus and the pre-consolidation pressure, necessary for the calculations, which have been performed with both an excel sheet and a software, GeoSuite Settlements. The calculations showed that the settlements are quite significant in all the areas, especially in the red one. In fact, for a load of 20 kPa they swing between a maximum of 0.270 m (red area, page 88) to a minimum of 0.022 m (blue area, page 90), while for a load of 30 kPa they swing betweem a maximum of 0.556 m (red area) and a minimum of 0.059 m (blue area).

These value of the settlements are surely not compatible with the project of Dalvägen, not so much for the construction of the road but for the laying of the pipeline. For this reason, the planning of the reinforcement of the soil was really essential. Another cause that led to a reiforcement of the soil is the rate of the settlements with time, that showed very long time for a complete consolidation (page 101), in fact it is around 50 years for the red area and 25 years for the yellow area, point 16B46.
A sensitivity analysis was performed to identify the parameter that most influence the settlements (page 99-100). It showed that the pre-consolidation pressure (and consequently the OCR degree) and the deformation parameters and in particular the Modulus have a bigger influence on the settlements than the thickness of the clay. This is highlighted by the resulting settlements of the green area, that notwithstanding it is characterized by the smallest thickness of clay, it shows higher settlements than the blue area and in some case even than the yellow area point 16B46 (see the recapitulatory Table at page 98). Another important result from the sensitivity analysis is that the OCR degree is the main parameter, because it has more influence on the settlements than the Modulus, given that the relation between the pre-consolidation pressure and settlements is linear.

Considering the slope stability analysis, the safety factor of 1.3 established by the swedish regulations is reached only in the green zone, in the 3 m excavation and only in the slope configuration of 1:2 (page 118), which is also not very significant and realistic, because usually the excavations have a slope of at least 1:1. As showed in the recapitulatory Table at page 125 and according to the geotechnical soil models, the safety factors are highest for the green area and lowest for the red area, while they are halfway for the blue and the yellow areas. This reflects the values of the main parameter for the slope stability analysis, which is obviously the shear strength.

Chaper 5 shows the many advantages of using the Lime-Cement columns instead of other methods, in both practical and economical points of view. A price analysis is performed and it underlines the very strong convenience of the columns, whose price is very derisory compared to other, in fact it is around 10 euros for 1 m of a standard column with 60 cm of diameter.
Explained the use of the columns, the calculations of the settlements after

the reinforcement are performed, with the aim to show the improvement that can be obtained with the columns. The calculations of the settlements after the reinforcement have been performed for three different loads and for three different center distance between the columns, to identify the configuration that best ensure admissible settlements. Considering a load of 20 kPa and a center distance of the columns of 1 m, they are significantly reduced: they swing between a maximum of 0.050 m (red area, page 136) to a minimum of 0.009 m (blue area, page 138), while for a load of 30 kPa and for the same center distance they swing between a maximum of 0.082 m (red area) to a minimum of 0.014 m (blue area). These values are more admissible than before the reinforcement, even if in some cases a settlement smaller than 5 cm would be preferable. In fact, if the pipeline that has to be layed is particularly breakable, settlements lower than 1 cm are necessary. For this reason, the application of pre-loads just after the installation of the columns would dissipate the settlements in a very short time, allowing the normal and safe laying of the pipeline. The application of the pre-loads is showed from page 163 to 169; it is quite necessary for the red area, while for the blue it is not, given that the settlements just after the application of the columns are lower than 1 cm.
Another important improvement given by the columns is the rate of the settlements, which is strongly reduced: from 48 years to 550 days for the red area (page 137), from 26 years to 450 days for the yellow area point 16B46 (page 140), which are the two worst areas. This improvement underlines the behavior of the columns, that act also as a drain.

Regarding the slope stability after the reinforcement, the method adopted is sort of reverse, because it starts from the center distance between the columns to reach a specific value of shear strength that ensure a safety factor of 1.3. The method is a trial and error type, that is based on the fundamental equation 1.2.2. When the desired safety factor is reached with a specific value of a and a specific shear strength, we also know the specific center distance between the columns. This method implicates that the columns improve proportionally the different values of shear strength (as it is in reality), and not that the columns homogenize all the soil.
So, excluding all the results for the slope configuration of 1:2 which shows very big columns distance ($11 - 16$ m) which are usually not realistic, the

distance between the slab of columns swings between 4.96 m and 11.6 m for the excavation of 3 m, while it swings between 2.73 m and 4.37m for the excavation of 4.5 m (see Tables at pages 161-162). It can be noticed that the excavation of 4.5 m gives more normal and ordinary values of the slab distance.
Another important consideration regards the shear strength and the consequently slab distance necessary to obtain the safety factor of 1.3 in the case of the green area with the excavation of 4.5 m and slope configuration of 2:1 and 1:1. In this case, notwithstanding the green area is the less critical (less clay and higher shear strength), the necessary shear strength is very high and the slab distance is very low, even lower than the red area. The reason of this is the shape of the slip surface (page 154), which is flatten for geometrical reasons, in particular because the clay between the excavation and the underlying granular soil has a thickness of only 0.5 m, and the slip surface is obliged to pass through this point.

In conclusion, considering both the settlements calculations and the slope stability analysis it is possible to notice that a soil reinforcement is necessary. The chosen methos is the Lime-Cement columns, which as previously discussed is a very safe and effective method if compared to many other, and even from an economical point of view it is very competitive. The geometrical configuration of the columns is showed at page 159; considering the results, the distance c_1 that would be the best for improving the settlements along Dalvägen road is 1 m, but with the application of pre-loads if the pipeline is particularly breakable. The distance c_3 is fixed and is 0.5 m, while the distance c_2 that would be the best for improving the slope stability is different depending on the depth of the excavation and on the slope configuration. It could be adopted the lowest, to ensure the slope stability in every area, or alternatively it could be adopted a different value for each area, modifying in this way the total price of the columns installation.

BIBLIOGRAPHY

AHNBERG H.(2006). “Strength of stabilised soils, a laboratory study on clays and organic soils stabilised with different types of binder,” Swedish Deep Stabilization Research Centre (2006), PhD Thesis

AXELSSON M., REHNMAN S.E. (1999). “Field methods for quality control at the dry jet mixing method,” Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.303-310.

BROMS B.B. (2004). “Lime and lime/cement columns,” Ground improvement, 2nd edition, M.P.Moseley, K. Kirsch - Spon Press. pp. 252-330.

BROMS B.B. (1999). “Keynote lecture: Design of lime, lime/cement and cement column” Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.125-153.

BRUCE D. (2000). “An introduction to the deep soil mixing methods as used in geotechnical application,” Federal HighWay Administration (FHWA). FHWA-RD-99-138

BRUCE D.A., BRUCE M.E.C., DIMILLIO A. (1998). “Deep mixing methods: a global perspective,” GeoCongress 1998: Soil improvement for big digs, ASCE Geotechnical special publication No.81, Boston, MA, pp. 1-26.

CARLSTEN P. (1989). " Manual till Limeset," SIG varia 248, Swedish geotechnical institute, Linköping.

CONNOLLY H.(1997). "World wide web pages for slope design," MEng final year project report, School of engineering, 1997, University of Durham, pp.43.

ESRIG M.I. (1999). "Keynote lecture: Properties of binders and stabilized soils," Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.67-72.

FUNK J.R., MESRI G. (2013). "Settlement of the Kansai international airport islands," Journal of geotechnical and geoenviromental engineering 141, American society of civil engineers.

HANSBO S., MASSARSCH K.R. (2005). "Standardization of Deep Mixing Methods," Proceedings of the International Conference on Deep Mixing, Stockholm 2005. Vol. 1, pp.3-10.

HAYASHI H., NISHIKAVA J. (1999). "Mixing efficiency Mixing efficiency of dry jet mixing methods applies to peaty soft ground," Dry Mix methods for Deep soil stabilization, Brendenberg, Holms, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp. 333 – 338.

HOLM G. (1992). "Keynote paper: Sweden, Denmark, Norway and Finland," Piling, European practice and worldwide trends – Proceedings of a conference organized by the institution of civil engineers, and held in London on 7-9 April 1992, pp. 25-31.

HOLM G. (1999). "Keynote lecture: applications of Dry Mix Methods for deep soil stabilization," Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.3-13.

HOLM G., RUIN M., HÅKANSSON S. (1999). "Column penetration tests and extraction of lime/cement columns ," Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.3-13.

JACOBSON J.R., FILZ G.M., MITCHELL J.K. (2005). "Factors affecting strength of lime-cement columns based on a laboratory study of three organic soils," Proceedings of deep mixing 2005, p.87-94.

JANZ M., JOHANSSON S.E. (2002). "The function of different binding agents in deep stabilization," Swedish deep stabilization research centre, report 9.

KARLSSON R., HANSBO S., SGF(1989). "Soil classification and identification,"

KIVELÖ M. (1998). "Stabilization of embankments on soft soil with lime/cement columns," PhD Thesis, Royal Institute of Technology, Stockholm.

KITAZUME M., TERASHI M. (2013). "The deep mixing method," CRC Press, Taylor and Francis group.

LANG S.P., LIAO L.H., CHEN L.G. (1999). "A new application example of DJM technology in China.," Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.33-36.

LARSSON R., SÄLLFORS G. (1986). "Automatic continuous consolidation testing in Sweden," Consolidation of soils, testing and evaluation, ASTM, pp. 299-328.

LARSSON S. (2005). "State of practice report: execution, monitoring and quality control," Proceeding of the International Conference on Deep Mixing – Best practice and recent advances. Vol. 2, pp. 732-785.

LARSSON S., DAHLSTRÖM M., NILSSON B. (2005). "Uniformity of lime-cement columns for deep mixing: a field study," Ground improvement Volume 9 Issue 1: pp.1-15.

NAVIN M., FILZ G. (2005). "Statistical analysis of strength data from ground improved with DMM columns," Proceedings of the International Conference On Deep Mixing – Best Practice and Recent Advances, vol. 1. Swedish Deep Stabilization Research Centre, Stockholm (Sweden), pp. 145–154.

NOVAPOINT GEOSUITE TOOLBOX - GS SETTLEMENT (2013). Help version 2.0, Vianova GeoSuite AB, Solna, Sweden.

NOVAPOINT GEOSUITE TOOLBOX – GS STABILITY (2015). Help version 5.0, Vianova GeoSuite AB, Solna, Sweden.

OSTERMAN J. (1963). "Studies on the properties and formation of quick clays," Clays and clay minerals v.12, pp. 87-108.

PORBAHA A., SHIMA M., MIURA H., ISHIKURA K. (1999). "Dry jet mixing method for liquefaction remediation," Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.27-32.

RANKKA K., SKÖLD Y.A., HULTEN C., LARSSON R., LEROUX V., DAHLIN T.(2004). "Quick clay in Sweden," Swedish Geotechnical Institute, report 65.

SGF (Swedish Geotechnical Society), BGS (Society of Engineering Geology) (2001). "System of notations for geotechnical investigations," Version 2001:2.

SHENGHUA X., SONGYU L., GUANGYIN D., ZHIBIN L. (2012). "In-situ Test on Soft Marine Clays with High Clay Content Improved by Bidirectional Dry Jet Mixing Method," GeoCongress 2012 State of the Art and Practice in Geotechnical Engineering, p 988-995, 2012.

SWEDISH GEOTECHNICAL INSTITUTE (2007). "Användarmanual CONRAD version 3.0"

SWEDISH GEOTECHNICAL SOCIETY (2000). "Kalk och kalkcementpelare," SGF report 4:95.

TERASHI M. (1997). "Theme lecture: deep mixing method, brief state of the art," Proceedings of the 14th International Conference on soil mechanics and geotechnical engineering, vol. 4, Hamburg, Germany, pp. 2475-2478.

VIBERG L., ERIKSSON B., JOHANSSON S. (1999). "Large ground deformations during cement lime column installation on road E4 in northern Sweden," Dry Mix Methods for Deep Soil Stabilization, Bredenberg, Holm, Broms, 1999 Balkema, Rotterdam. The International Conference on Deep Mixing – Recent Advances and Best practice, Stockholm 1999, pp.275-281.

Websites

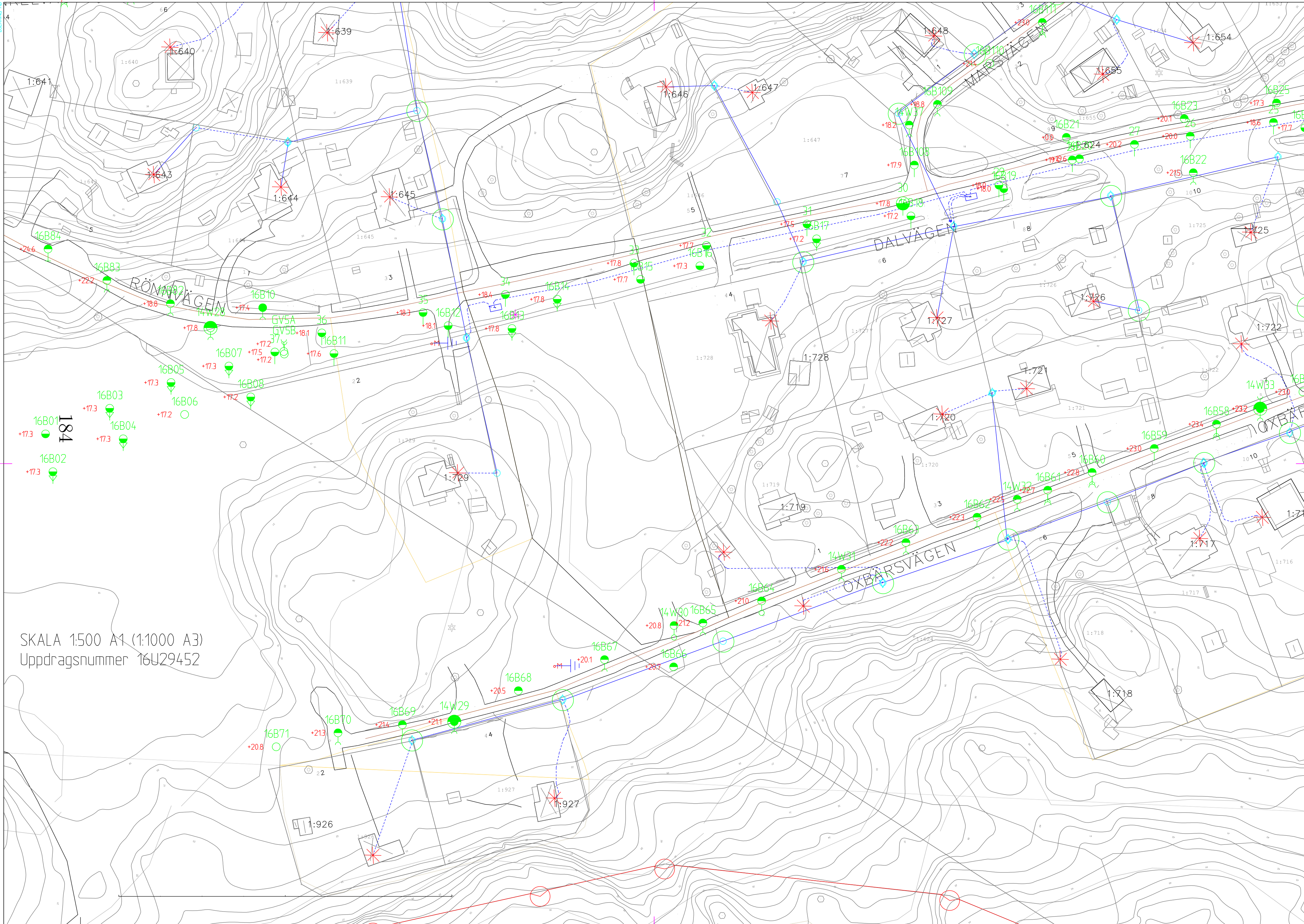
http://www.sgu.se/en/geology-of-sweden/
http://kartor.eniro.se/
http://www.raito.co.jp/
http://indiamart.com/
http://civcal.media.hku.hk/

Software

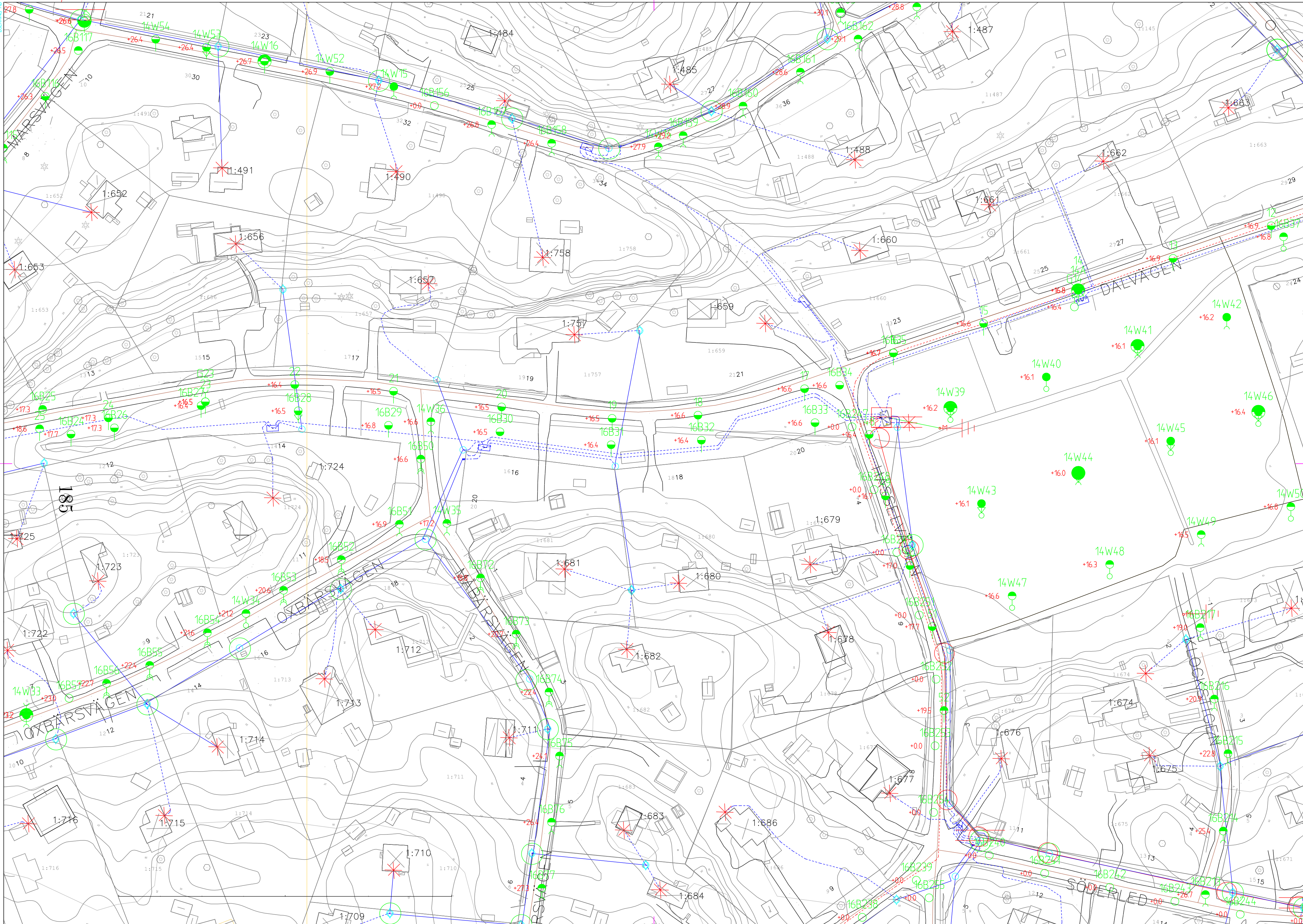
GeoSuite settlement, version 15.0.1.0
GeoSuite stability, version 15.0.1.0
Limeset, version 4.0, modified by Rolf Lingfors, 1993
Autodesk AutoCAD 2015
Conrad, version 3.1.1, SGI

ATTACHMENTS

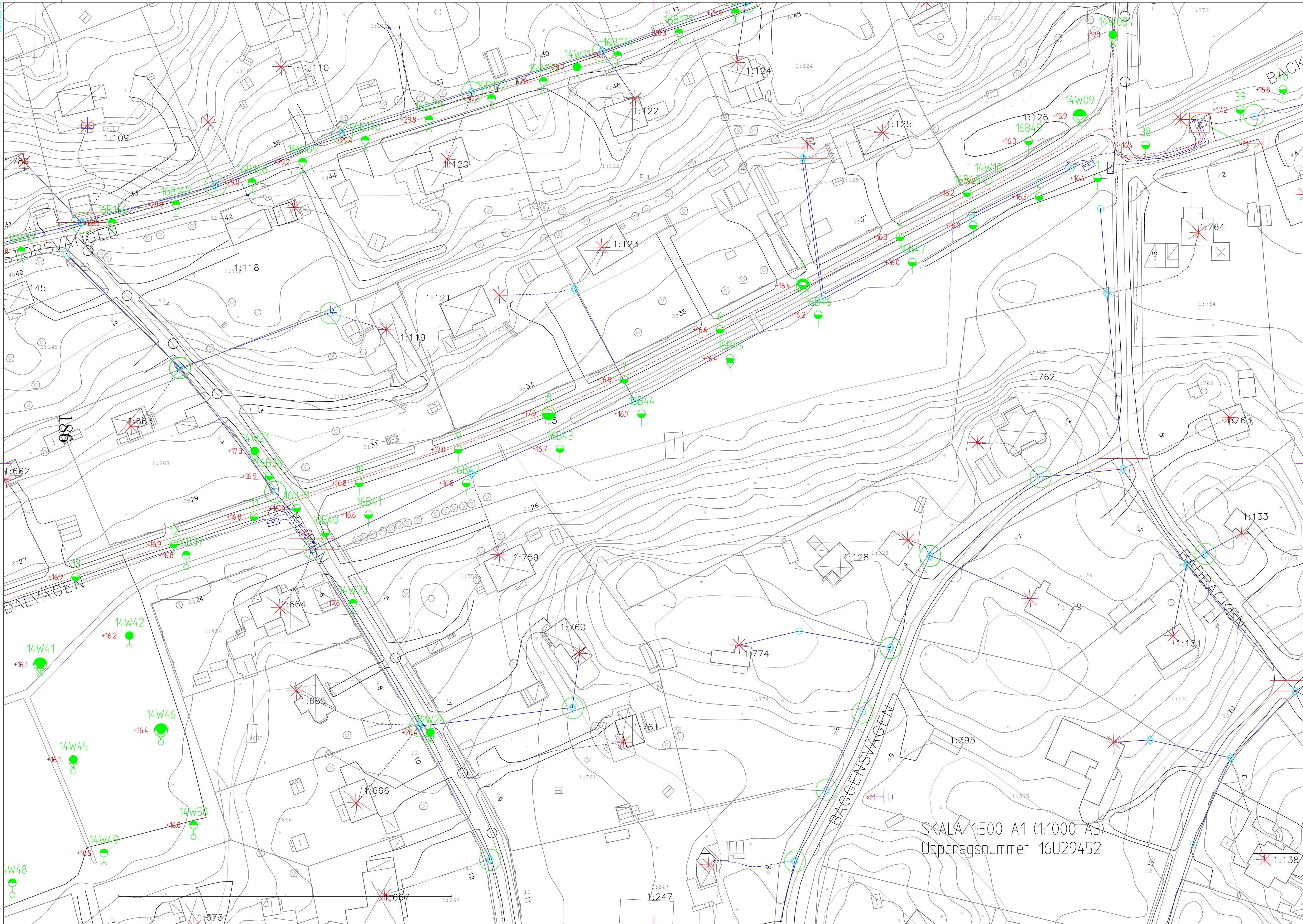
- Maps of Dalvägen road with the conducted surveys
- Sections of the different areas along Dalvägen
- CPT elaborations
- Lab results on some specimens
- CRS
- Settlements calculations before LCC with excel
- Settlements calculations before LCC with GS settlements
- Stability calculations before LCC with GS stability
- Settlements calculations after LCC with excel
- Settlements calculations after LCC with Limeset
- Stability calculations after LCC with GS stability


RÖNNVÄGEN
DALVÄGEN
MARSVÄGEN
OXBÄRSVÄGEN
184
SKALA 1:500 A1 (1:1000 A3)
Uppdragsnummer 16U29452

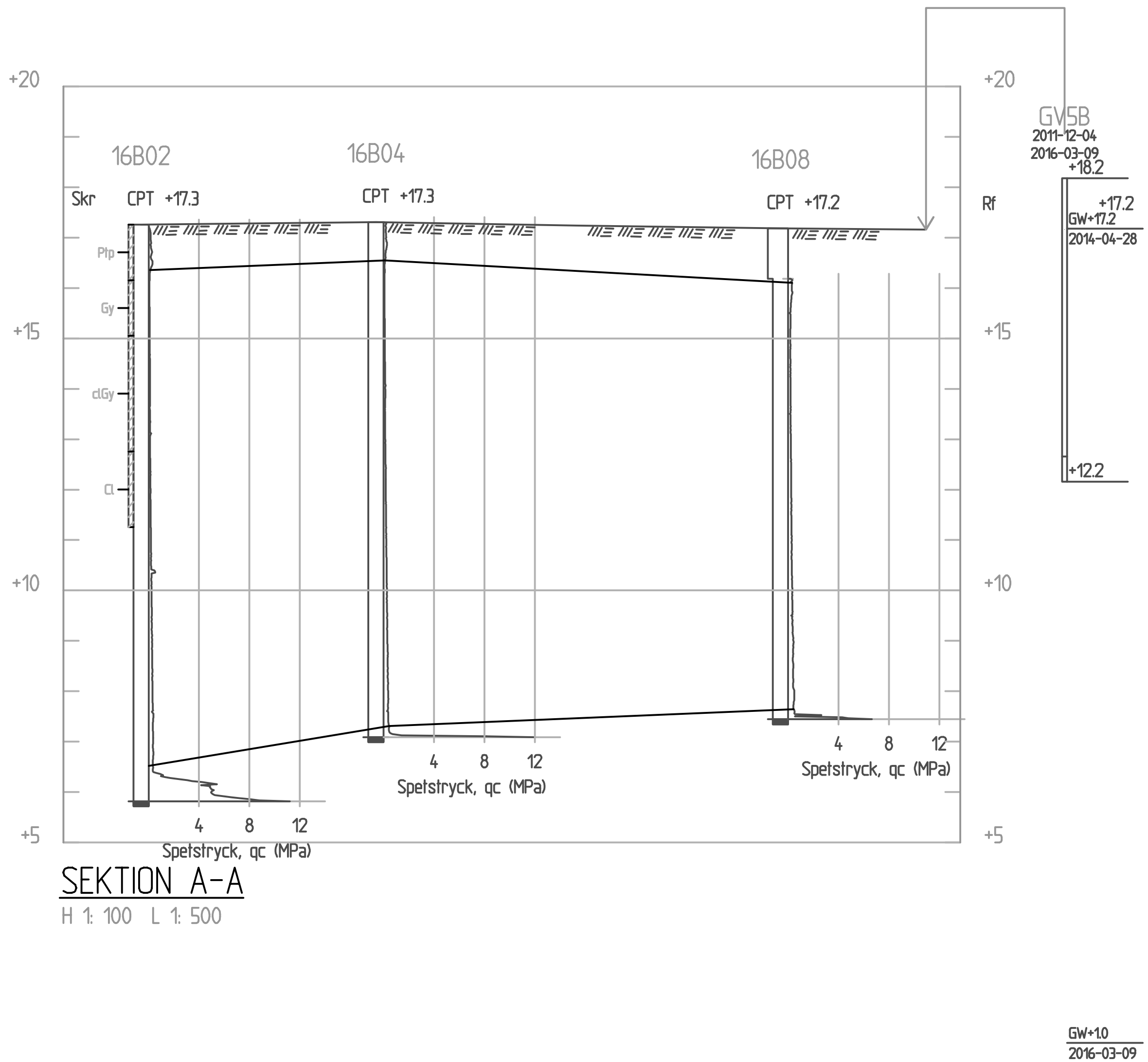
A1


MARSVÄGEN
OXBÄRSVÄGEN
DALVÄGEN
SÖDERLED
1:484
1:485
1:487
1:488
1:490
1:491
1:652
1:653
1:656
1:657
1:659
1:660
1:661
1:662
1:663
1:679
1:680
1:681
1:682
1:683
1:684
1:686
1:709
1:710
1:712
1:713
1:714
1:715
1:716
1:722
1:723
1:724
1:725
1:757
1:758
185

A1


STORSVÄNGEN
DALVÄGEN
BAGGENSVÄGEN
BADBACKEN
BACK
SKALA 1:500 A1 (1:1000 A3)
Uppdragsnummer 16U29452

A1



SEKTION A-A

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

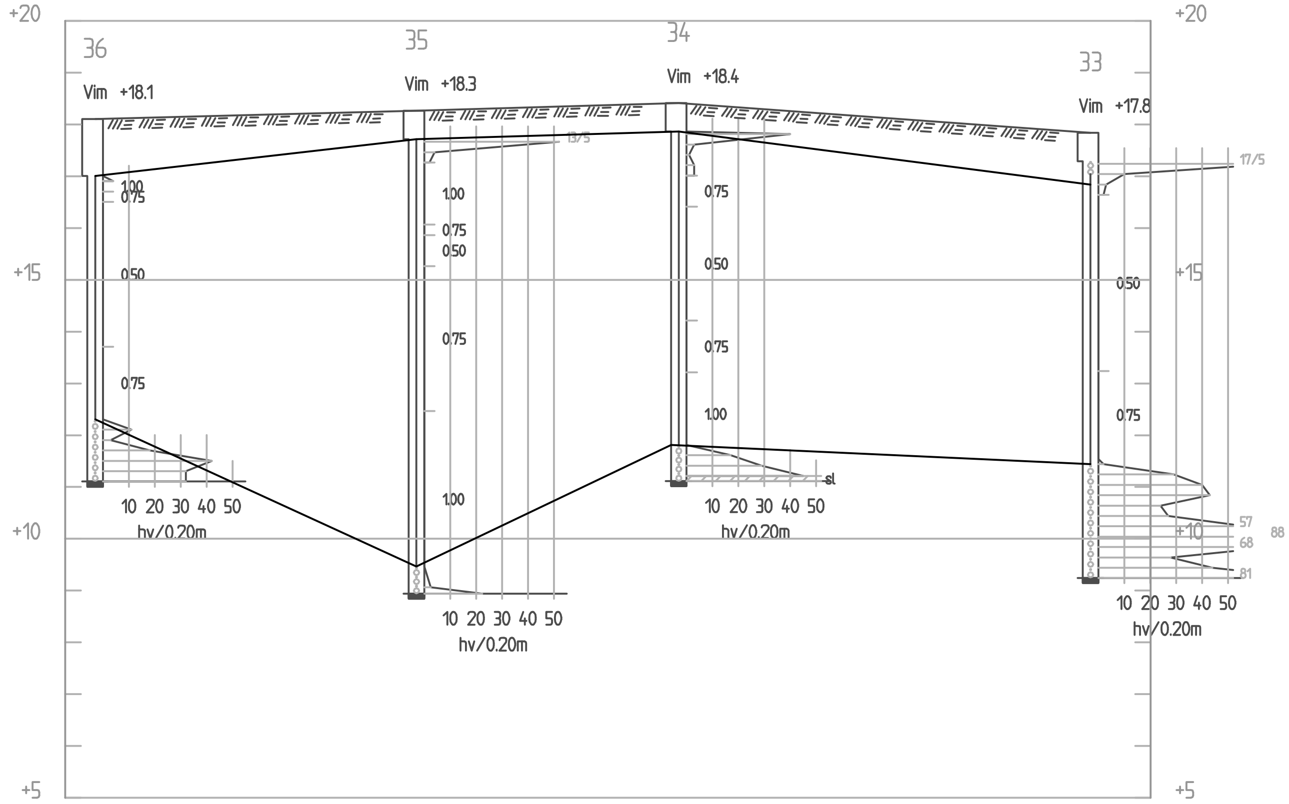
Dalvägen, 16U29452

HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER | ÄNDR

188



SEKTION B-B

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

Dalvägen, 16U29452

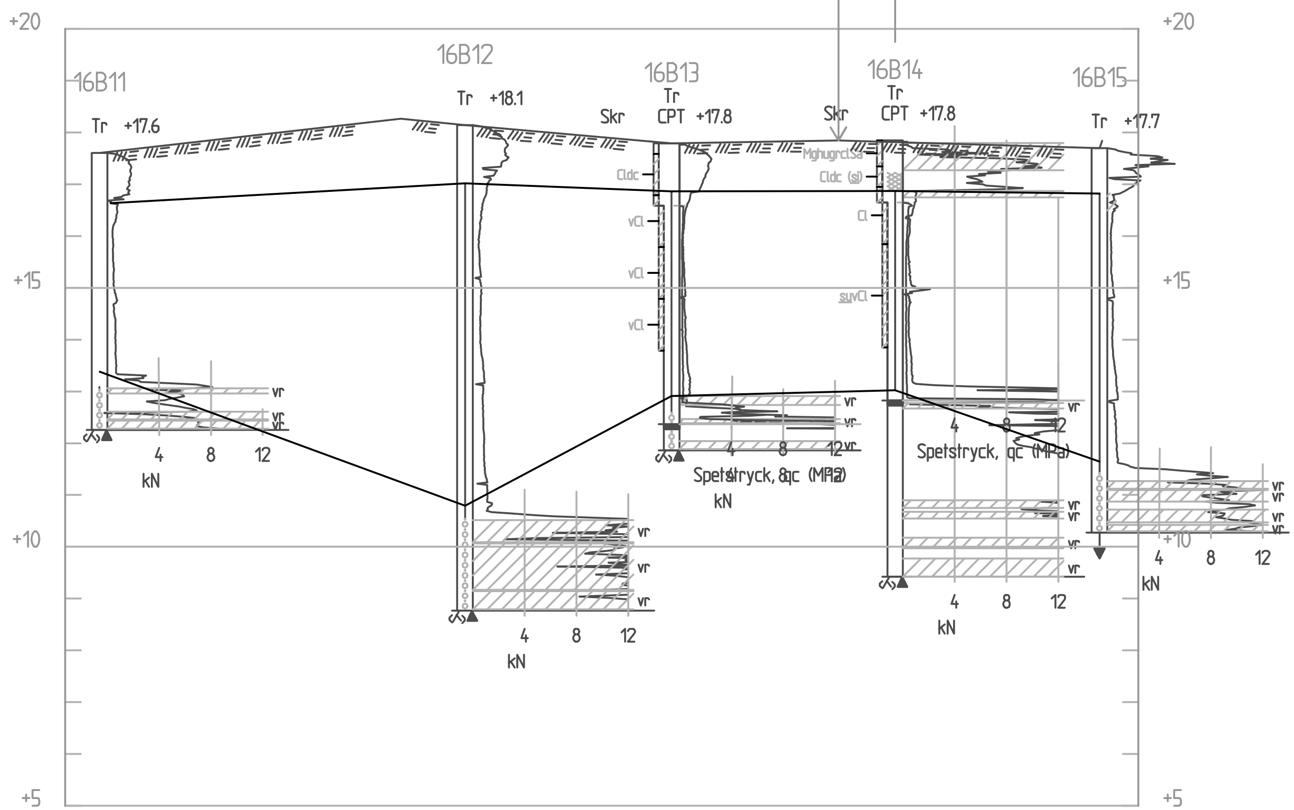
HANDLÄGGARE

RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER

ÄNDR



SEKTION B-Bn

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE

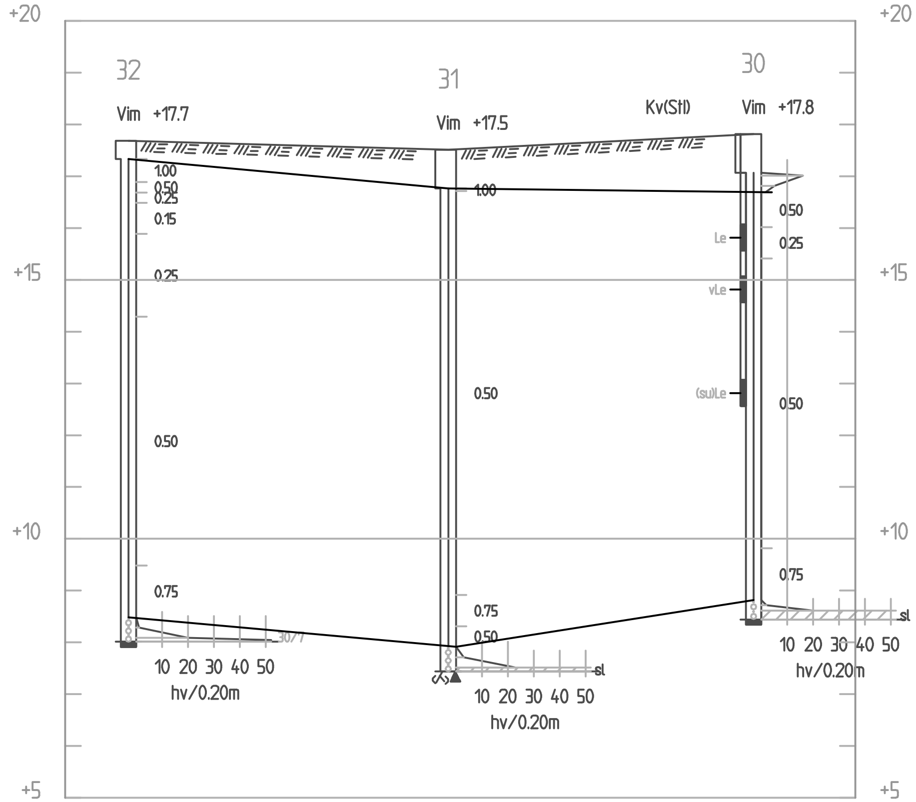
RITAD AV

Sektionsritning

SKALA L 1:500, H 1:100

RITNINGSNUMMER

ÄNDR



SEKTION C-C

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

Dalvägen, 16U29452

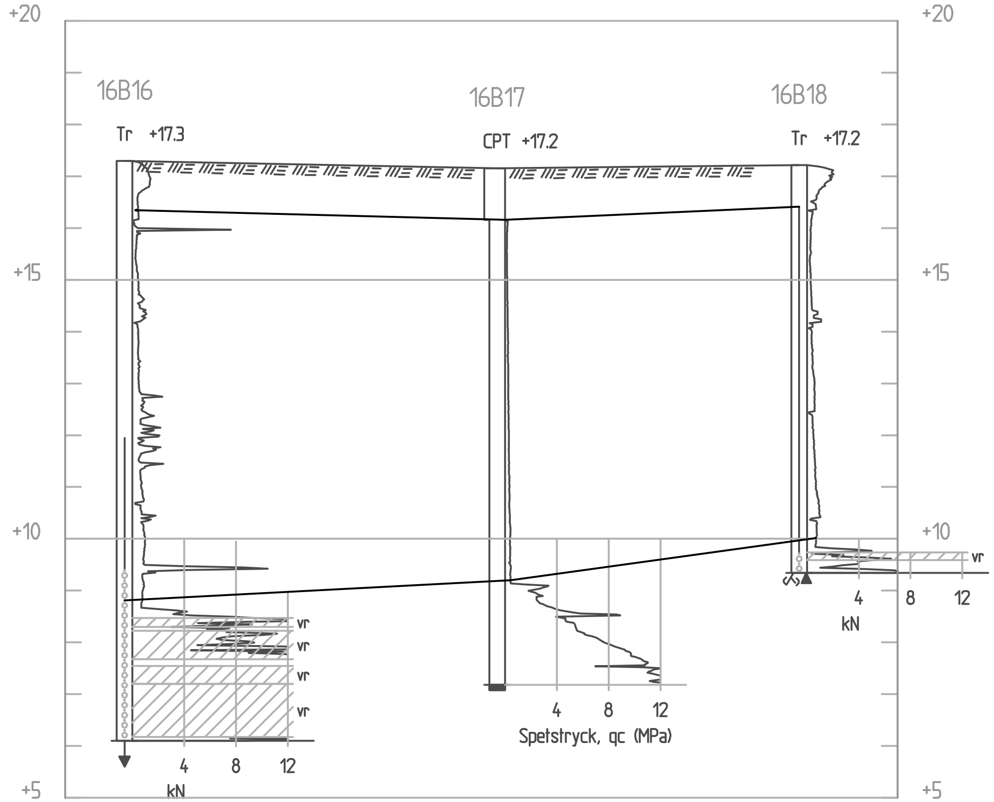
HANDLÄGGARE

RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER

ÄNDR



SEKTION C-Cn

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

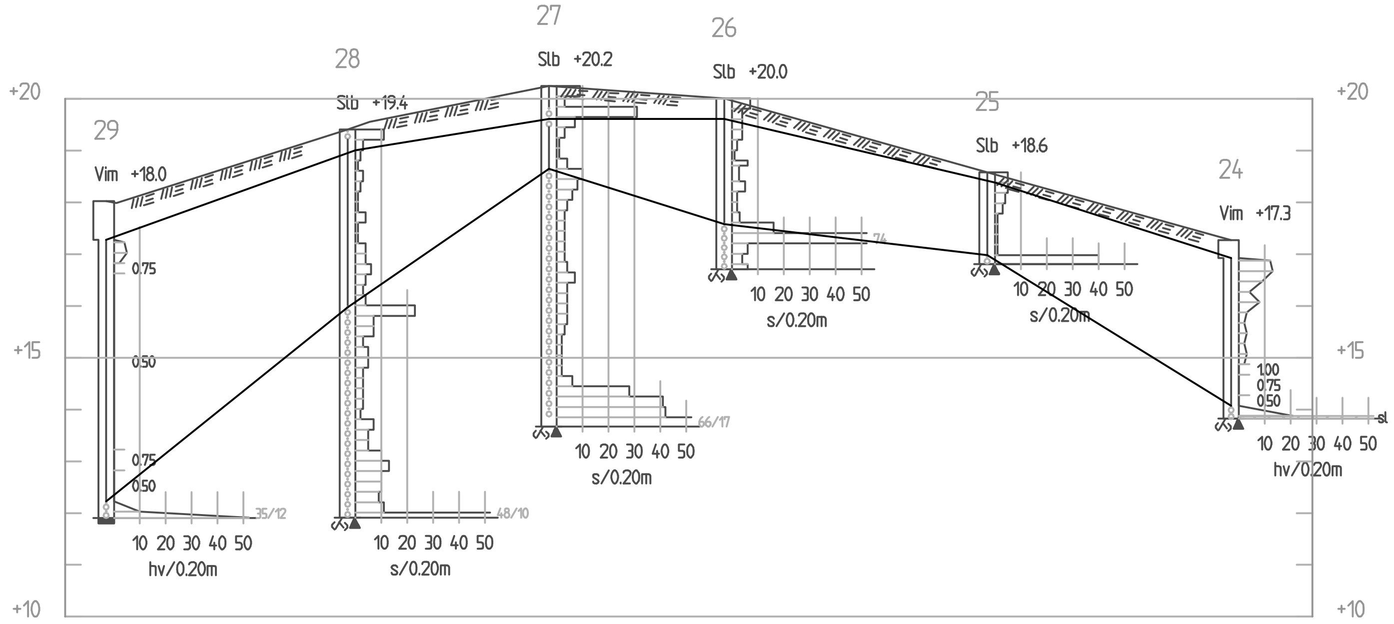
AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE	RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER	ÄNDR



SEKTION D-D
H 1: 100 L 1: 500

192

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

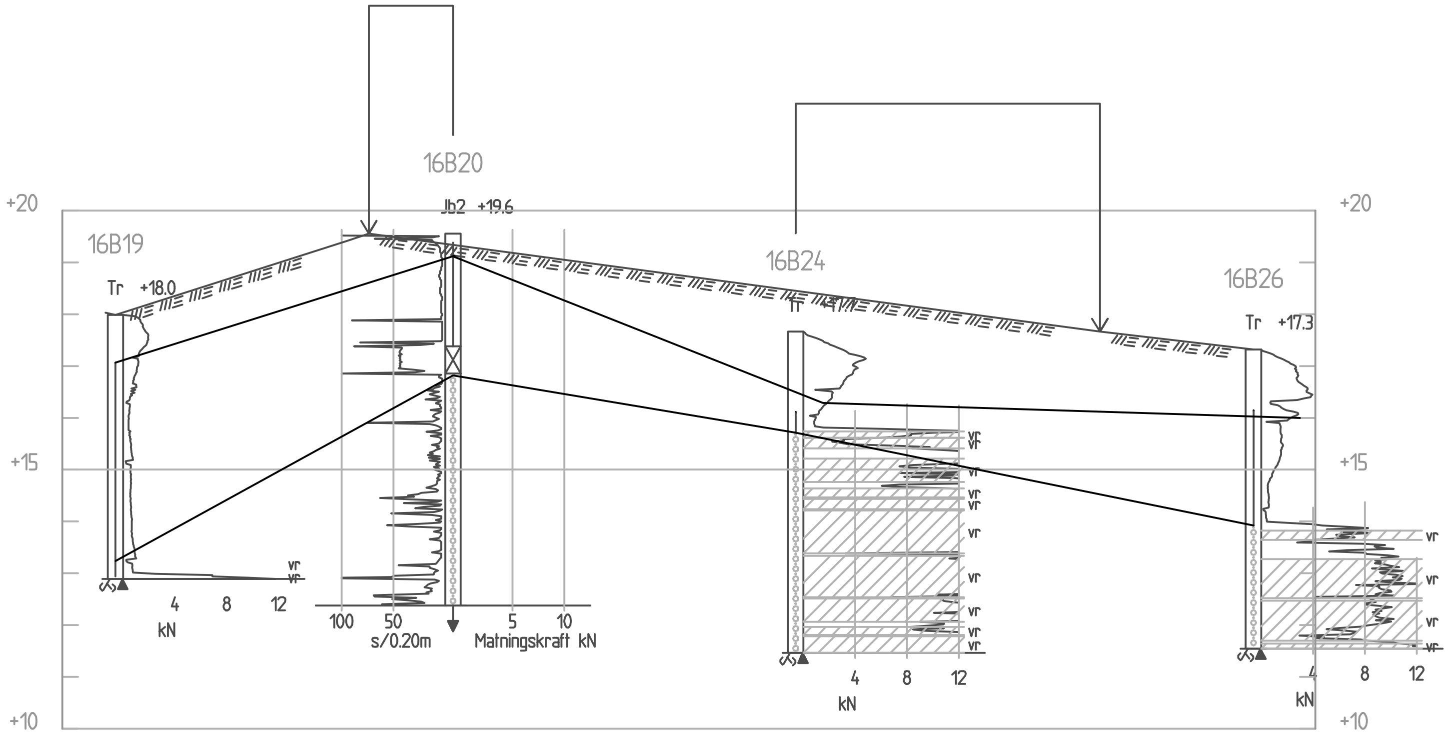
AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER | ÄNDR



SEKTION D-Dn

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

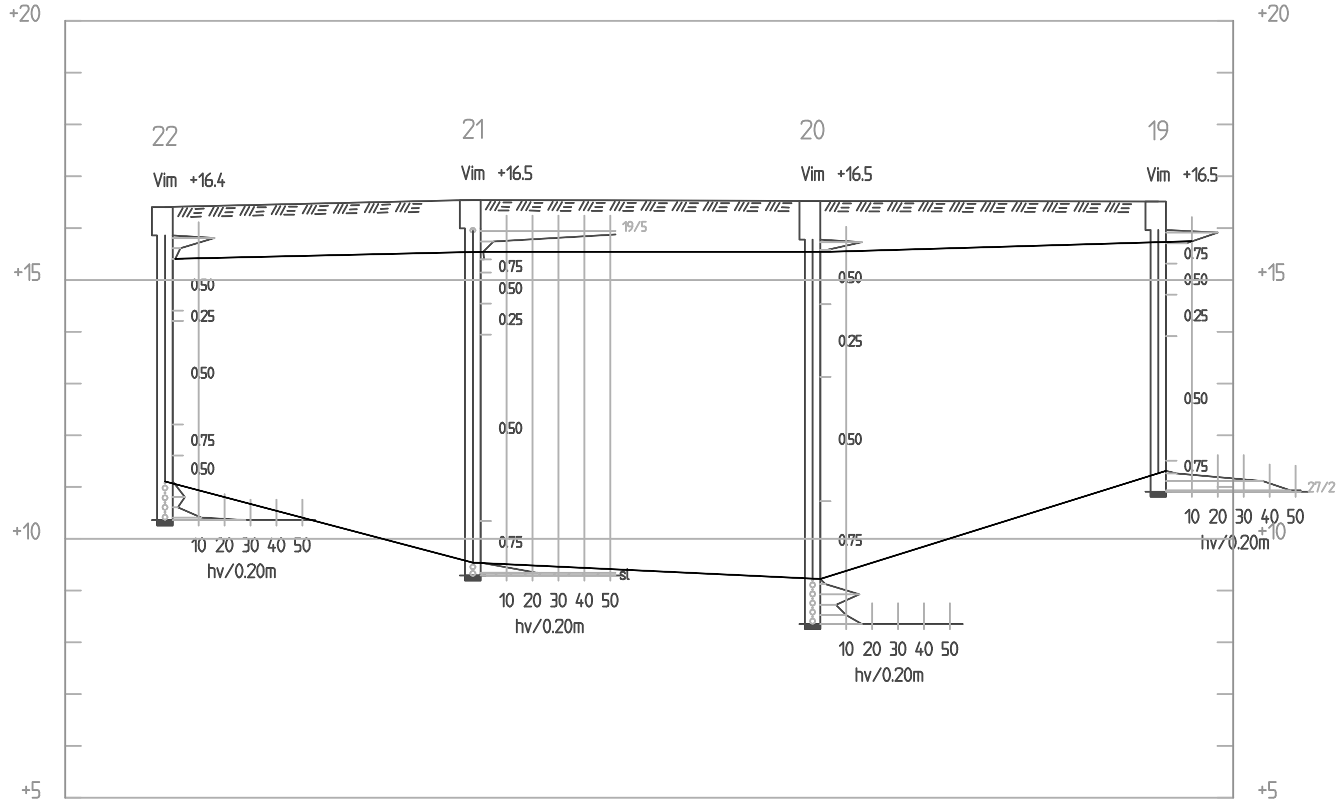
AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER | ÄNDR



SEKTION E-E

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE

RITAD AV

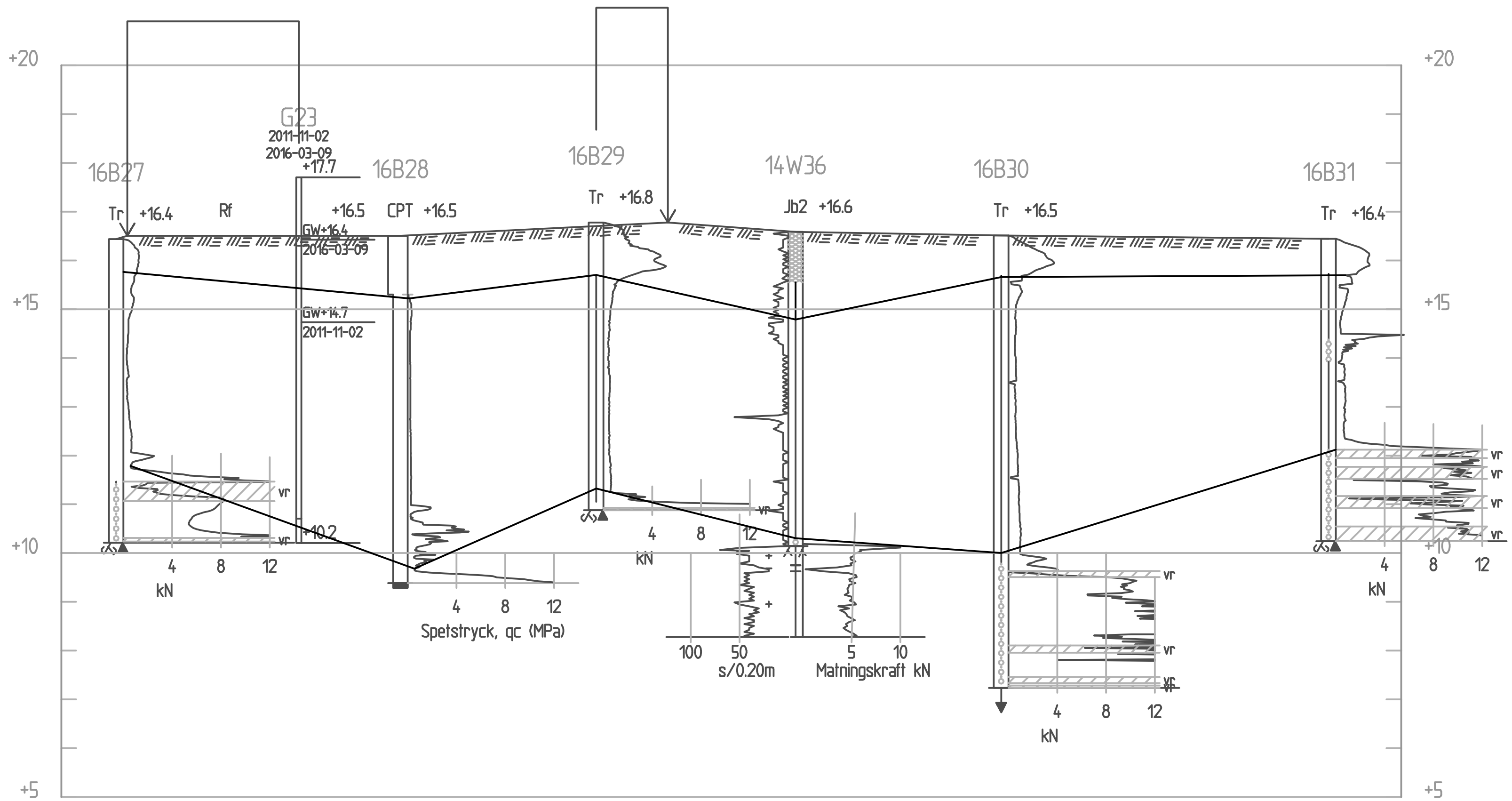
Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER E-E

ÄNDR

C:\Users\CGI\Desktop\Ny mapp\EE.dwg, 2016-05-25 15:18:42, Bluebeam PDF svartvit.pc3

AutoGRAF



SEKTION E-En

H 1: 100 L 1: 500

195

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

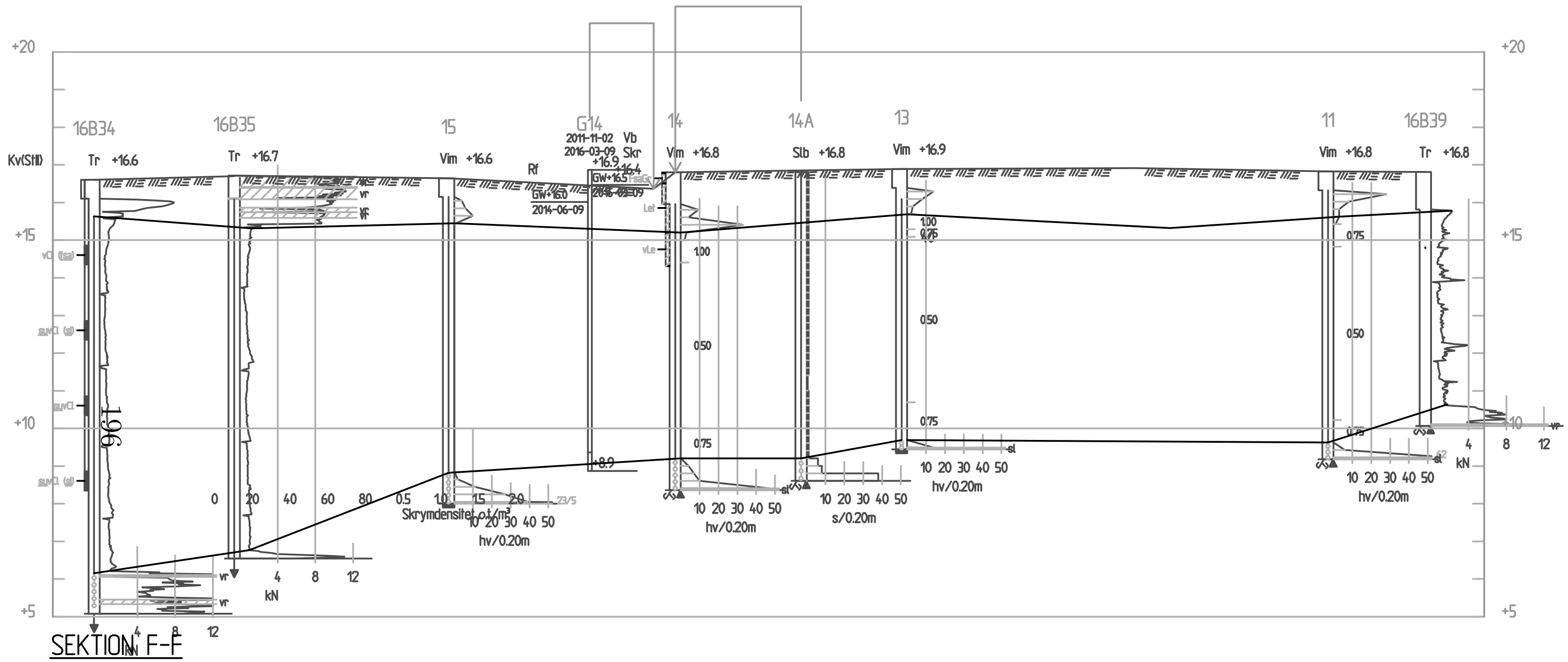
AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER | ÄNDR



SEKTION F-F

H 1: 100 L 1: 500

196

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

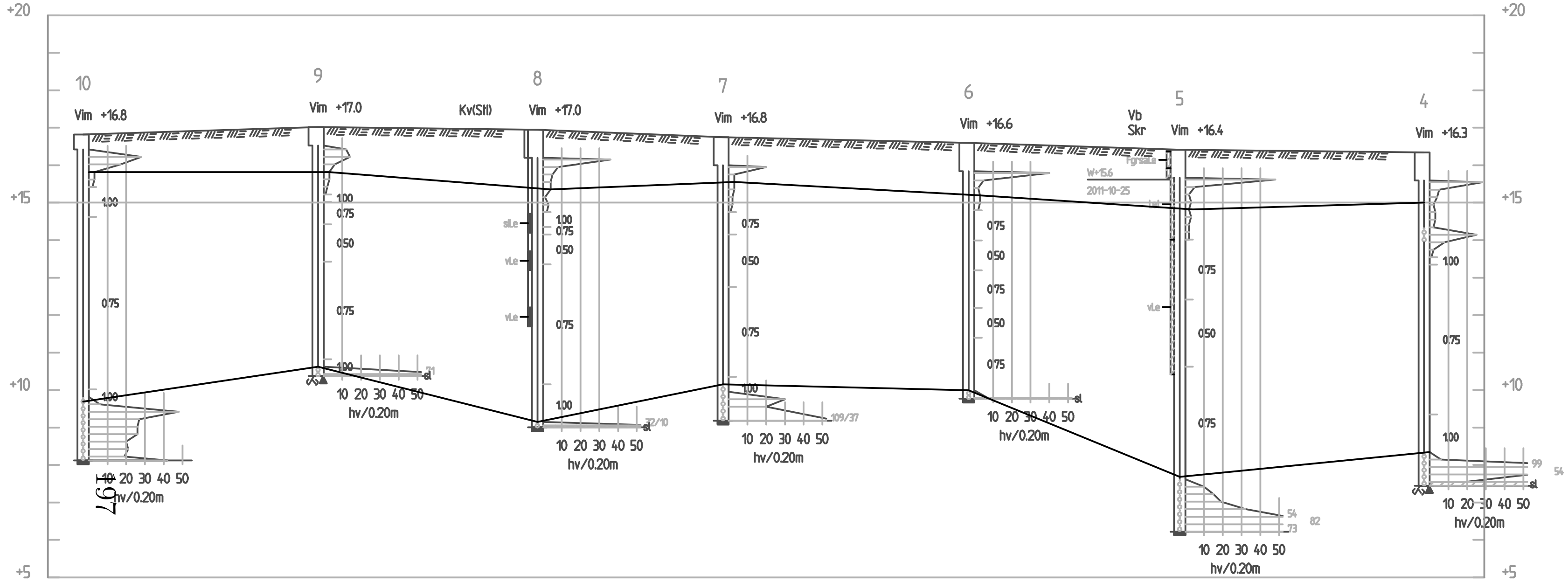
Dalvägen, 16U29452

HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER 0:0 | ÄNDR

C:\Users\CGI\Desktop\Ny mapp\FF.dwg, 2016-05-25 14:32:51, Bluebeam PDF svartvit.pc3



SEKTION G-G

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

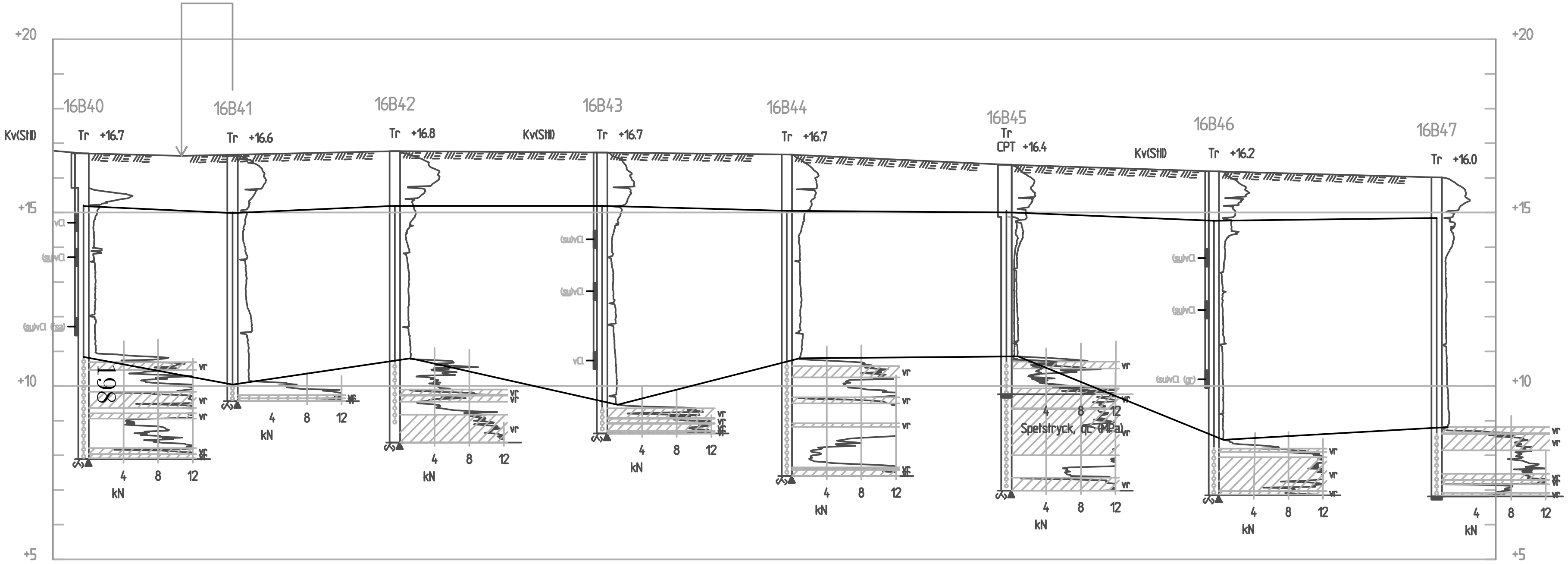
AutoGRAF

Dalvägen, 16U29452

HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER 0:0 | ÄNDR



SEKTION G-Gn

H 1: 100 L 1: 500

BET	ANT	ÄNDRINGEN AVSER	SIGN	DATUM

AutoGRAF

Dalvägen, 16U29452

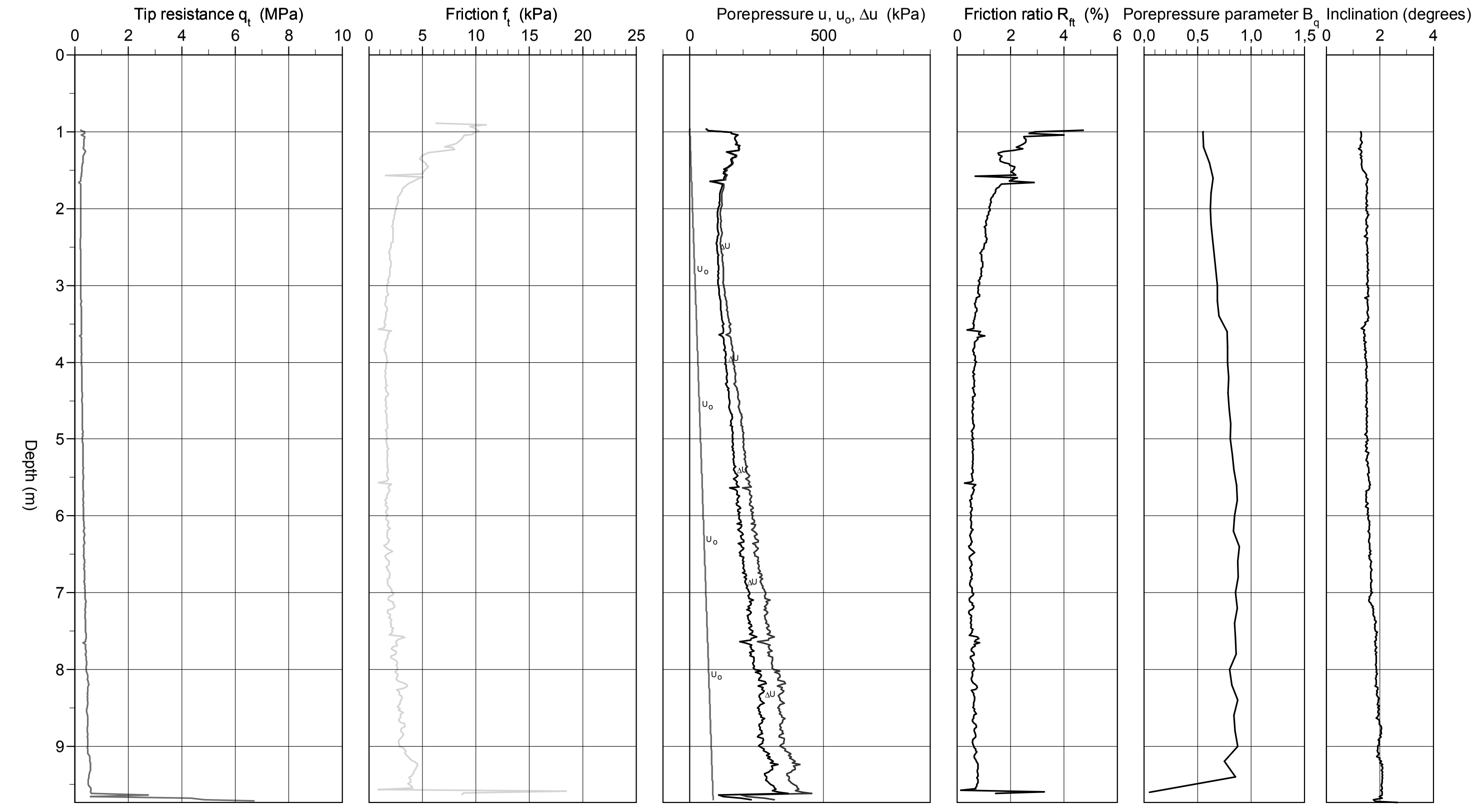
HANDLÄGGARE | RITAD AV

Sektionsritning SKALA L 1:500, H 1:100

RITNINGSNUMMER 0:0 | ÄNDR

CPT-test performed according to EN ISO 22476-1

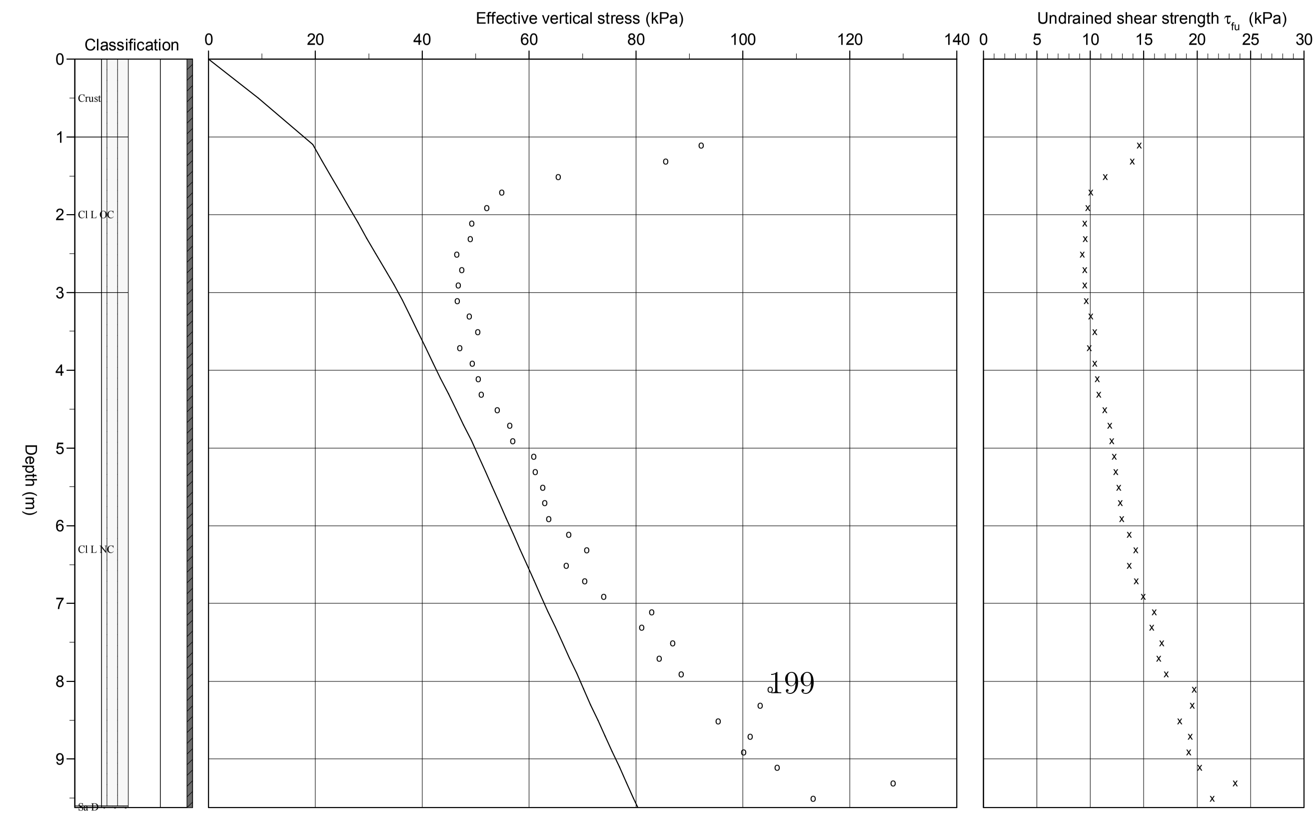
Predrilling depth	1,00 m	Reference		Fluid in filter		Project	Dalvägen
Start depth	1,00 m	Level at reference		Coordinats		Project nr	16U29452
Stop depth	9,74 m	Predrilled material		Equipment		Site	Dalvägen
Ground water level	1,00 m	Geometry	Normal	Cone nr	4603	Designation	16B08
						Date	2016-04-21



F:\Tesi\Dalvägen files\CPT\CPT 16B08.cpw
2016-06-09

CPT test evaluated accorrding to SGI Information 15 rev. 2007

Reference		Predrilling depth	1,00 m	Evaluator	Project	Dalvägen
Ground water level		Predrilled material		Evaluation date	Project nr	16U29452
Grundvattenyta	1,00 m	Equipment			Site	Dalvägen
Start depth	1,00 m	Geometry	Normal		Designation	16B08
					Date	2016-04-21

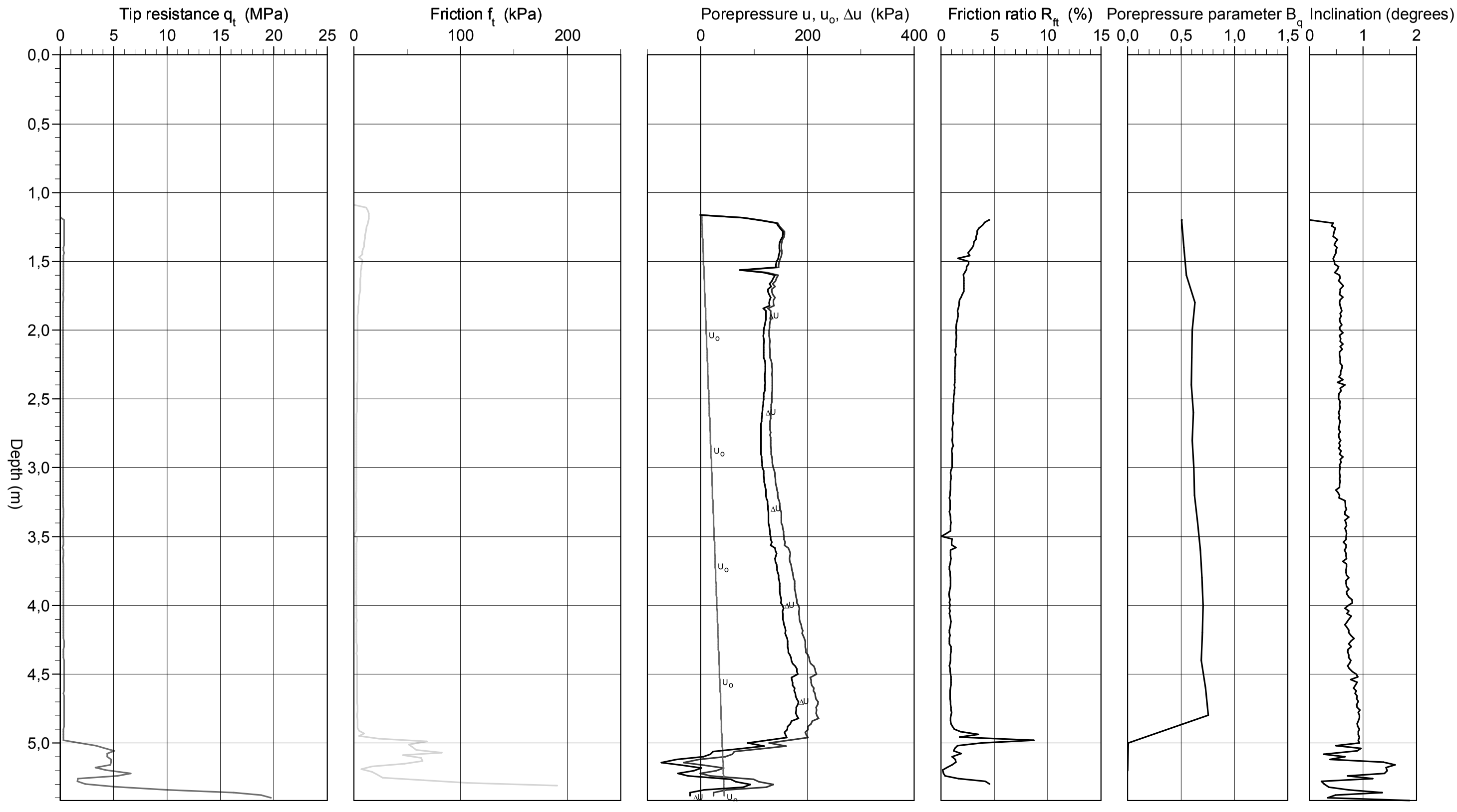


F:\Tesi\Dalvägen files\CPT\CPT 16B08.cpw 2016-06-09

CPT-test performed according to EN ISO 22476-1

Predrilling depth	1,20 m	Reference		Fluid in filter	
Start depth	1,20 m	Level at reference		Coordinats	
Stop depth	5,42 m	Predrilled material		Equipment	
Ground water level	1,00 m	Geometry	Normal	Cone nr	4603

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B13
Date	2016-04-05

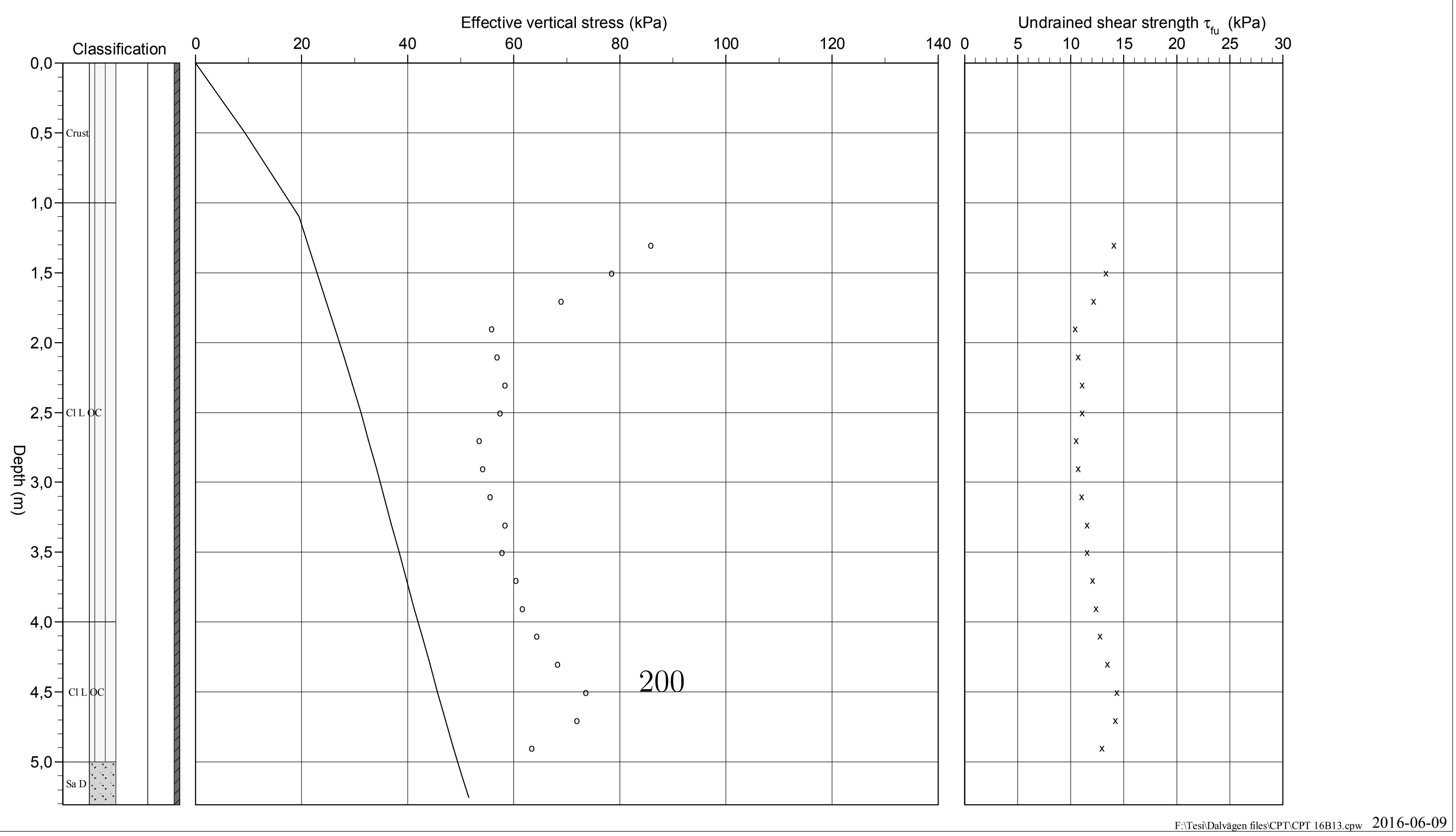


F:\Tesi\Dalvägen files\CPT\CPT 16B13.cpw
2016-06-09

CPT test evaluated accorrding to SGI Information 15 rev. 2007

Reference		Predrilling depth	1,20 m	Evaluator	
Ground water level		Predrilled material		Evaluation date	
Grundvattenyta	1,00 m	Equipment			
Start depth	1,20 m	Geometry	Normal		

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B13
Date	2016-04-05

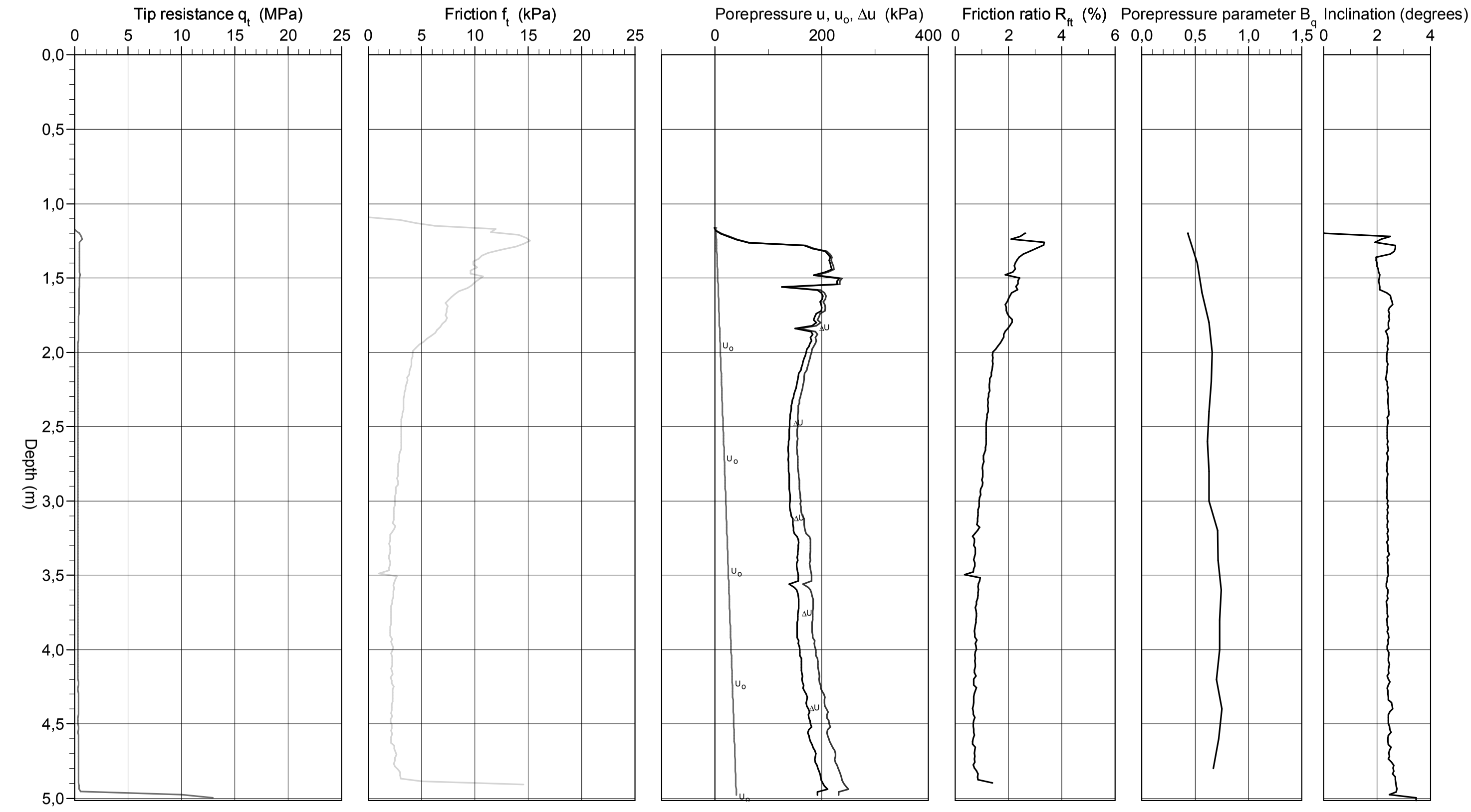


F:\Tesi\Dalvägen files\CPT\CPT 16B13.cpw 2016-06-09

CPT-test performed according to EN ISO 22476-1

Predrilling depth	1,20 m	Reference		Fluid in filter		
Start depth	1,20 m	Level at reference		Coordinats		
Stop depth	5,02 m	Predrilled material		Equipment		
Ground water level	1,00 m	Geometry	Normal	Cone nr	4603	

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B14
Date	2016-04-05

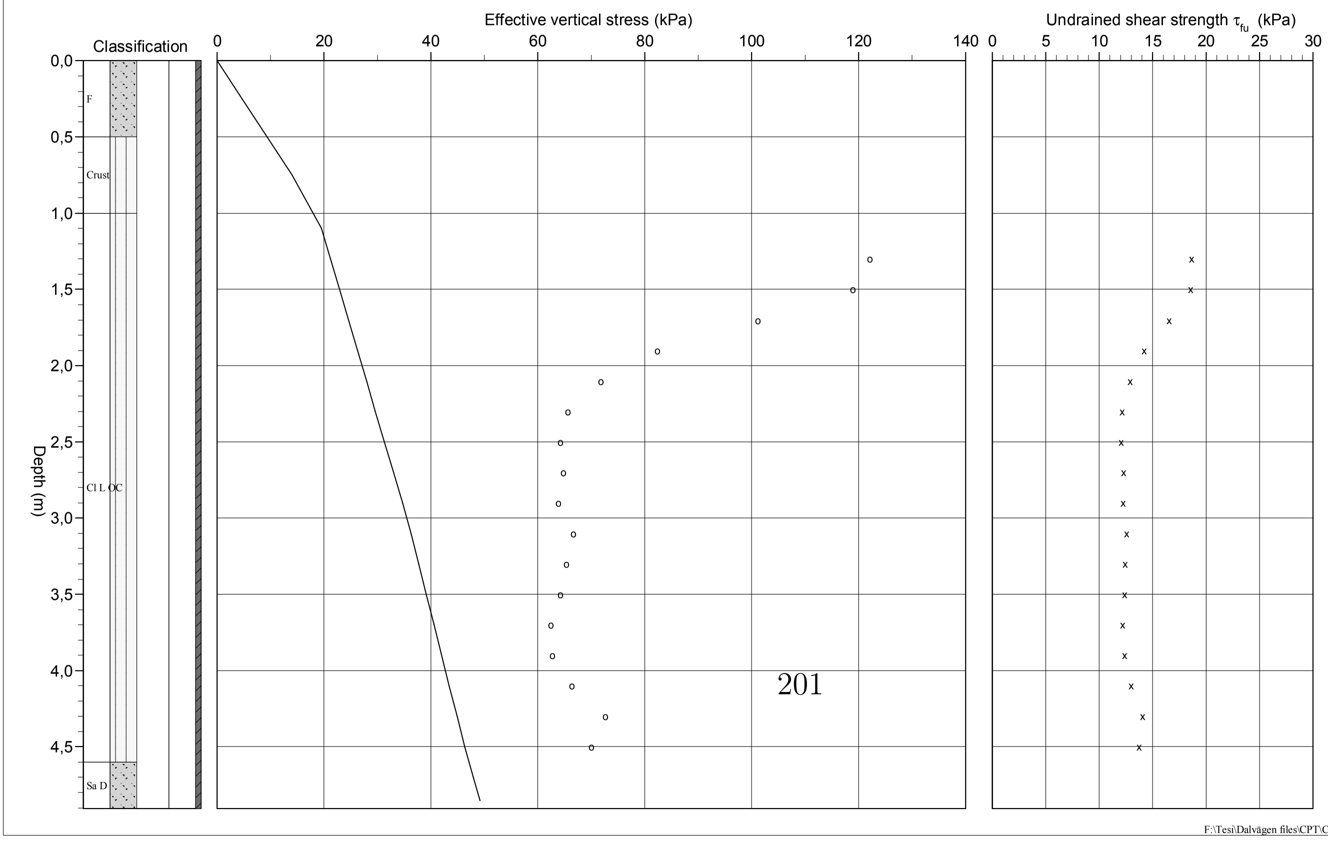


F:\Tesi\Dalvägen files\CPT\CPT 16B14.cpw
2016-06-09

CPT test evaluated accorrding to SGI Information 15 rev. 2007

Reference		Predrilling depth	1,20 m	Evaluator	
Ground water level		Predrilled material		Evaluation date	
Grundvattenyta	1,00 m	Equipment			
Start depth	1,20 m	Geometry	Normal		

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B14
Date	2016-04-05

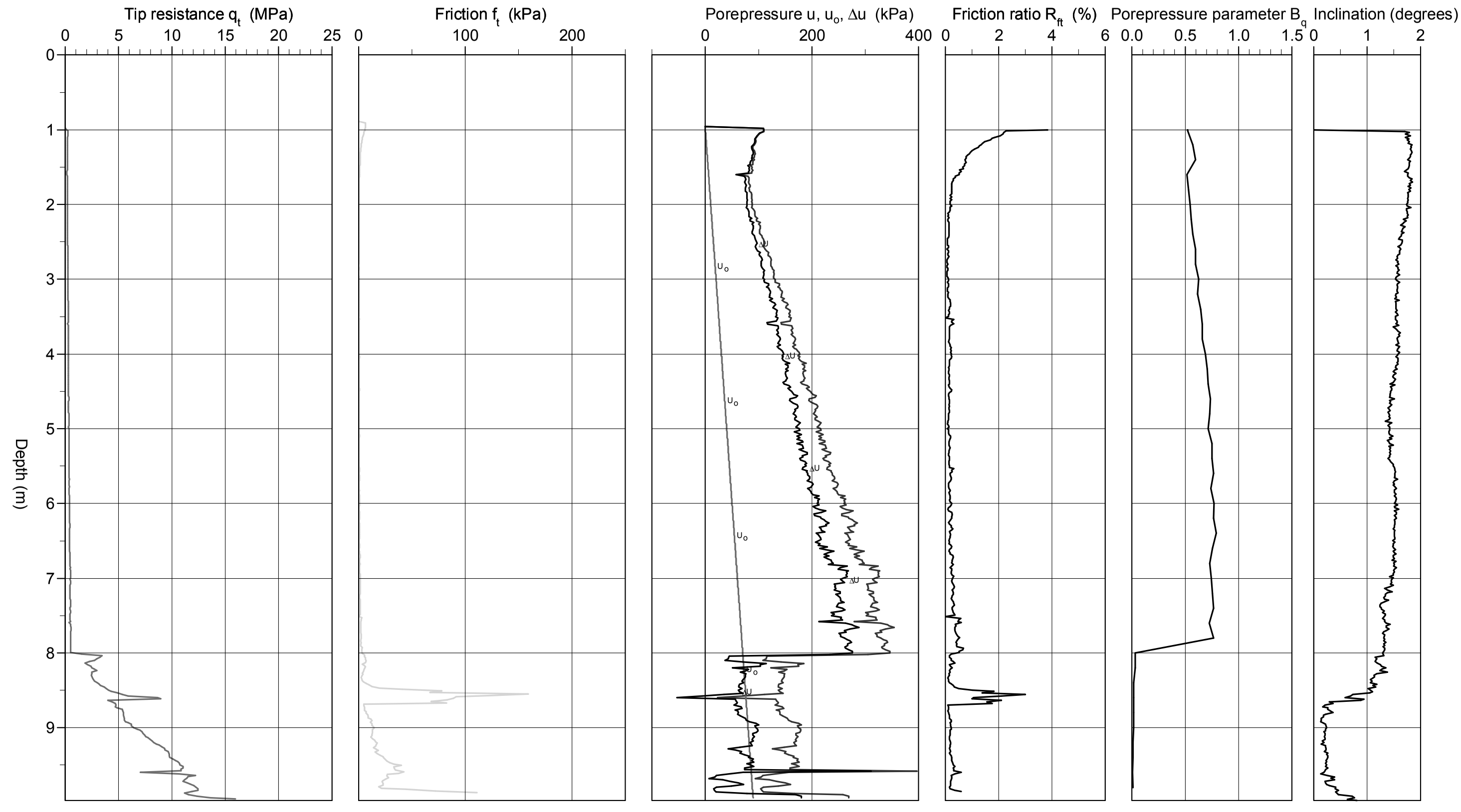


F:\Tesi\Dalvägen files\CPT\CPT 16B14.cpw 2016-06-09

CPT-test performed according to EN ISO 22476-1

Predrilling depth	1.00 m	Reference		Fluid in filter	
Start depth	1.00 m	Level at reference		Coordinats	
Stop depth	9.98 m	Predrilled material		Equipment	
Ground water level	1.00 m	Geometry	Normal	Cone nr	4603

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B17
Date	2016-04-12

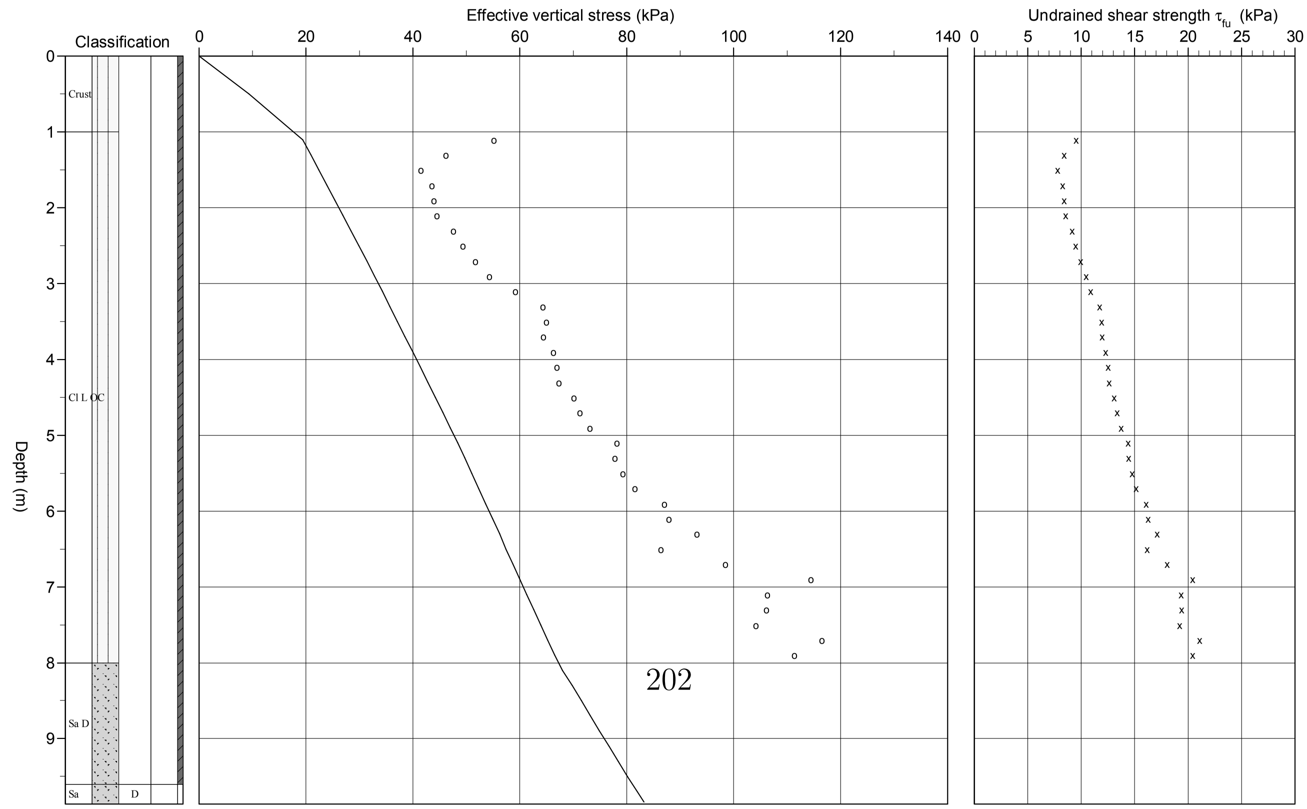


C:\Users\CGI\Desktop\CPT 16B17.cpw
2016-05-19

CPT test evaluated accorrding to SGI Information 15 rev. 2007

Reference		Predrilling depth	1.00 m	Evaluator
Ground water level		Predrilled material		Evaluation date
Grundvattenyta	1.00 m	Equipment		
Start depth	1.00 m	Geometry	Normal	

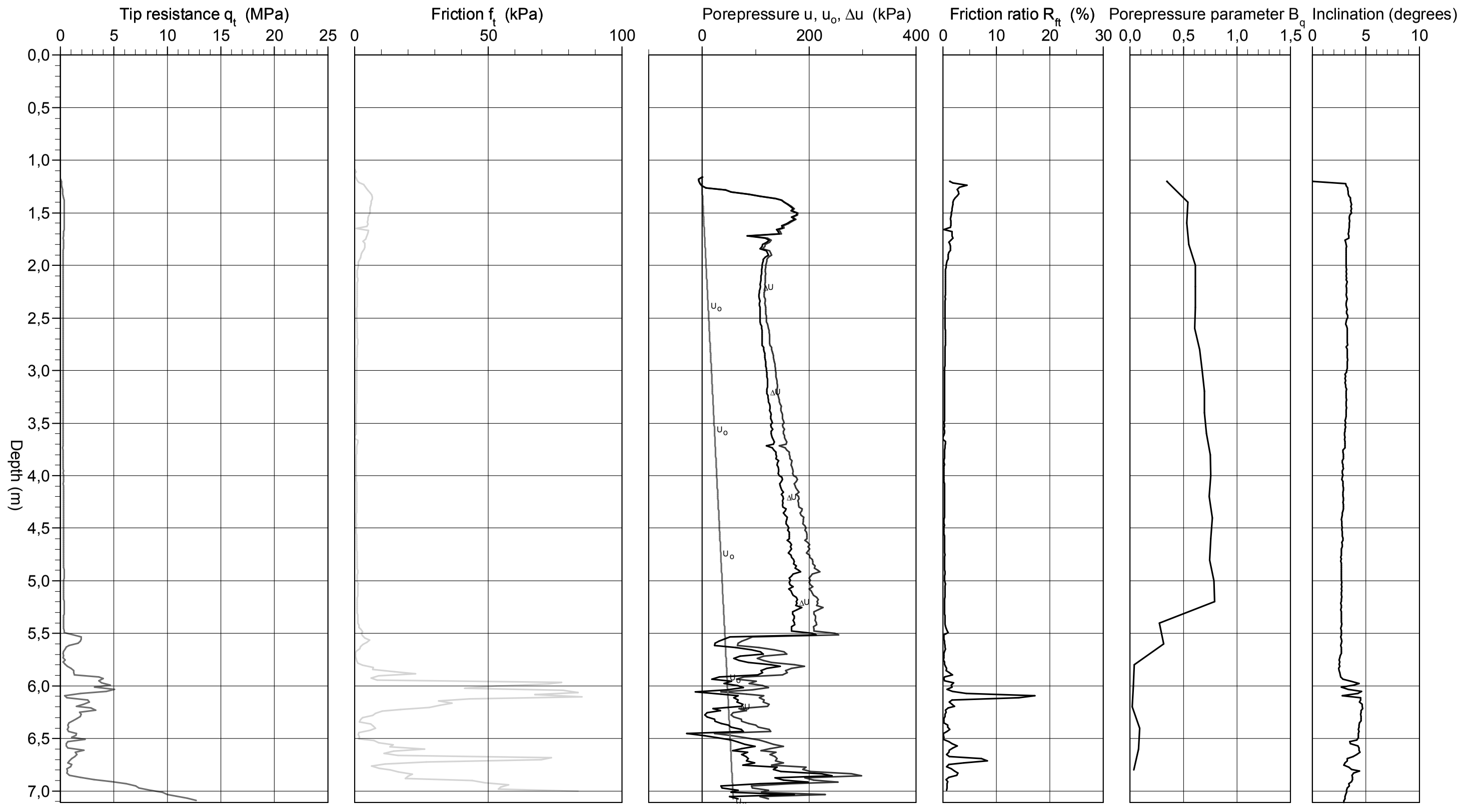
Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B17
Date	2016-04-12



C:\Users\CGI\Desktop\CPT 16B17.cpw 2016-05-19

CPT-test performed according to EN ISO 22476-1

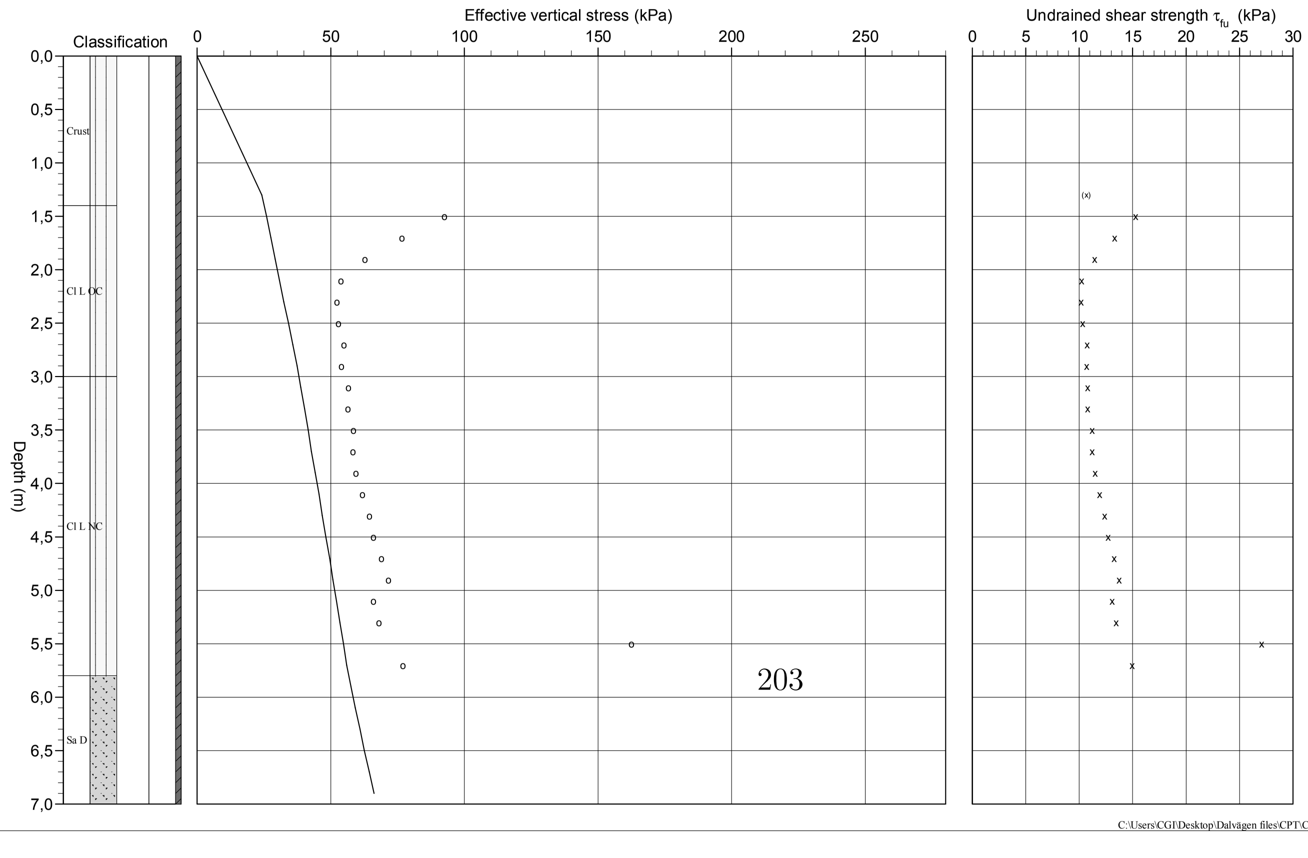
Predrilling depth	1,20 m	Reference		Fluid in filter		Project	Dalvägen
Start depth	1,20 m	Level at reference		Coordinats		Project nr	16U29452
Stop depth	7,12 m	Predrilled material		Equipment		Site	Dalvägen
Ground water level	1,30 m	Geometry	Normal	Cone nr	4603	Designation	16B28
						Date	2016-04-08



C:\Users\CGI\Desktop\Dalvägen files\CPT\CPT 16B28.cpw
2016-05-19

CPT test evaluated accorrding to SGI Information 15 rev. 2007

Reference		Predrilling depth	1,20 m	Evaluator		Project	Dalvägen
Ground water level		Predrilled material		Evaluation date		Project nr	16U29452
Grundvattenyta	1,30 m	Equipment				Site	Dalvägen
Start depth	1,20 m	Geometry	Normal			Designation	16B28
						Date	2016-04-08

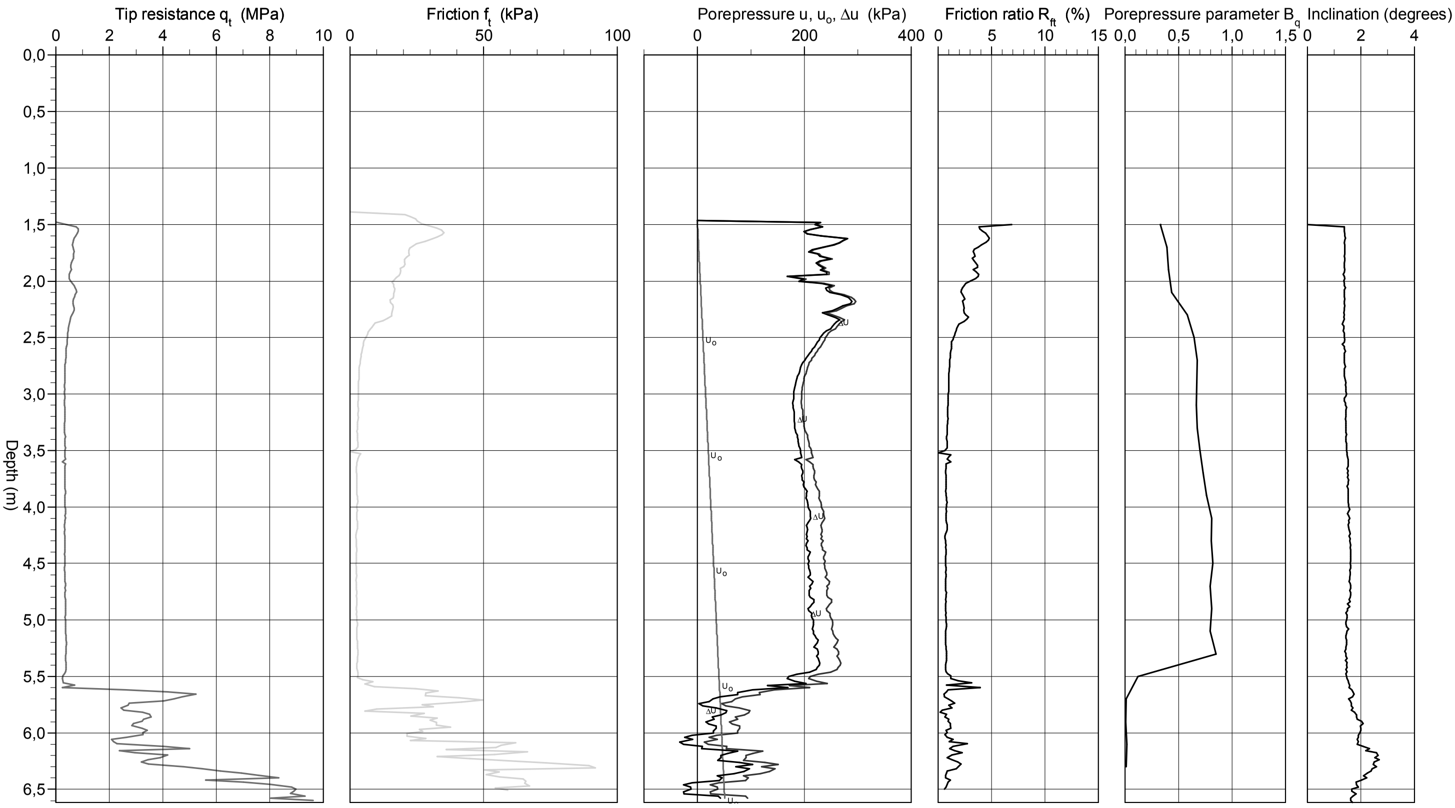


C:\Users\CGI\Desktop\Dalvägen files\CPT\CPT 16B28.cpw 2016-05-19

CPT-test performed according to EN ISO 22476-1

Predrilling depth	1,50 m	Reference		Fluid in filter		
Start depth	1,50 m	Level at reference		Coordinats		
Stop depth	6,62 m	Predrilled material		Equipment		
Ground water level	1,50 m	Geometry	Normal	Cone nr	4603	

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B45
Date	2016-04-06

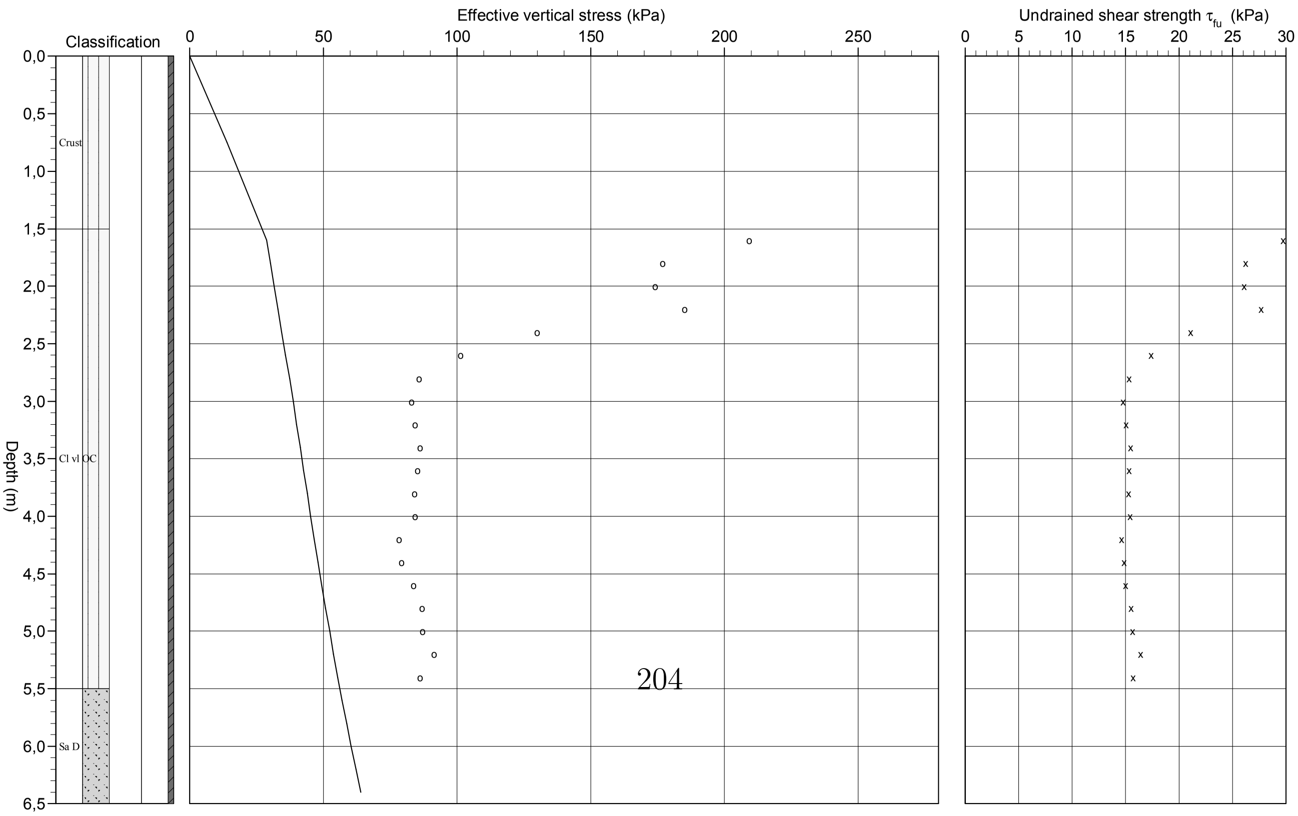


F:\Tesi\Dalvägen files\CPT\CPT 16B45.cpw
2016-06-09

CPT test evaluated accorrding to SGI Information 15 rev. 2007

Reference		Predrilling depth	1,50 m	Evaluator	
Ground water level		Predrilled material		Evaluation date	
Grundvattenyta	1,50 m	Equipment			
Start depth	1,50 m	Geometry	Normal		

Project	Dalvägen
Project nr	16U29452
Site	Dalvägen
Designation	16B45
Date	2016-04-06



F:\Tesi\Dalvägen files\CPT\CPT 16B45.cpw 2016-06-09

Konprovstabell

Projekt **Dalvägen**		*Löp-nr* 30056	*Gransk./Tabell*
Uppdragsnummer 16U29452	*Uppdragsgivare* Bjerking AB, Stockholm	*Provtagningsdatum* 2016-04-20 / *Provtagningsredskap* Kv St II ø 50mm	*Datum/Sign* 2016-05-06
Referensnivå		*Vattennivå / Datum* /	*Undersökningsdatum* 2016-05-06

Sektion Djup [m]	Borrhål **16B34** Benämning[1)]	Densitet Diameter [cm]	Densitet Vikt/Längd [g/cm]	Densitet ρ [t/m³]	Konprov Ostört [mm][2)]	Konprov Ostört Medel [mm/g]	Konprov Omrört [mm/g]	Skjuvhållfasthet Ostört τ_{fu} [kPa][3)]	Skjuvhållfasthet Omrört [kPa]	Sensitivitet S_t	Konflytgräns w_L [%]	w-våt w-torr [g]	Vattenkvot w [%]	Skål nr	Jordartsförkortning (enl. Beteckningsblad IEG 2011-05-08)
2.0	Grå rostfläckig varvig lera med enstaka tunna finsandsskikt samt enstaka gruskorn skredtecken	5,00	618.0 / 17.0	**1.85**	12.7 12.5 12.8 13.2 12.3 12.3	12.6 / 400	6.4 / 60	**25**	3.6	**7**	**49**	86.2 61.3	**41**	219	vCl (fsa)
							8.1 / 60					76.5 52.5		220	
4.0	Grå sulfidbandad varvig lera med enstaka tunna siltskikt samt enstaka sandkorn	5,00	568.0 / 17.0	**1.70**	9.1 9.2 9.6 9.4 9.1 9.1	9.3 / 100	17.3 / 60	**11**	0.49	**22**	**45**	48.3 31.2	**55**	221	suvCl (si)
							11.4 / 60					69.3 47.3		222	
6.0	Grå sulfidbandad varvig lera	5,00	557.0 / 17.0	**1.67**	9.0 9.0 9.1 8.9 9.1 9.0	9.0 / 100	19.7 / 60	**12**	0.38	**32**	**46**	61.6 37.5	**64**	223	suvCl
							11.8 / 60					84.1 56.7		224	
8.0	Grå sulfidbandad varvig lera med enstaka tunna siltskikt	5,00	571.0 / 17.0	**1.71**	8.9 8.2 8.6 8.5 8.7 8.2	8.5 / 100	19.3 / 60	**14**	0.40	**35**	**44**	71.8 45.5	**58**	225	suvCl (si)
							13.7 / 60					85.2 57.7		226	

205

1) Okulär jordartsklassificering enl. SS-EN ISO 14688-1+2
2) Fallhöjd: 0 mm har använts
3) Okorrigerat värde. Korrigeringen rekommenderas enl. SGF-INFO nr 3. Avvikelse från SS027125: Om konintrycket är mindre än 7,0 mm med 100g konen, används 400g konen, enligt rekommendation från SGF:s laboratoriekommitté.

P:\2172\Uppdrag 2016\30056\[Kon 16B34 160506.xlsx]

SWECO

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044,
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60,
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

SWECO GEOLAB

Konprovstabell

Projekt **Dalvägen**		*Löp-nr* 30056	*Gransk./Tabell*
Uppdragsnummer 16U29452	*Uppdragsgivare* Bjerking AB, Stockholm	*Provtagningsdatum* 2016-04-20	*Datum/Sign* 2016-05-06
		Provtagningsredskap Kv St II ø 50mm	*Undersökningsdatum* 2016-05-06
Referensnivå		*Vattennivå / Datum* /	

Sektion Djup [m]	Borrhål **16B40** Benämning[1)]	Densitet Diameter [cm]	Densitet Vikt/Längd [g/cm]	Densitet ρ [t/m^3]	Konprov Ostört [mm][2)]	Konprov Ostört Medel [mm/g]	Konprov Omrört [mm/g]	Skjuvhållfasthet Ostört τ_{fu} [kPa][3)]	Skjuvhållfasthet Omrört [kPa]	Sensitivitet S_t	Konflytgräns w_L [%]	w-våt w-torr [g]	Vattenkvot w [%]	Skål nr	Jordartsförkortning (enl. Beteckningsblad IEG 2011-05-08)
2.0	Grå rostfläckig varvig lera	5,00	586.0 / 17.0	**1.76**	11.0 11.0 11.0 13.0 13.0 11.5	11.8 / 400	7.0 / 60	**28**	3.0	**9**	**58**	83.8 55.6	**51**	586	vCl
3.0	Grå något sulfidbandad varvig lera	5,00	558.0 / 17.0	**1.67**	9.2 8.8 9.0 9.0 9.0 9.1	9.0 / 100	14.6 / 60	**12**	0.69	**17**	**54**	52.5 32.7	**61**	587	(su)vCl
5.0	Grå något sulfidbandad varvig lera med enstaka mycket tunna finsandsskikt	5,00	561.0 / 17.0	**1.68**	9.3 9.5 8.4 9.5 9.7 9.4	9.3 / 100	16.4 / 60	**11**	0.55	**20**	**43**	56.4 36.7	**54**	588	(su)vCl (fsa)
							10.8 / 60					83.6 58.1		589	

1) Okulär jordartsklassificering enl. SS-EN ISO 14688-1+2

2) Fallhöjd: 0 mm har använts

P:\2172\Uppdrag 2016\30056\[Kon 16B40 160506.xlsx]

3) Okorrigerat värde. Korrigeringen rekommenderas enl. SGF-INFO nr 3. Avvikelse från SS027125: Om konintrycket är mindre än 7,0 mm med 100g konen, används 400g konen, enligt rekommendation från SGF:s laboratoriekommitté.

SWECO

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044,
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60,
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

SWECO GEOLAB

Konprovstabell

Projekt **Dalvägen**		*Löp-nr* 30056	*Gransk./Tabell*
Uppdragsnummer	*Uppdragsgivare*	*Provtagningsdatum* / *Provtagningsredskap*	*Datum/Sign* 2016-04-20
16U29452	Bjerking AB, Stockholm	Kv St II ø 50mm	*Undersökningsdatum*
Referensnivå		*Vattennivå / Datum* /	2016-04-20

Sektion Djup [m]	Borrhål **16B43** Benämning[1)]	Densitet Diameter [cm]	Densitet Vikt/Längd [g/cm]	Densitet ρ [t/m³]	Konprov Ostört [mm][2)]	Konprov Ostört Medel [mm/g]	Konprov Omrört [mm/g]	Skjuvhållfasthet Ostört τ_{fu} [kPa][3)]	Skjuvhållfasthet Omrört [kPa]	Sensitivitet S_t	Konflytgräns w_L [%]	w-våt w-torr [g]	Vattenkvot w [%]	Skål nr	Jordartsförkortning (enl. Beteckningsblad IEG 2011-05-08)
2.5	Grå något sulfidfläckig varvig lera	5,00	593.0 / 17.0	**1.78**	7.3 7.0 7.0 7.0 7.0 7.2	7.1 / 100	9.0 / 60	**20**	1.8	**11**	**49**	80.0 54.2	**48**	394	(su)vCl
4.0	Grå något sulfidbandad varvig lera	5,00	586.0 / 17.0	**1.76**	8.5 8.9 8.3 8.9 8.8 8.8	8.7 / 100	14.8 / 60	**13**	0.67	**19**	**44**	74.5 50.1	**49**	395	(su)vCl
6.0	Grå varvig lera	5,00	565.0 / 17.0	**1.69**	8.7 8.5 8.2 8.5 8.4 8.4	8.4 / 100	16.0 / 60	**14**	0.58	**24**	**43**	66.1 43.0	**54**	254	vCl
							9.9 / 60					84.1 58.8		255	

207

1) Okulär jordartsklassificering enl. SS-EN ISO 14688-1+2
2) Fallhöjd: 0 mm har använts
3) Okorrigerat värde. Korrigeringen rekommenderas enl. SGF-INFO nr 3. Avvikelse från SS027125: Om konintrycket är mindre än 7,0 mm med 100g konen, används 400g konen, enligt rekommendation från SGF:s laboratoriekommitté.

P:\2172\Uppdrag 2016\30056\[Kon 16B43 160420.xlsx]

SWECO

Konprovstabell

Projekt **Dalvägen**		*Löp-nr* 30056	*Gransk./Tabell*
Uppdragsnummer 16U29452	*Uppdragsgivare* Bjerking AB, Stockholm	*Provtagningsdatum* 2016-04-07 / *Provtagningsredskap* Kv St II ø 50mm	*Datum/Sign* 2016-04-22 / *Undersökningsdatum*
Referensnivå		*Vattennivå / Datum* /	2016-04-22

Sektion Djup [m]	Borrhål **16B46** Benämning[1)]	Diameter [cm]	Densitet Vikt/Längd [g/cm]	Densitet ρ [t/m^3]	Konprov Ostört [mm][2)]	Konprov Ostört Medel [mm/g]	Konprov Omrört [mm/g]	Skjuvhållfasthet Ostört τ_{fu} [kPa][3)]	Skjuvhållfasthet Omrört [kPa]	Sensitivitet S_t	Konflytgräns w_L [%]	w-våt w-torr [g]	Vattenkvot w [%]	Skål nr	Jordartsförkortning (enl. Beteckningsblad IEG 2011-05-08)
2.5	Brungrå något sulfidbandad varvig lera skredtecken	5,00	585.0 / 17.0	**1.75**	7.0 7.2 7.7 7.0 7.2 7.5	7.3 / 100	12.5 / 60	**19**	0.94	**20**	**48**	60.6 40.1	**51**	269	(su)vCl
4.0	Brungrå något sulfidbandad varvig lera	5,00	564.0 / 17.0	**1.69**	8.5 8.5 9.0 9.0 9.0 8.8	8.8 / 100	16.3 / 60	**13**	0.55	**24**	**46**	75.1 48.0	**56**	270	(su)vCl
							11.0 / 60					73.5 49.8		348	
6.0	Brungrå något sulfidfläckig varvig lera med enstaka grusskikt	5,00	596.0 / 17.0	**1.79**	9.2 8.8 9.0 8.8 9.0 9.1	9.0 / 100	16.8 / 60	**12**	0.52	**23**	**42**	77.9 51.4	**52**	271	(su)vCl (gr)
							13.0 / 60					85.1 58.5		349	

1) Okulär jordartsklassificering enl. SS-EN ISO 14688-1+2
2) Fallhöjd: 0 mm har använts

P:\2172\Uppdrag 2016\30056\[Kon 16B46 160422.xlsx]

3) Okorrigerat värde. Korrigeringen rekommenderas enl. SGF-INFO nr 3. Avvikelse från SS027125: Om konintrycket är mindre än 7,0 mm med 100g konen, används 400g konen, enligt rekommendation från SGF:s laboratoriekommitté.

SWECO

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044,
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60,
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

SWECO GEOLAB

Konprovstabell

Projekt **Södra Boo**		*Löp-nr* 27005	*Gransk./Tabell*
Uppdragsnummer 10192941	*Uppdragsgivare* WSP Samhällsbyggnad, Stockholm	*Provtagningsdatum* 2014-05-21 / *Provtagningsredskap* Kv St I ø 50mm	*Datum/Sign* 2014-06-05 / *Undersökningsdatum*
Referensnivå	My	*Vattennivå / Datum* /	2014-06-05

Sektion Djup [m]	Borrhål **14W17** Benämning[1]	Densitet Diameter [cm]	Densitet Vikt/Längd [g/cm]	Densitet ρ [t/m³]	Konprov Ostört [mm][2]	Konprov Ostört Medel [mm/g]	Konprov Omrört [mm/g]	Skjuvhållfasthet Ostört τ_{fu} [kPa][3]	Skjuvhållfasthet Omrört [kPa]	Sensitivitet S_t	Konflytgräns w_L [%]	w-våt w-torr [g]	Vattenkvot w [%]	Skål nr	Jordartsförkortning (enl. SGF/BGS Beteckningssystem 2001:1)
2.0	Grå rostfläckig varvig lera med enstaka tunna siltskikt torrskorpekaraktär	5,00	614.0 / 17.0	**1.84**	9.3 9.2 9.3 9.2 9.2 9.2	9.2 / 400	5.0 / 60	**46**	5.9	**8**	**49**	33.9 24.4	**39**	131	vLe(t) (si)
							7.0 / 60					85.4 59.6		132	
3.0	Grå varvig lera	5,00	564.0 / 17.0	**1.69**	8.9 9.0 8.8 9.0 9.2 9.0	9.0 / 100	13.6 / 60	**12**	0.80	**15**	**50**	76.0 49.2	**54**	133	vLe
4.0	Grå varvig lera med enstaka tunna finsandsskikt	5,00	448.0 / 13.4	**1.70**	7.8 8.0 8.0 8.1 8.1 8.0	8.0 / 100	12.1 / 60	**15**	1.0	**15**	**60**	71.6 43.7	**64**	134	vLe (saf)

1) Okulär jordartsklassificering enl. SGF 1981
2) Fallhöjd: 0 mm har använts
3) Okorrigerat värde. Korrigeringen rekommenderas enl. SGF-INFO nr 3

P:\2172\Uppdrag 2014\27005\[Kon 14W17 140605.xlsx]

SWECO

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044,
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60,
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

SWECO GEOLAB

Konprovstabell

Projekt **Södra Boo**		*Löp-nr* 27005	*Gransk./Tabell*
Uppdragsnummer 10192941	*Uppdragsgivare* WSP Samhällsbyggnad, Stockholm	*Provtagningsdatum* 2014-04-23 / *Provtagningsredskap* Kv St I ø 50mm	*Datum/Sign* 2014-05-20
Referensnivå		*Vattennivå / Datum* /	*Undersökningsdatum* 2014-05-20

Sektion / Djup [m]	Borrhål **14W44** Benämning[1]	Diameter [cm]	Densitet Vikt/Längd [g/cm]	Densitet ρ [t/m³]	Konprov Ostört [mm][2]	Konprov Ostört Medel [mm/g]	Konprov Omrört [mm/g]	Skjuvhållfasthet Ostört τ_{fu} [kPa][3]	Skjuvhållfasthet Omrört [kPa]	Sensitivitet S_t	Konflytgräns w_L [%]	w-våt w-torr [g]	Vattenkvot w [%]	Skål nr	Jordartsförkortning (enl. SGF/BGS Beteckningssystem 2001:1)
2.0	Grå sulfidfläckig varvig lera	5,00	549.0 / 17.0	**1.64**	9.9 10.0 10.1 10.0 10.3 10.2	10.1 / 100	17.0 / 60	**9.7**	0.51	**19**	**49**	33.1 20.4	**62**	425	suvLe
							13.0 / 60					25.3 16.5		426	
4.0	Grå sulfidhaltig varvig lera	5,00	554.0 / 17.0	**1.66**	10.2 10.1 9.9 9.9 9.9 9.9	10.0 / 100	10.0 / 10	**10**	0.25	**40**	**43**	33.7 21.0	**60**	427	suvLe
							11.5 / 60					56.9 39.4		428	
6.0	Brungrå något sulfidhaltig varvig lera	5,00	553.0 / 17.0	**1.66**	9.2 9.2 9.0 8.8 9.2 9.2	9.1 / 100	10.2 / 10	**12**	0.24	**50**	**47**	48.8 30.2	**62**	429	(su)vLe
							11.0 / 60					45.7 30.9		430	
8.0	Brungrå varvig lera	5,00	565.0 / 17.0	**1.69**	8.1 7.9 7.8 8.2 8.2 8.0	8.0 / 100	16.0 / 60	**15**	0.58	**26**	**49**	37.0 23.4	**58**	431	vLe
							7.5 / 60					52.6 36.4		432	

1) Okulär jordartsklassificering enl. SGF 1981
2) Fallhöjd: 0 mm har använts
3) Okorrigerat värde. Korrigeringen rekommenderas enl. SGF-INFO nr 3

P:\2172\Uppdrag 2014\27005\[Kon 14W44 140520.xlsx]

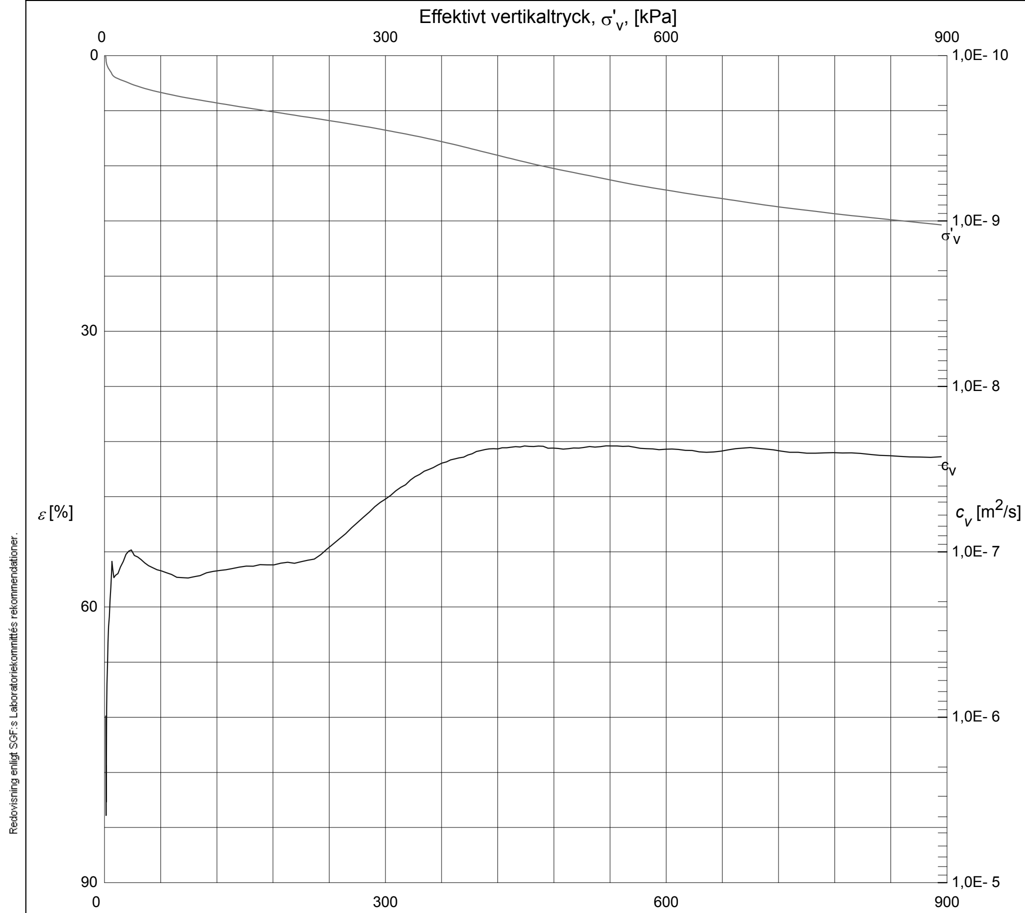
SWECO

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044,
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60,
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**		
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17	*Djup:* 2,0 m	*Ödometer nr:* 2
Densitet: 1,84 t/m^3 *Vattenkvot:* 39 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Rostfl. v lera m enst tu siltskikt torrsk.karaktär		*Provhöjd:* 20 mm *Def.hastighet:* 0,68 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och *k* har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\ min}$, m^2/s	k_i , m/s	β_k
285	4041	421	13,2	2,3E-8	2,1E-10	5,0

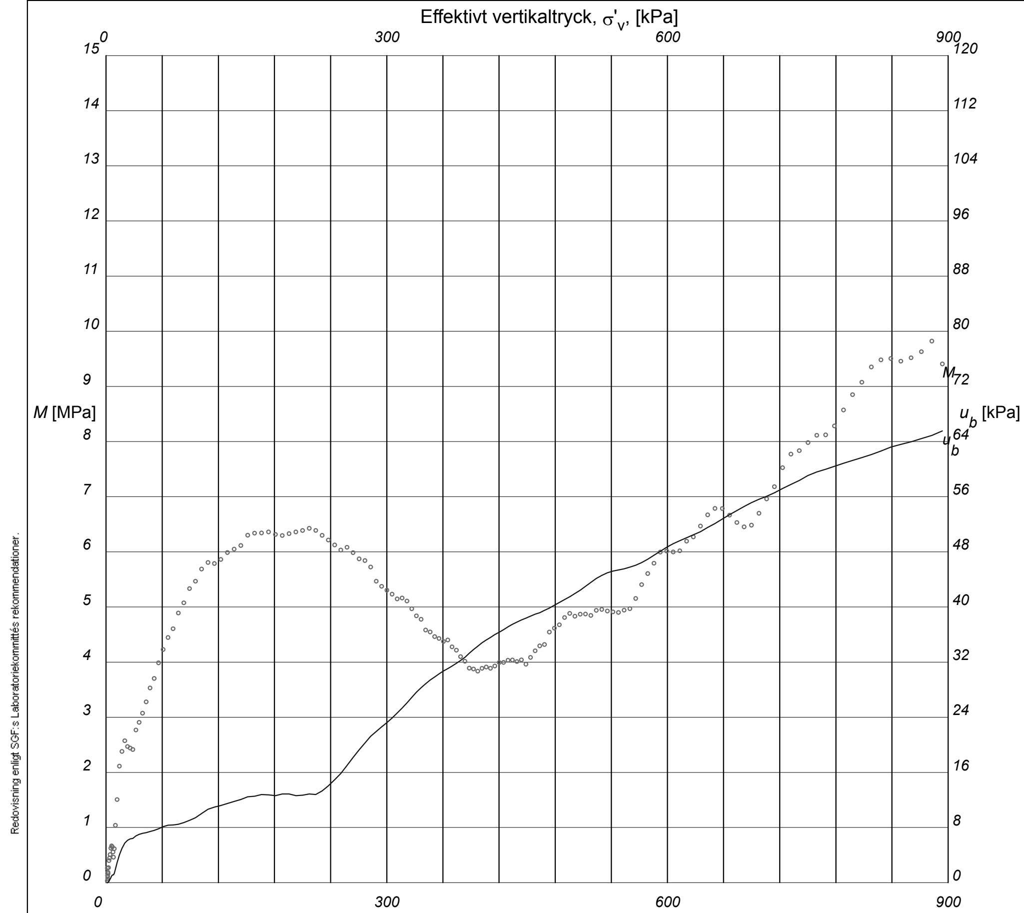
Anm.

Skalan i diagrammet avviker från den av SGF:s Laboratoriekommité satta rekommendation.

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**		
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17	*Djup:* 2,0 m	*Ödometer nr:* 2
Densitet: 1,84 t/m^3 *Vattenkvot:* 39 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Rostfl. v lera m enst tu siltskikt torrsk.karaktär		*Provhöjd:* 20 mm *Def.hastighet:* 0,68 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

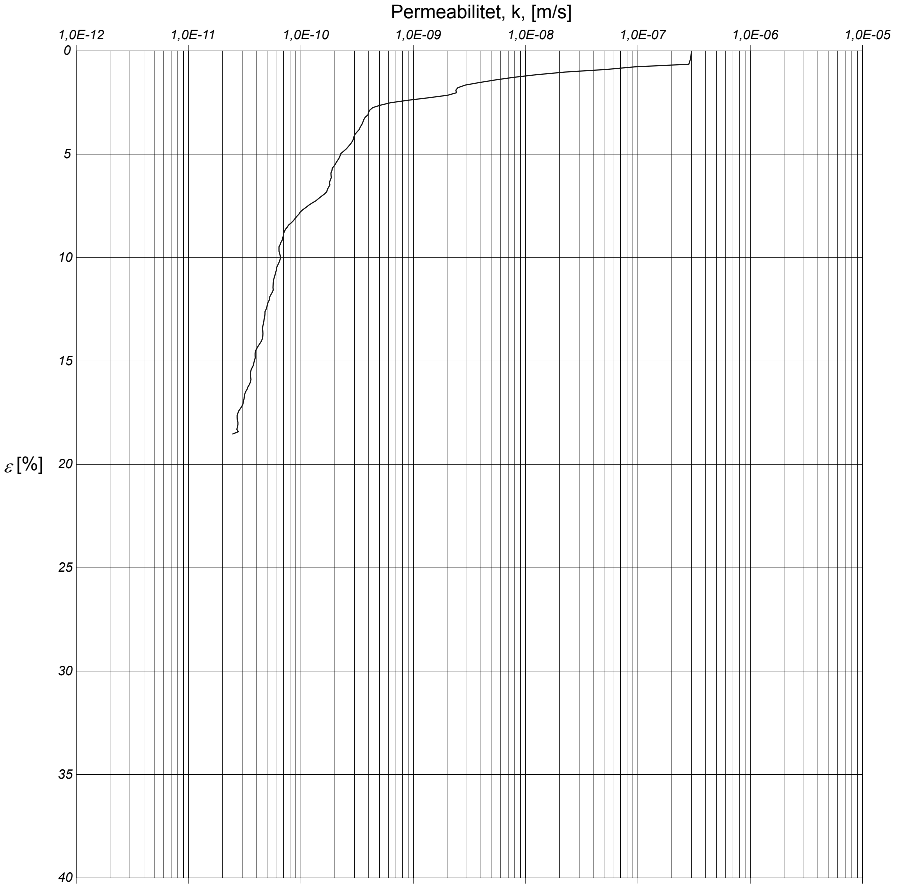
M'	σ'_L, kPa
13,2	421

Anm.

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Södra Boo**		
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17 *Djup:* 2,0 m *Densitet:* 1,84 t/m^3 *Vattenkvot:* 39 % *Provningstemp.:* 20 ^{o}C *Benämning:* Rostfl. v lera m enst tu siltskikt torrsk.karaktär		*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,68 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

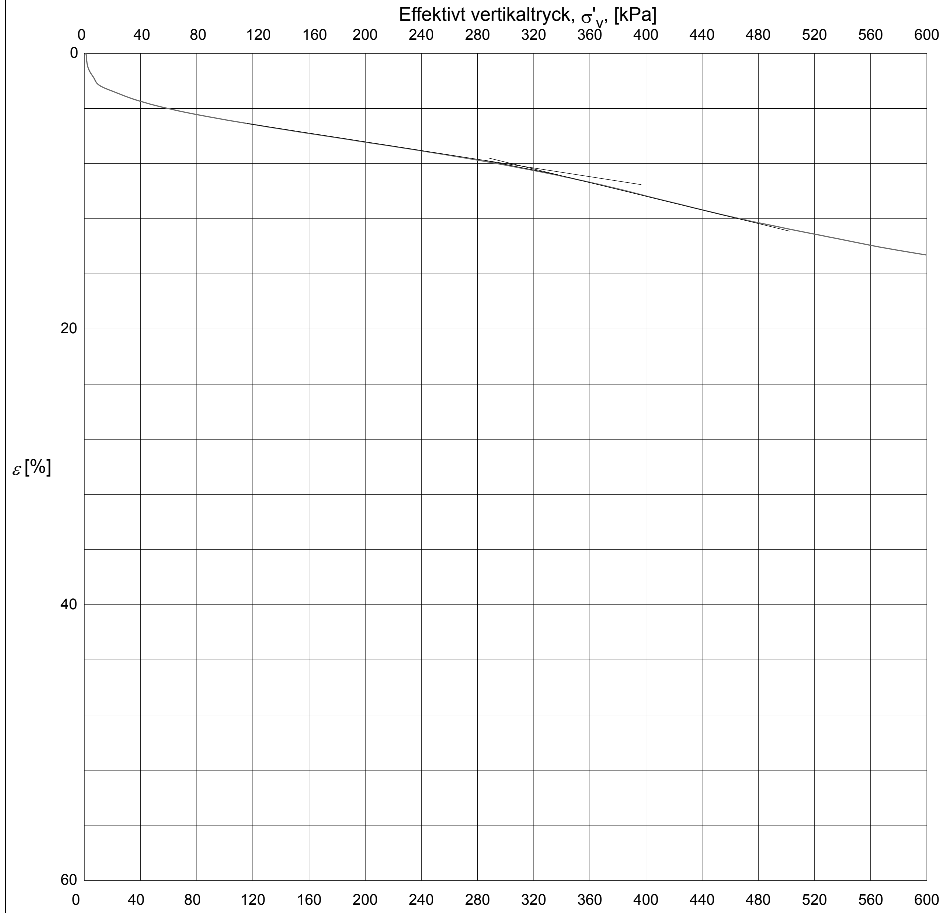
k_i, m/s	β_k
2,1E-10	5,0

Anm.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**		
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17 *Djup:* 2,0 m *Densitet:* 1,84 t/m^3 *Vattenkvot:* 39 % *Provningstemp.:* 20 ^{o}C *Benämning:* Rostfl. v lera m enst tu siltskikt torrsk.karaktär		*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,68 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

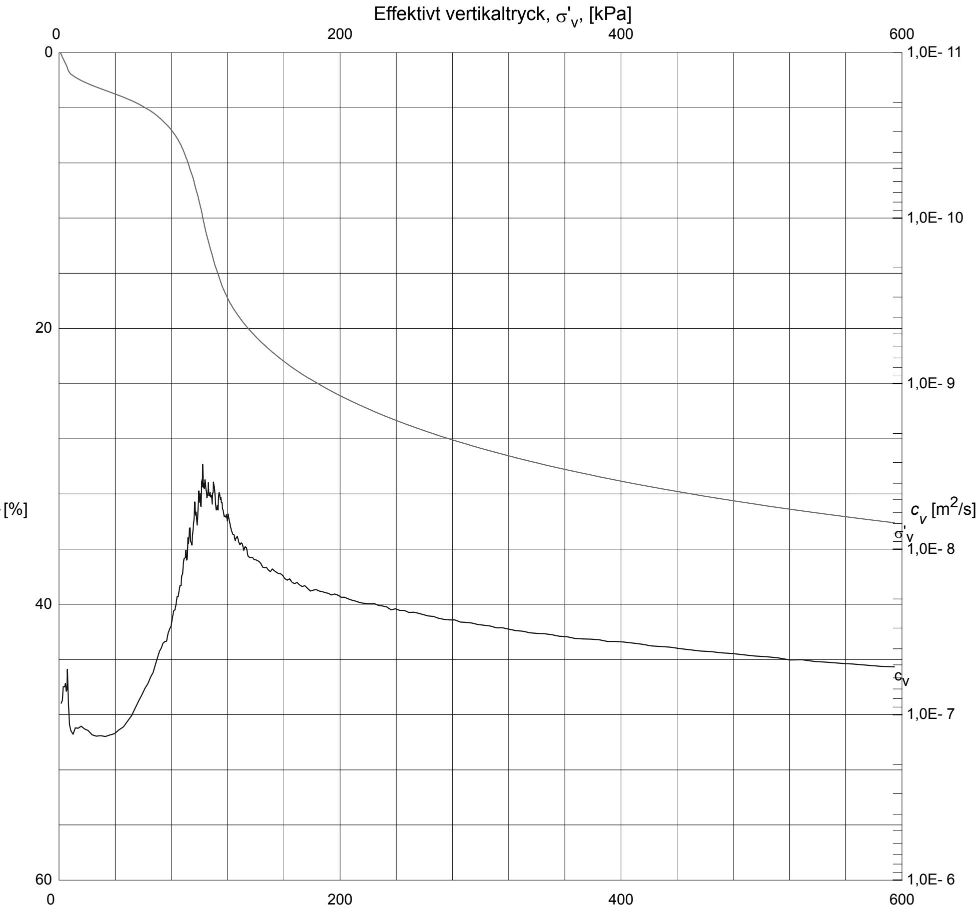
σ'_c, kPa	M_L, kPa	σ'_L, kPa
285	4041	421

Anm.

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**		
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17 *Densitet:* 1,69 t/m^3 *Vattenkvot:* 54 % *Benämning:* Varvig lera	*Djup:* 3,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\ min}$, m^2/s	k_i, m/s	β_k
68	223	106	17,7	4,2E-9	3,6E-10	2,3

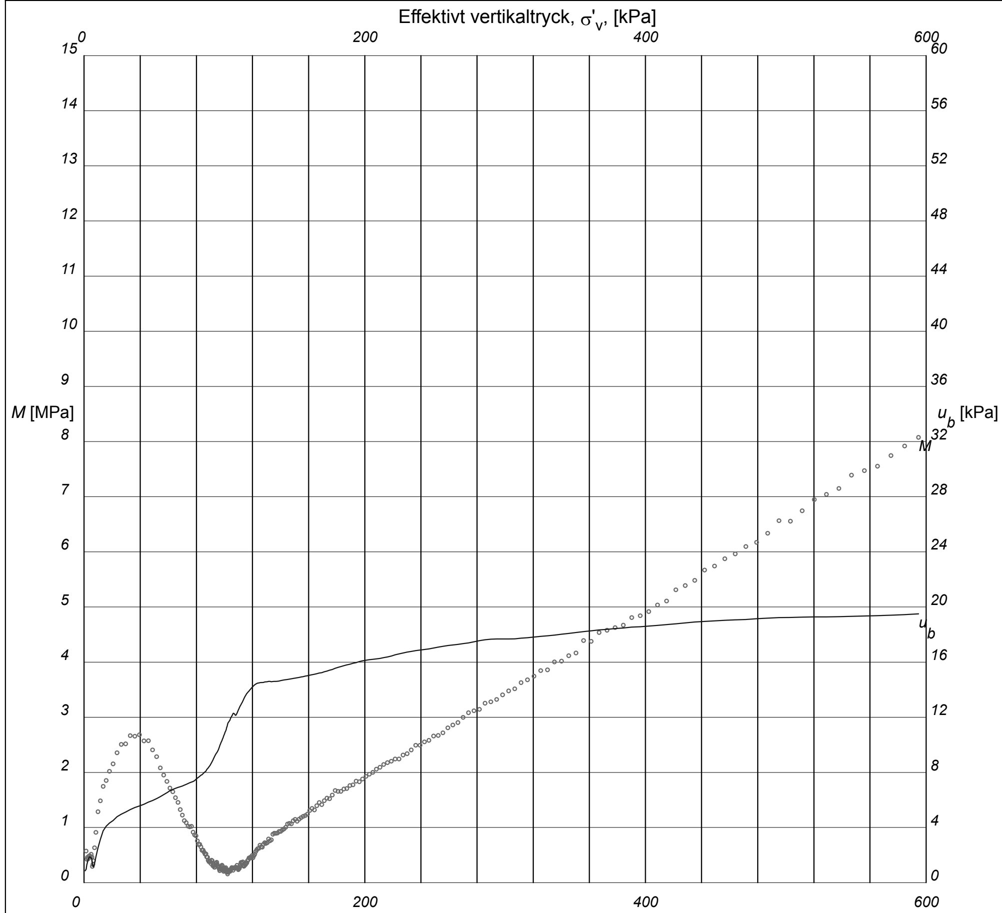
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**		
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17 *Densitet:* 1,69 t/m^3 *Vattenkvot:* 54 % *Benämning:* Varvig lera	*Djup:* 3,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
17,7	106

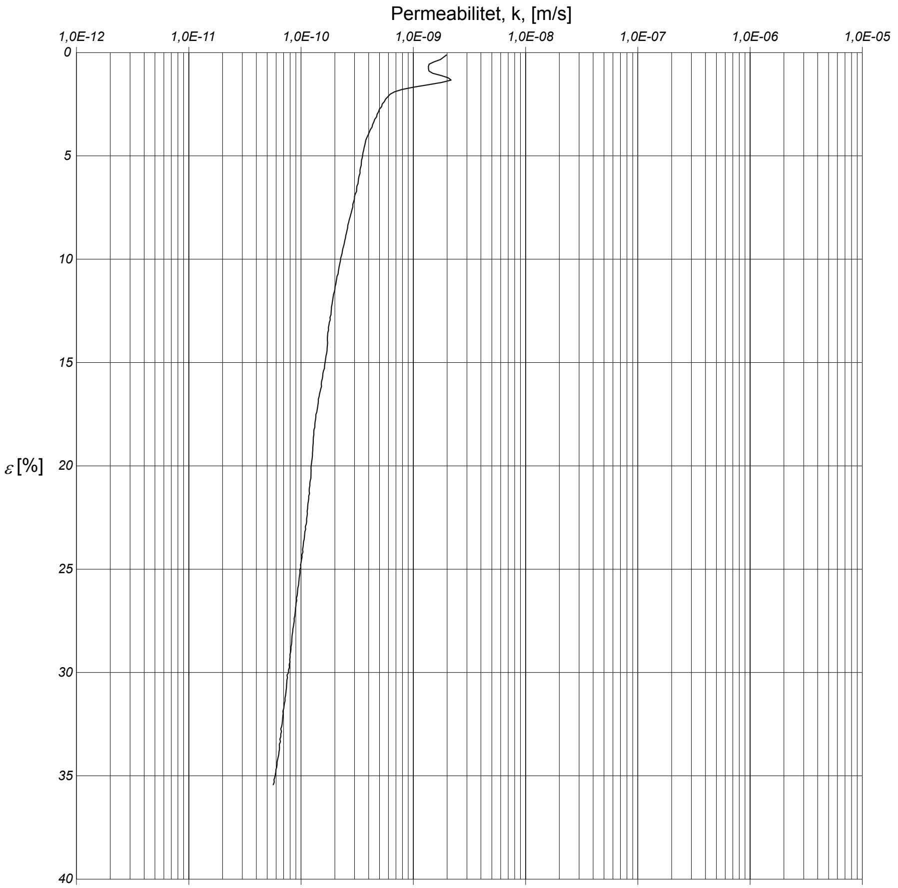
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Södra Boo**			
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm		*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17		*Djup:* 3,0 m	*Ödometer nr:* 3
Densitet: 1,69 t/m^3	*Vattenkvot:* 54 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Varvig lera			*Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

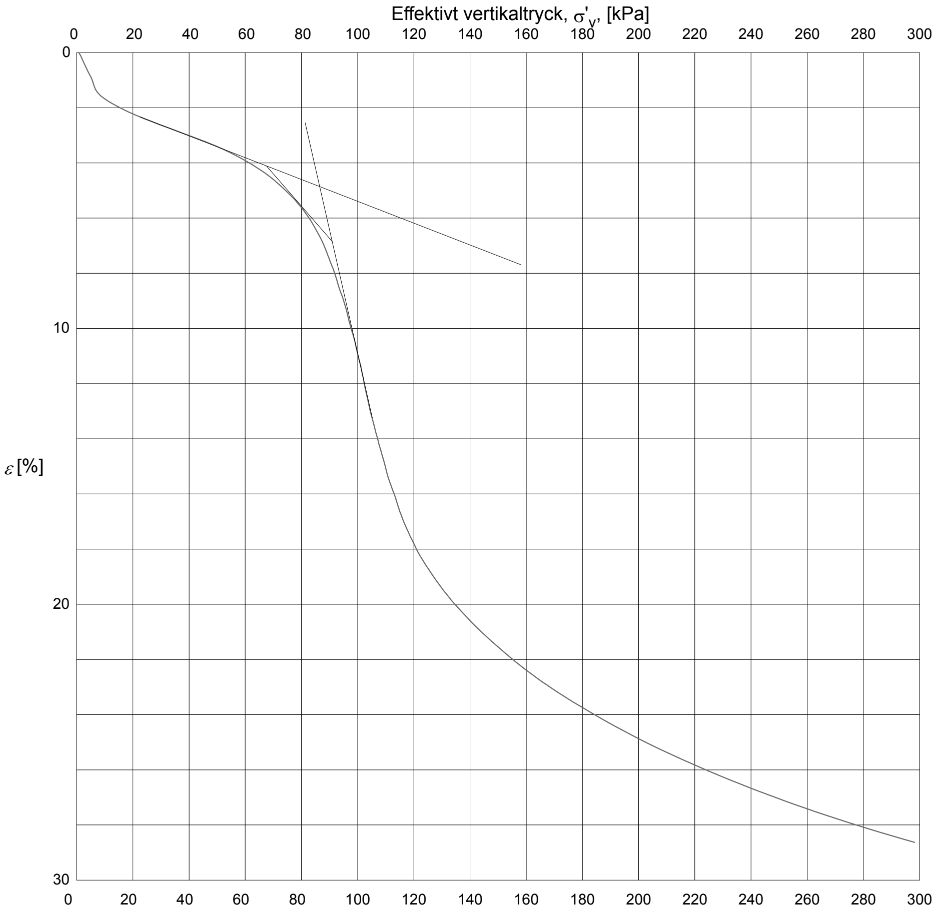
k_i , m/s	β_k
3,6E-10	2,3

Anm.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**			
Uppdragsnummer: 10192941	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm		*Datum/Sign:* 2014-05-30 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W17		*Djup:* 3,0 m	*Ödometer nr:* 3
Densitet: 1,69 t/m^3	*Vattenkvot:* 54 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Varvig lera			*Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
68	223	106

Anm.

SWECO GEOLAB

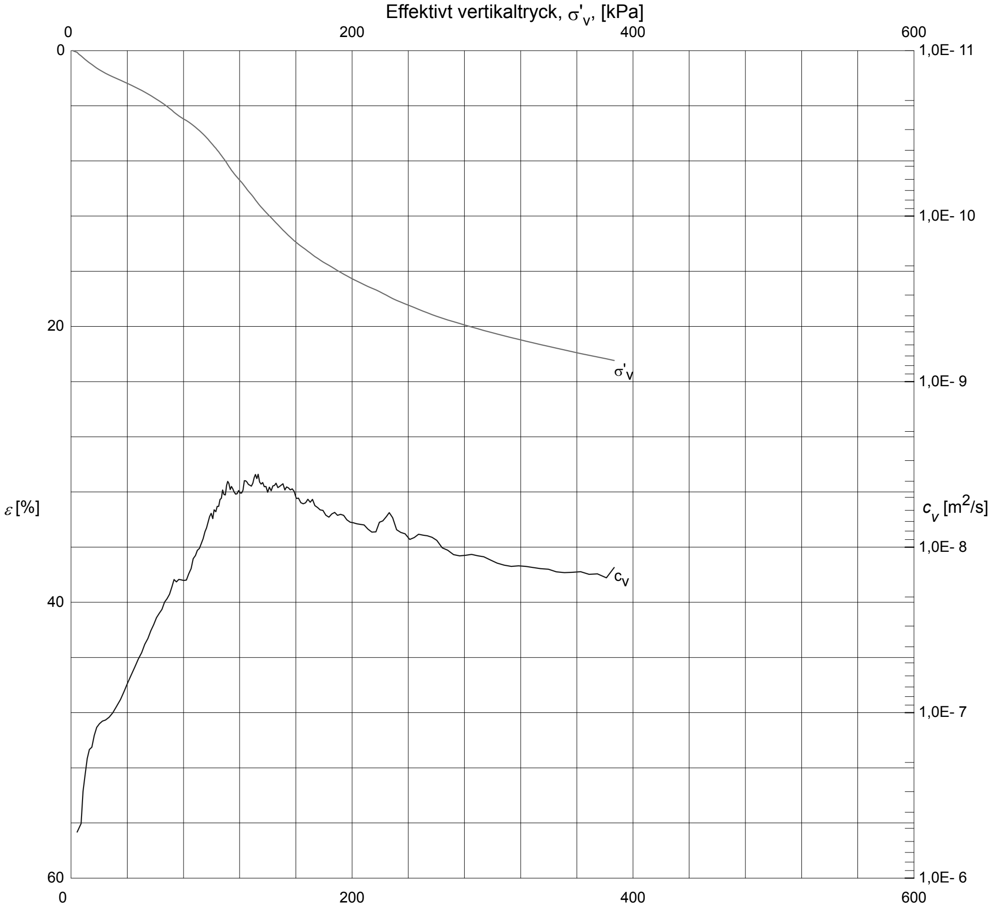
Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2014-06-02
10192941	WSP Samhällsbyggnad, Stockholm	*Löp-nr/Gransk.:* 27005

Sektion/borrhål: 14W17 *Djup:* 4,0 m
Densitet: 1,7 t/m^3 *Vattenkvot:* 64 % *Provningstemp.:* 20 ^{o}C
Benämning: Varvig lera med enstaka tunna finsandsskikt

Ödometer nr: 4
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,75 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och *k* har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\,min}$, m^2/s	k_i , m/s	β_k
62	760	134	16,9	4,3E-9	6,9E-11	1,7

Anm.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\a14_573.txt 2014-06-05

SWECO GEOLAB

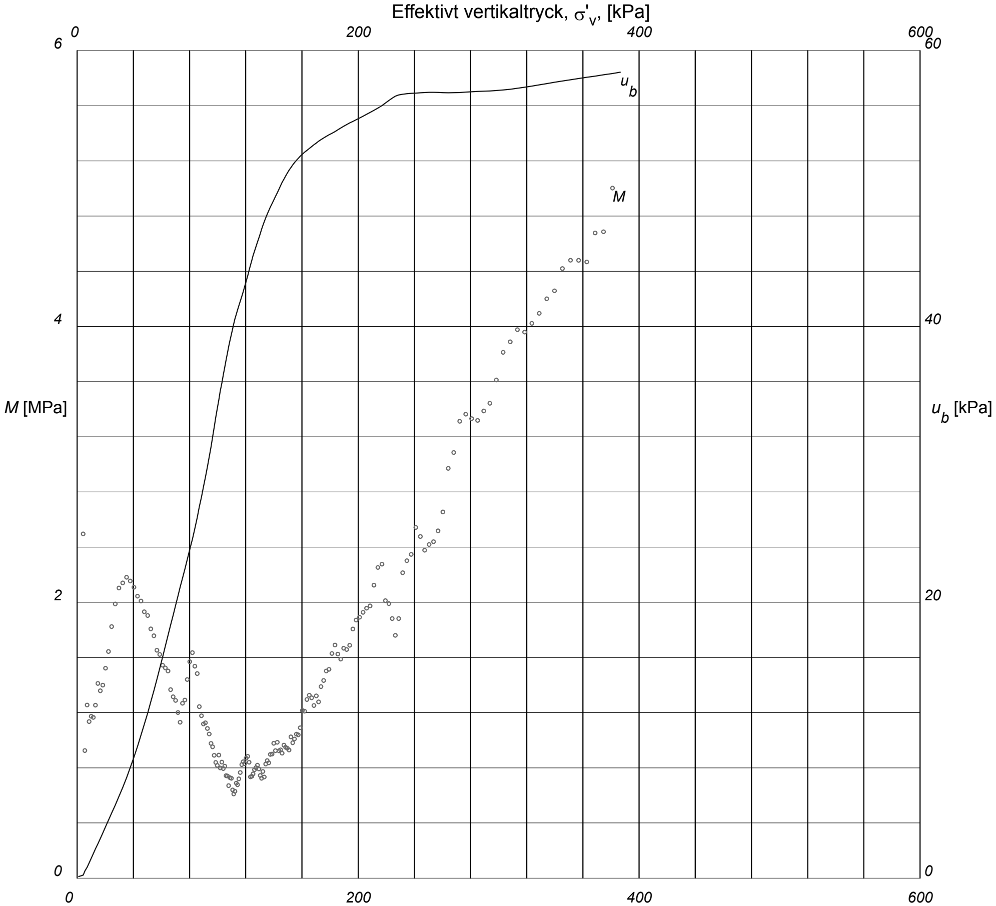
Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2014-06-02
10192941	WSP Samhällsbyggnad, Stockholm	*Löp-nr/Gransk.:* 27005

Sektion/borrhål: 14W17 *Djup:* 4,0 m
Densitet: 1,7 t/m^3 *Vattenkvot:* 64 % *Provningstemp.:* 20 ^{o}C
Benämning: Varvig lera med enstaka tunna finsandsskikt

Ödometer nr: 4
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,75 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
16,9	134

Anm.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\a14_573.txt 2014-06-05

SWECO GEOLAB

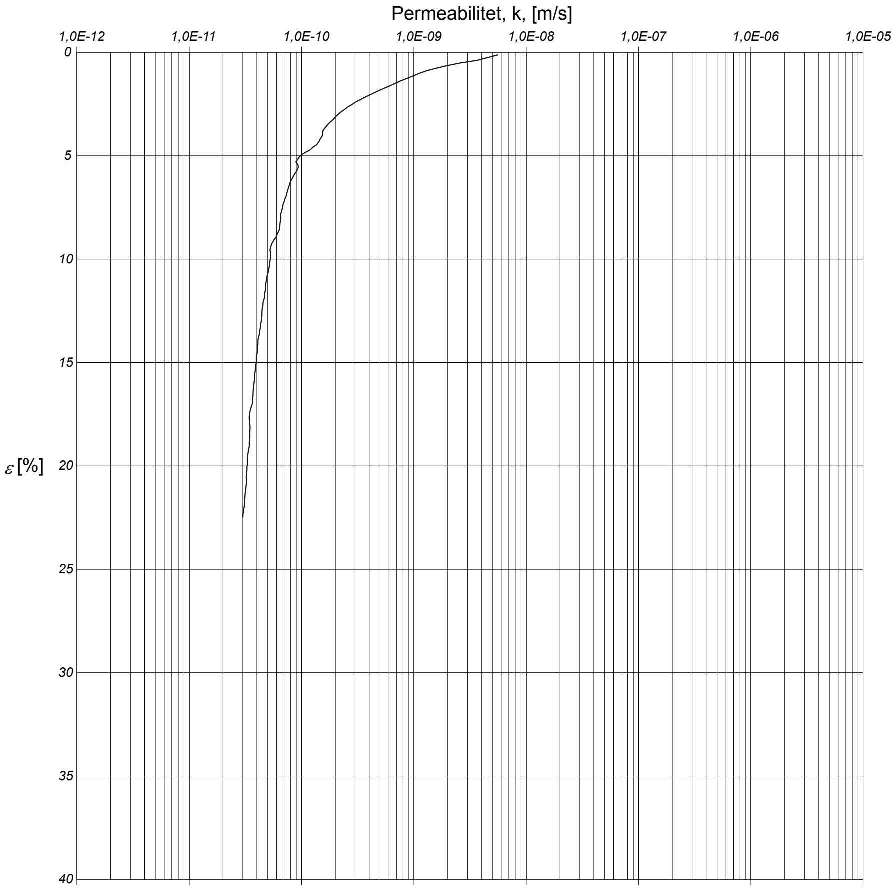
Utvärdering av permeabilitet

Projekt: **Södra Boo**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2014-06-02
10192941	WSP Samhällsbyggnad, Stockholm	*Löp-nr/Gransk.:* 27005

Sektion/borrhål: 14W17 *Djup:* 4,0 m
Densitet: 1,7 t/m^3 *Vattenkvot:* 64 % *Provningstemp.:* 20 ^oC
Benämning: Varvig lera med enstaka tunna finsandsskikt

Ödometer nr: 4
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,75 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC.

k_i, m/s	β_k
6,9E-11	1,7

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

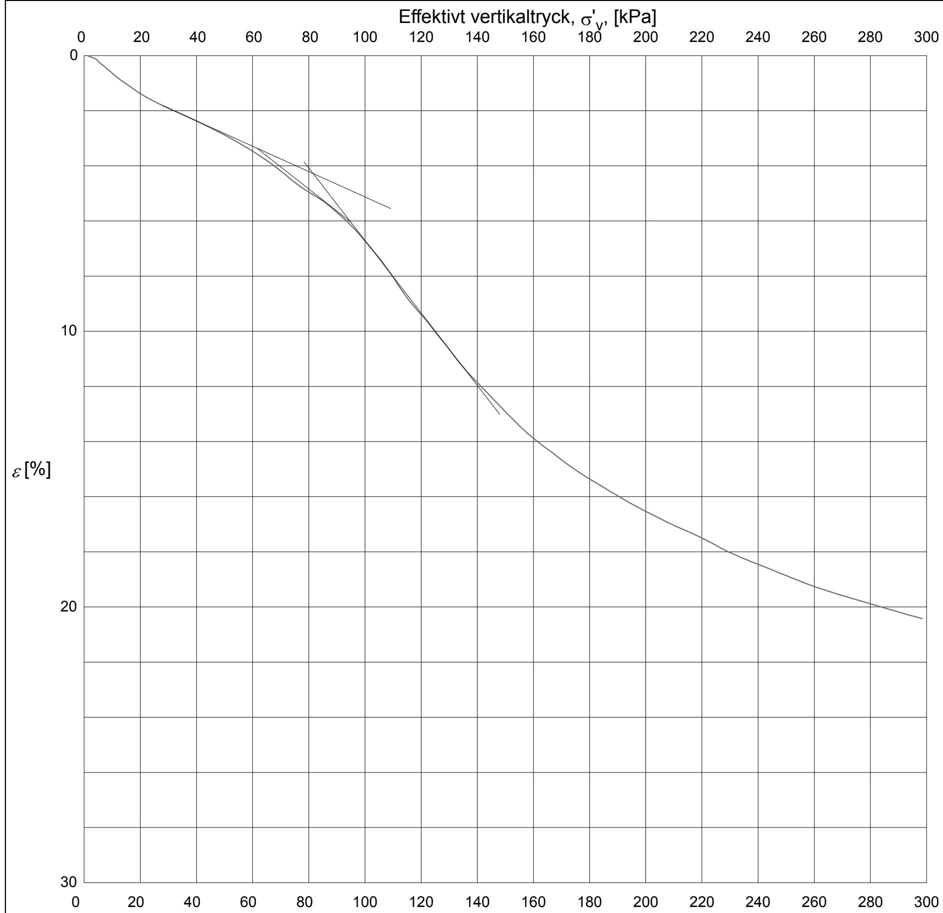
Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2014-06-02
10192941	WSP Samhällsbyggnad, Stockholm	*Löp-nr/Gransk.:* 27005

Sektion/borrhål: 14W17 *Djup:* 4,0 m
Densitet: 1,7 t/m^3 *Vattenkvot:* 64 % *Provningstemp.:* 20 ^oC
Benämning: Varvig lera med enstaka tunna finsandsskikt

Ödometer nr: 4
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,75 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
62	760	134

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

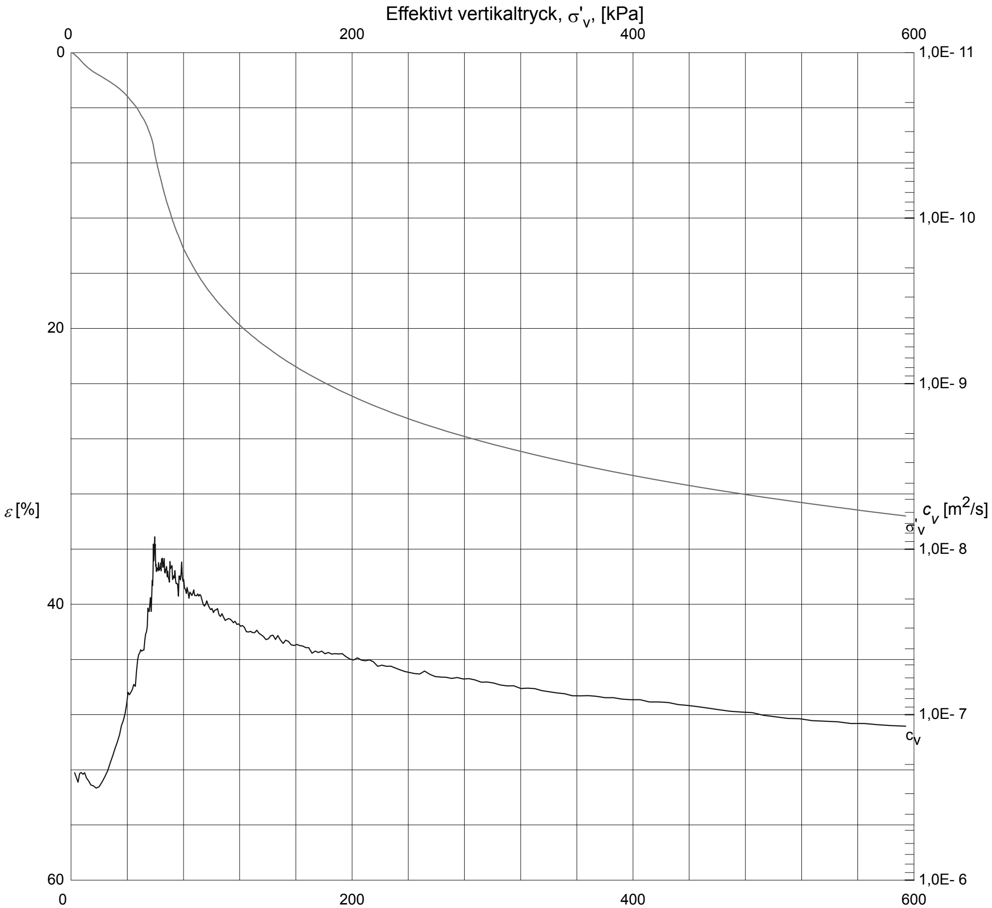
SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**

Uppdragsnummer: 10192148
Uppdragsgivare: WSP Samhällsbyggnad, Stockholm
Datum/Sign: 2014-05-16
Löp-nr/Gransk.: 27005

Sektion/borrhål: 14W44 *Djup:* 2,0 m
Ödometer nr: 3
Densitet: 1,64 t/m^3 *Vattenkvot:* 62 % *Provningstemp.:* 20 ^{o}C
Provdiameter: 50 mm
Benämning: Sulfidfläckig varvig lera
Provhöjd: 20 mm
Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v, min}$, m^2/s	k_i, m/s	β_k
38	224	52	14,9	1,3E-8	7,5E-10	2,2

Anm.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\a14_504.txt 2014-05-20

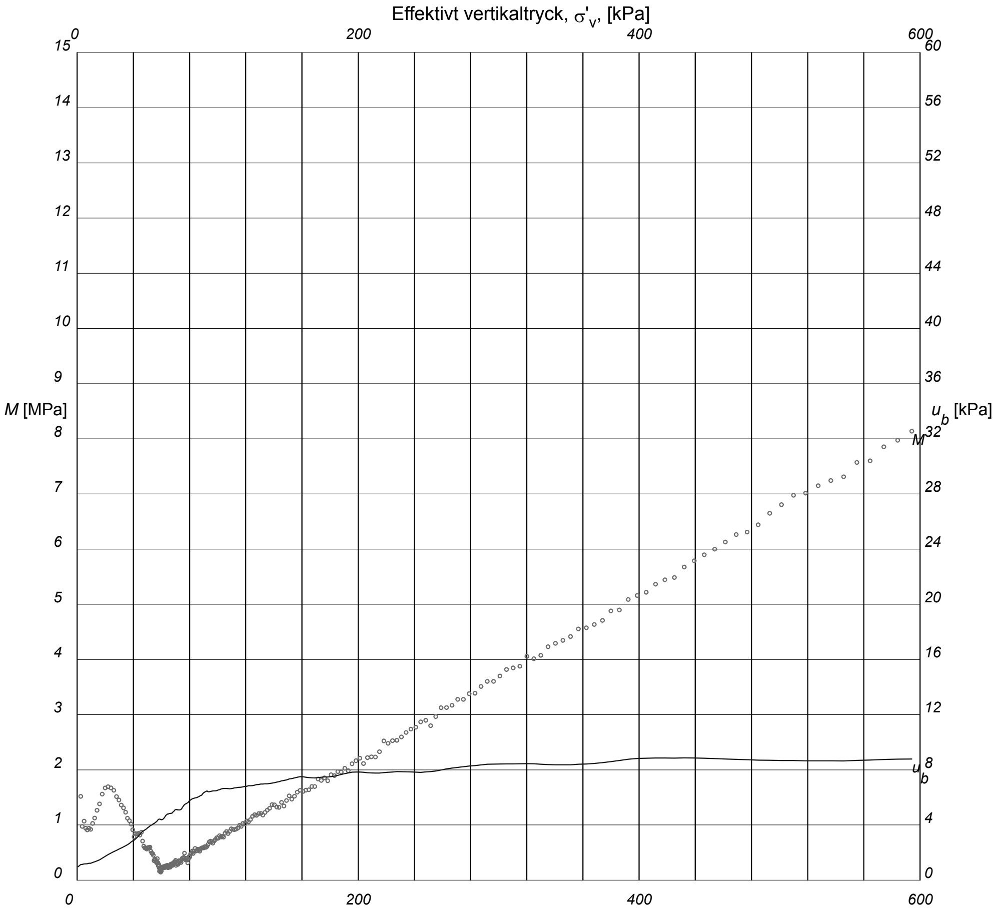
SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**

Uppdragsnummer: 10192148
Uppdragsgivare: WSP Samhällsbyggnad, Stockholm
Datum/Sign: 2014-05-16
Löp-nr/Gransk.: 27005

Sektion/borrhål: 14W44 *Djup:* 2,0 m
Ödometer nr: 3
Densitet: 1,64 t/m^3 *Vattenkvot:* 62 % *Provningstemp.:* 20 ^{o}C
Provdiameter: 50 mm
Benämning: Sulfidfläckig varvig lera
Provhöjd: 20 mm
Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
14,9	52

Anm.

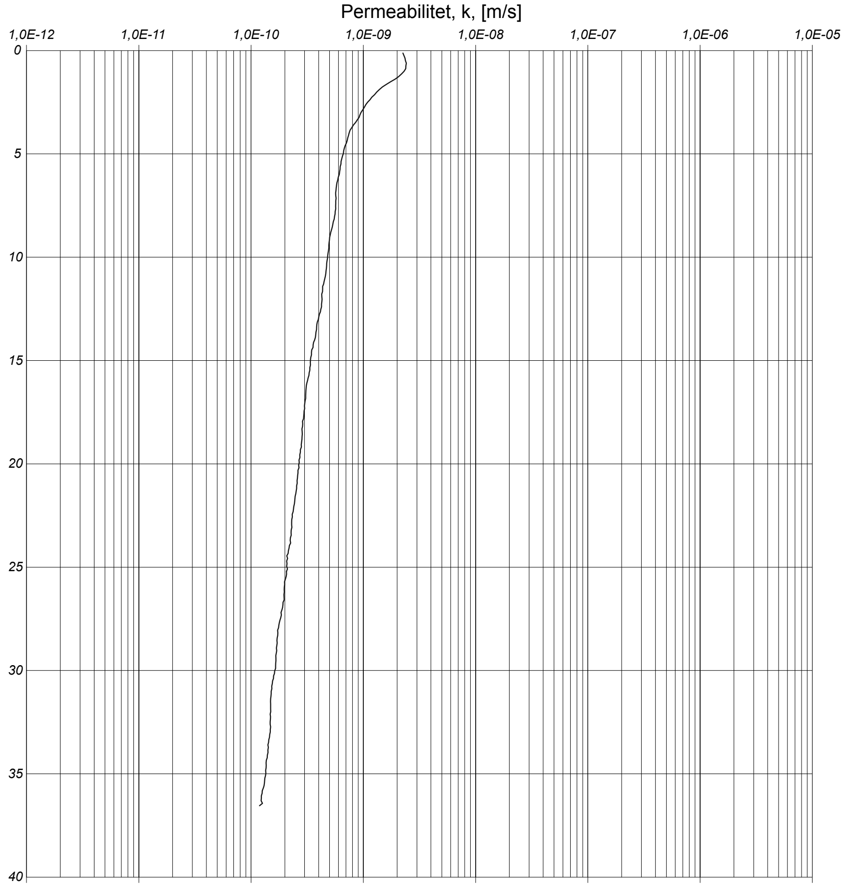
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\a14_504.txt 2014-05-20

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Södra Boo**			
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm		*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44		*Djup:* 2,0 m	*Ödometer nr:* 3
Densitet: 1,64 t/m^3	*Vattenkvot:* 62 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Sulfidfläckig varvig lera			*Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

k_i, m/s	β_k
7,5E-10	2,2

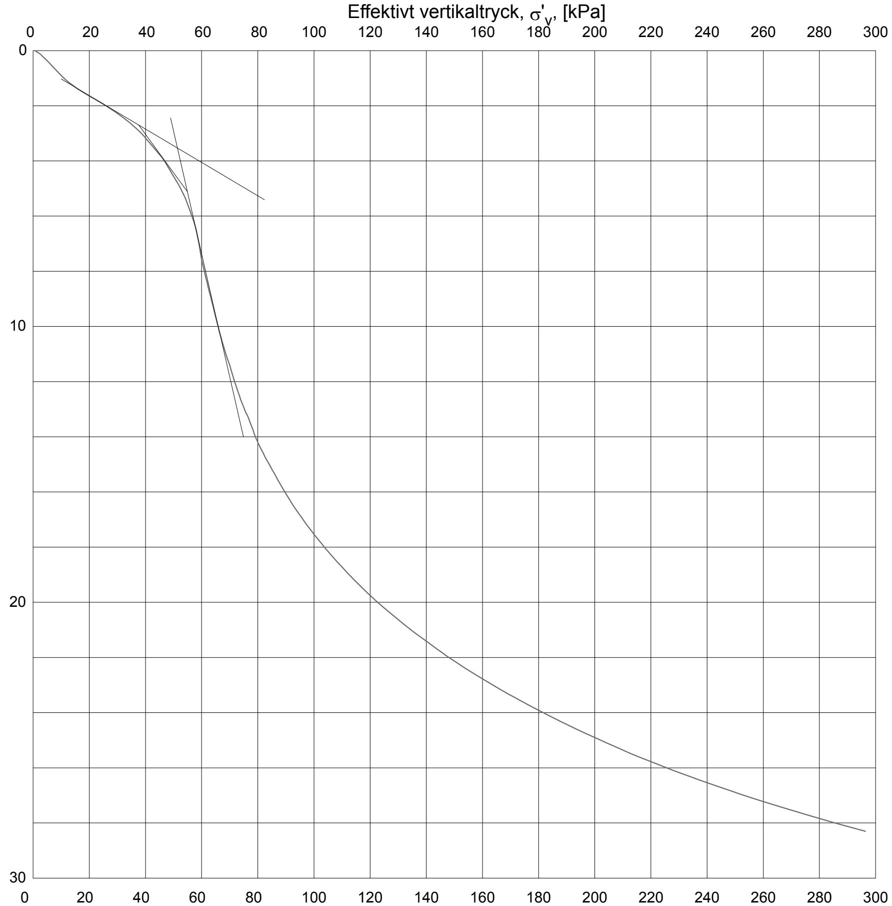
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**			
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm		*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44		*Djup:* 2,0 m	*Ödometer nr:* 3
Densitet: 1,64 t/m^3	*Vattenkvot:* 62 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Sulfidfläckig varvig lera			*Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
38	224	52

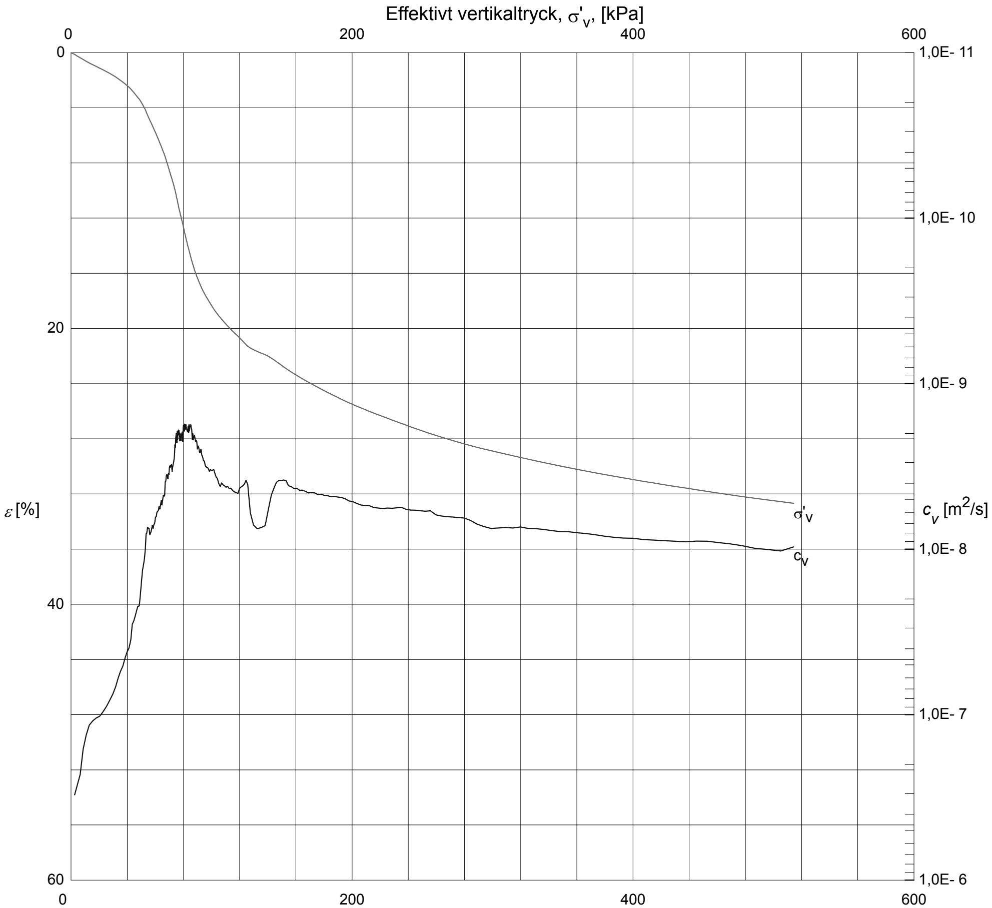
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**			
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm		*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44		*Djup:* 4,0 m	*Ödometer nr:* 1
Densitet: 1,66 t/m^3	*Vattenkvot:* 60 %	*Provningstemp.:* 20 ^oC	*Provdiameter:* 50 mm
Benämning: Sulfidfläckig varvig lera			*Provhöjd:* 20 mm
			Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v, min}$, m^2/s	k_i, m/s	β_k
39	240	83	18,0	1,9E-9	2,4E-10	3,5

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

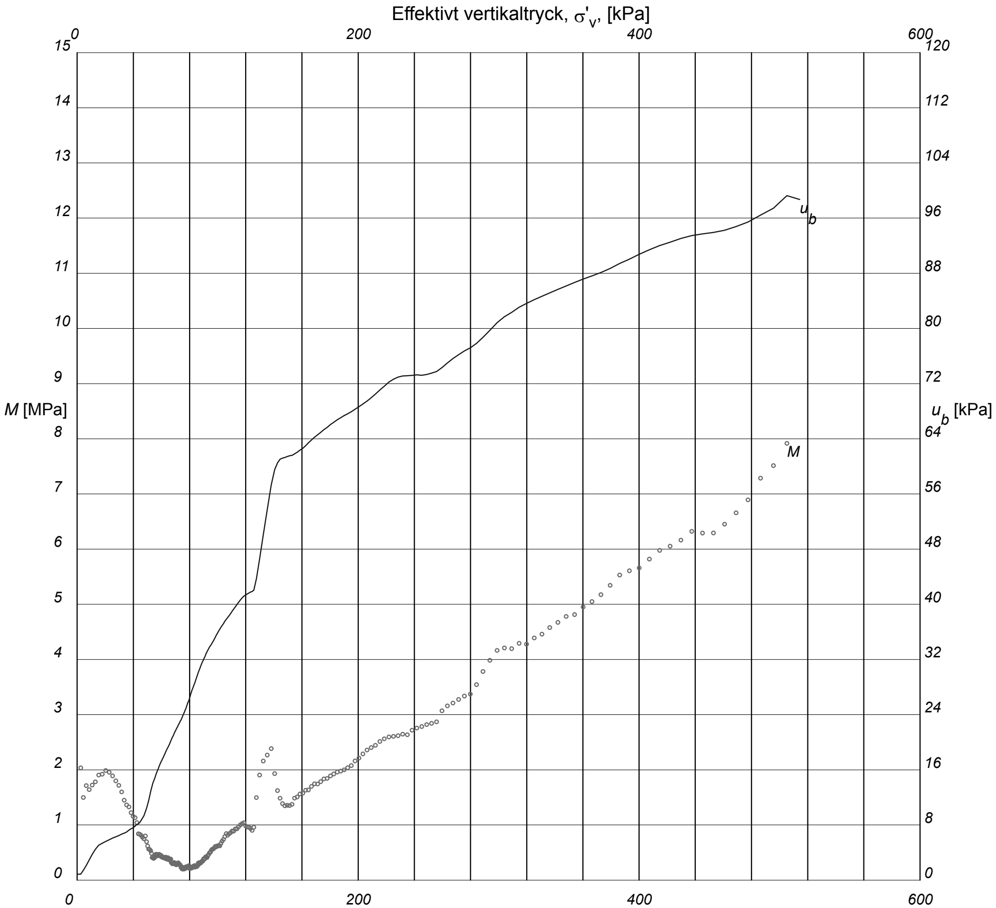
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\b14_520.txt 2014-05-20

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**			
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm		*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44		*Djup:* 4,0 m	*Ödometer nr:* 1
Densitet: 1,66 t/m^3	*Vattenkvot:* 60 %	*Provningstemp.:* 20 ^oC	*Provdiameter:* 50 mm
Benämning: Sulfidfläckig varvig lera			*Provhöjd:* 20 mm
			Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
18,0	83

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\b14_520.txt 2014-05-20

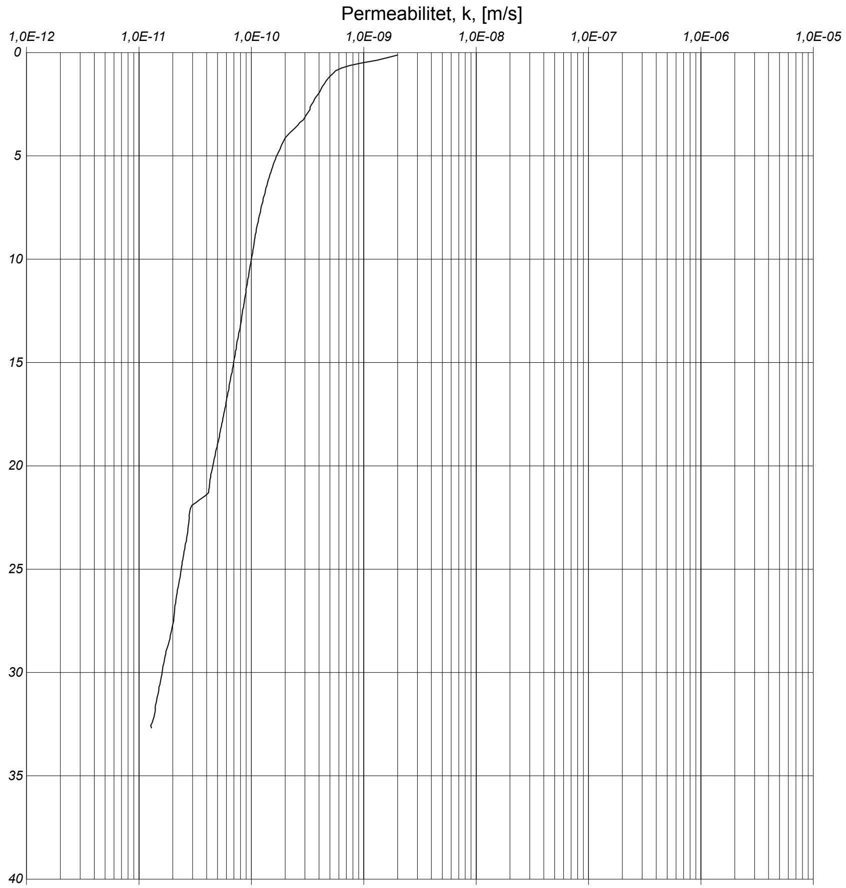
SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Södra Boo**

Uppdragsnummer: 10192148
Uppdragsgivare: WSP Samhällbyggnad, Stockholm
Datum/Sign: 2014-05-16
Löp-nr/Gransk.: 27005

Sektion/borrhål: 14W44 *Djup:* 4,0 m
Densitet: 1,66 t/m^3 *Vattenkvot:* 60 % *Provningstemp.:* 20 ^{o}C
Benämning: Sulfidfläckig varvig lera
Ödometer nr: 1
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,74 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

k_i, m/s	β_k
2,4E-10	3,5

Anm.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\b14_520.txt 2014-05-20

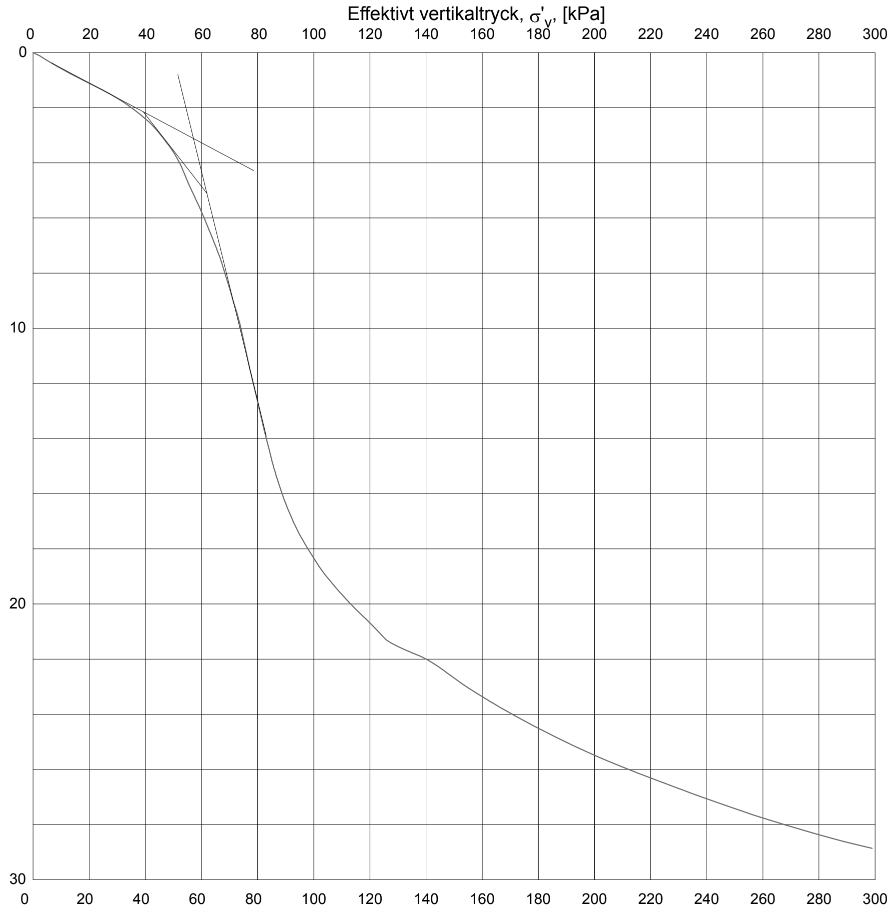
SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**

Uppdragsnummer: 10192148
Uppdragsgivare: WSP Samhällbyggnad, Stockholm
Datum/Sign: 2014-05-16
Löp-nr/Gransk.: 27005

Sektion/borrhål: 14W44 *Djup:* 4,0 m
Densitet: 1,66 t/m^3 *Vattenkvot:* 60 % *Provningstemp.:* 20 ^{o}C
Benämning: Sulfidfläckig varvig lera
Ödometer nr: 1
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,74 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
39	240	83

Anm.

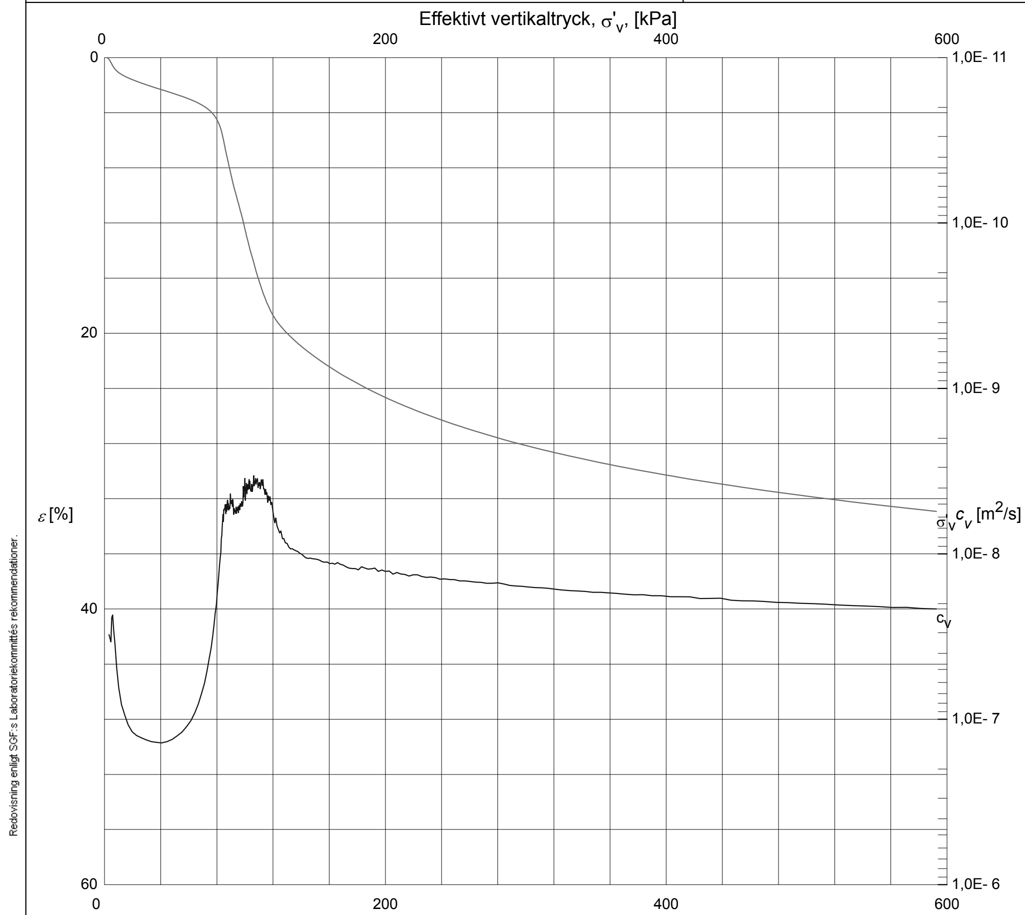
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\b14_520.txt 2014-05-20

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**			
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm		*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44		*Djup:* 6,0 m	*Ödometer nr:* 2
Densitet: 1,66 t/m^3	*Vattenkvot:* 62 %	*Provningstemp.:* 20 ^oC	*Provdiameter:* 50 mm
Benämning: Något sulfidhaltig varvig lera			*Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

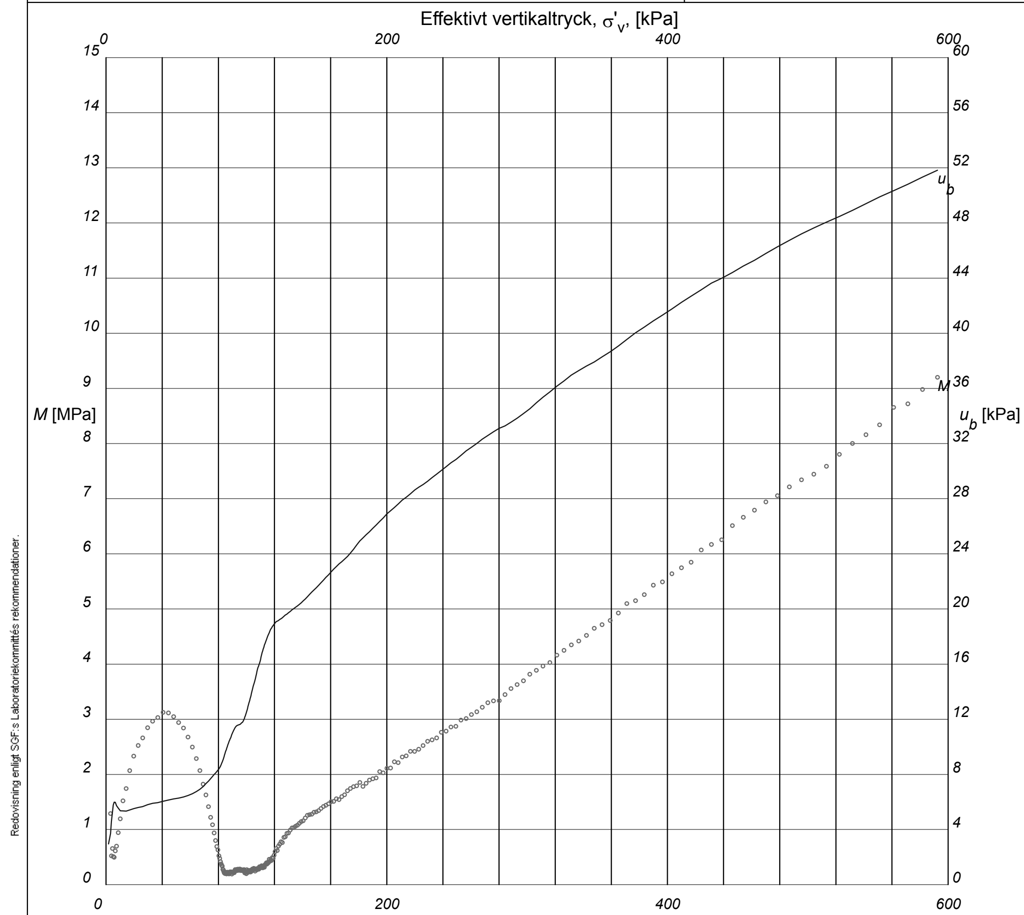
σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v, min}$, m^2/s	k_i, m/s	β_k
70	230	96	20,4	3,9E-9	8,3E-10	4,7

Anm.

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**			
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm		*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44		*Djup:* 6,0 m	*Ödometer nr:* 2
Densitet: 1,66 t/m^3	*Vattenkvot:* 62 %	*Provningstemp.:* 20 ^oC	*Provdiameter:* 50 mm
Benämning: Något sulfidhaltig varvig lera			*Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
20,4	96

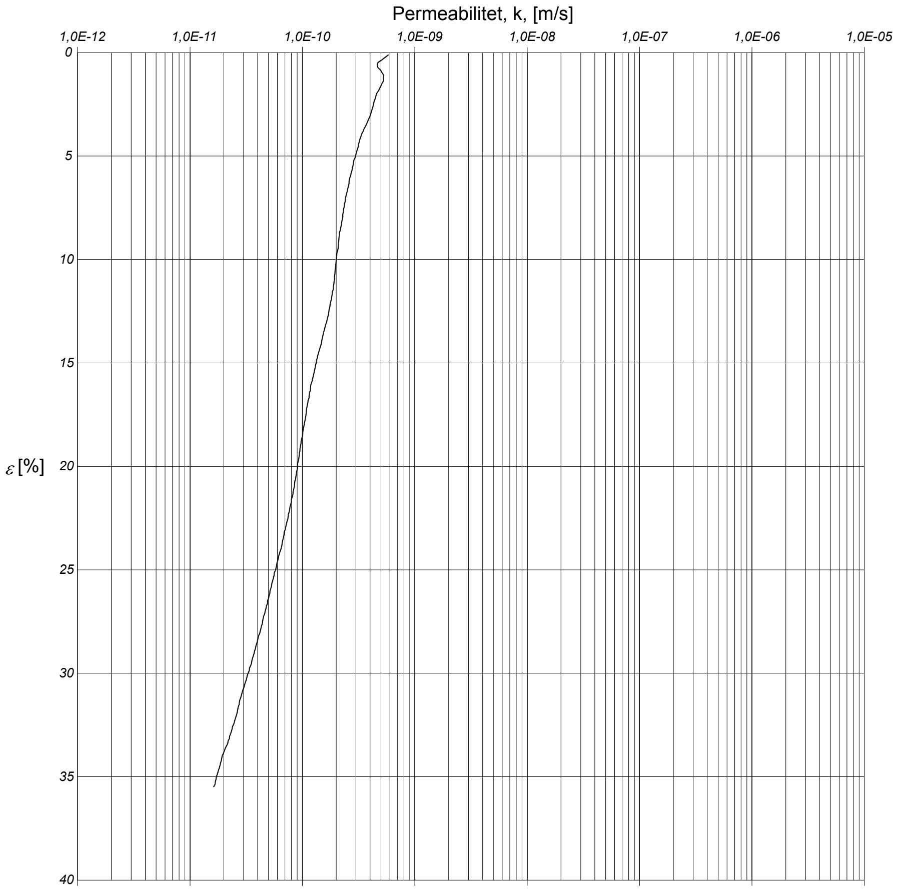
Anm.

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Södra Boo**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2014-05-16
10192148	WSP Samhällbyggnad, Stockholm	*Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44	*Djup:* 6,0 m	*Ödometer nr:* 2
Densitet: 1,66 t/m^3 *Vattenkvot:* 62 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Något sulfidhaltig varvig lera		*Provhöjd:* 20 mm
		Def.hastighet: 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

k_i, m/s	β_k
8,3E-10	4,7

Anm.

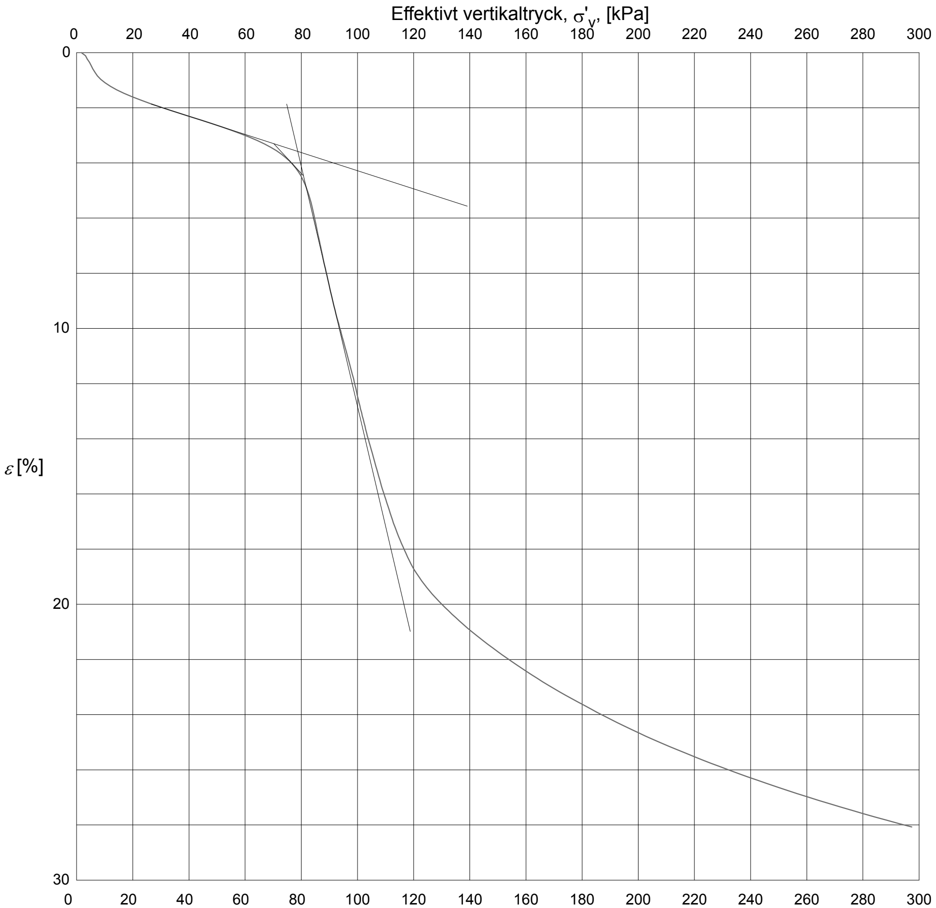
Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2014-05-16
10192148	WSP Samhällbyggnad, Stockholm	*Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44	*Djup:* 6,0 m	*Ödometer nr:* 2
Densitet: 1,66 t/m^3 *Vattenkvot:* 62 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Något sulfidhaltig varvig lera		*Provhöjd:* 20 mm
		Def.hastighet: 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
70	230	96

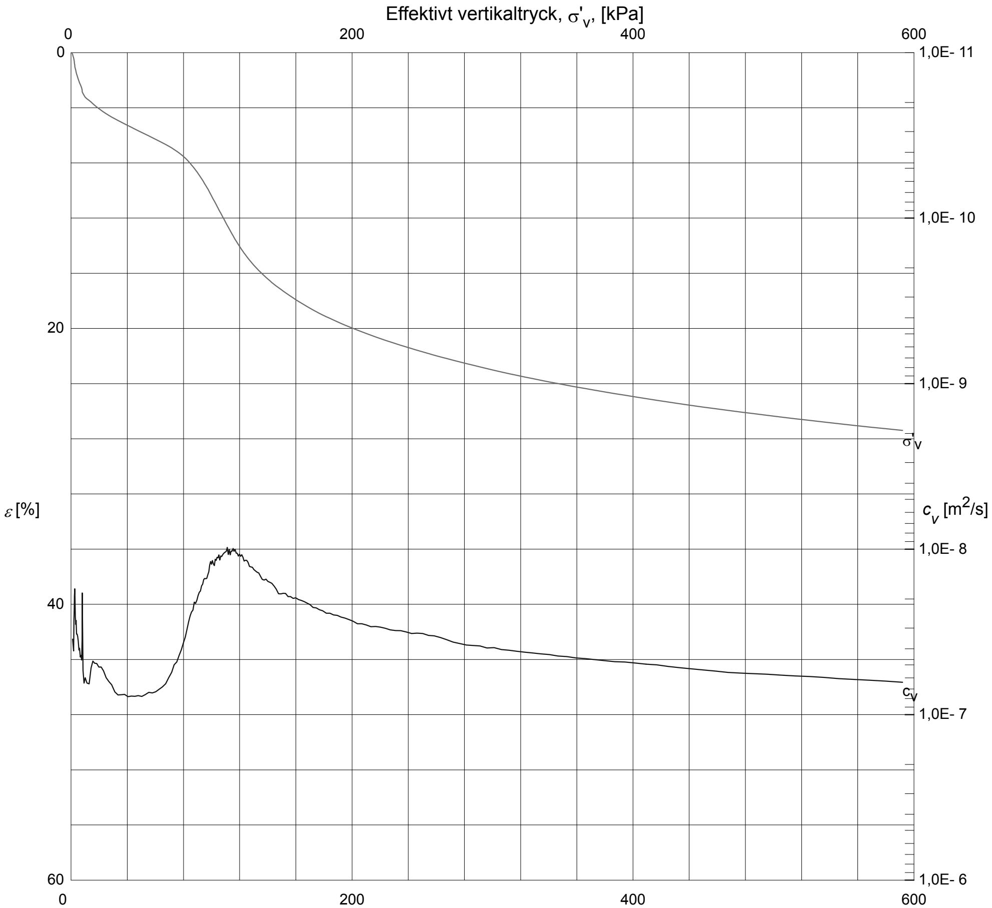
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Södra Boo**		
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm	*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44 *Densitet:* 1,69 t/m^3 *Vattenkvot:* 58 % *Benämning:* Varvig lera	*Djup:* 8,0 m *Provningstemp.:* 20 ^oC	*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\ min}$, m^2/s	k_i, m/s	β_k
75	544	106	20,2	1,0E-8	4,9E-10	3,2

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

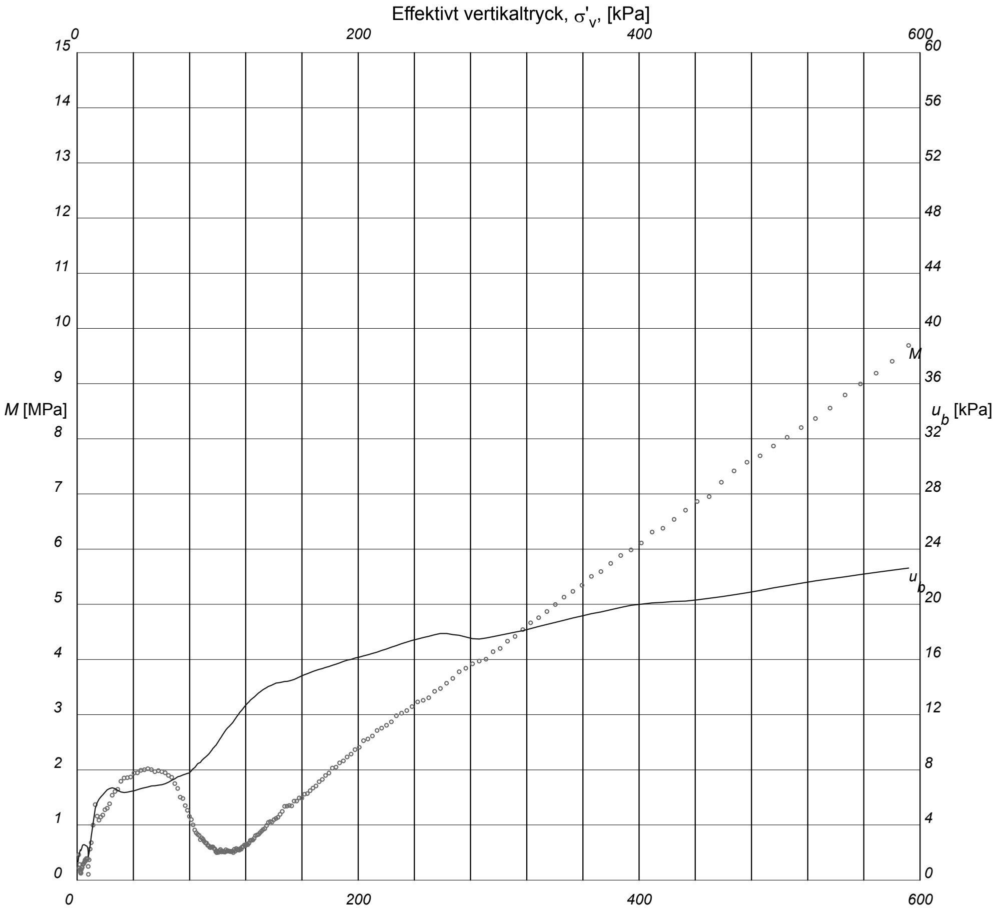
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\b14_522.txt 2014-05-20

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Södra Boo**		
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm	*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44 *Densitet:* 1,69 t/m^3 *Vattenkvot:* 58 % *Benämning:* Varvig lera	*Djup:* 8,0 m *Provningstemp.:* 20 ^oC	*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
20,2	106

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

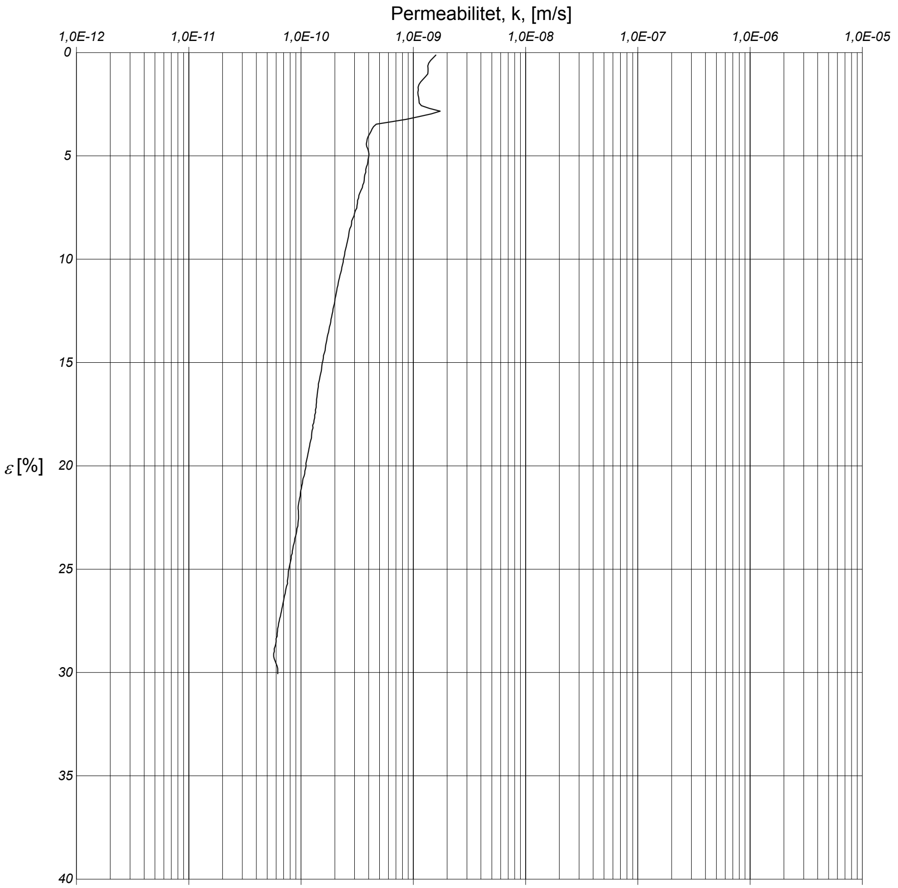
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2014\27005\b14_522.txt 2014-05-20

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Södra Boo**		
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm	*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44 *Densitet:* 1,69 t/m^3 *Vattenkvot:* 58 % *Benämning:* Varvig lera	*Djup:* 8,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{O}C.

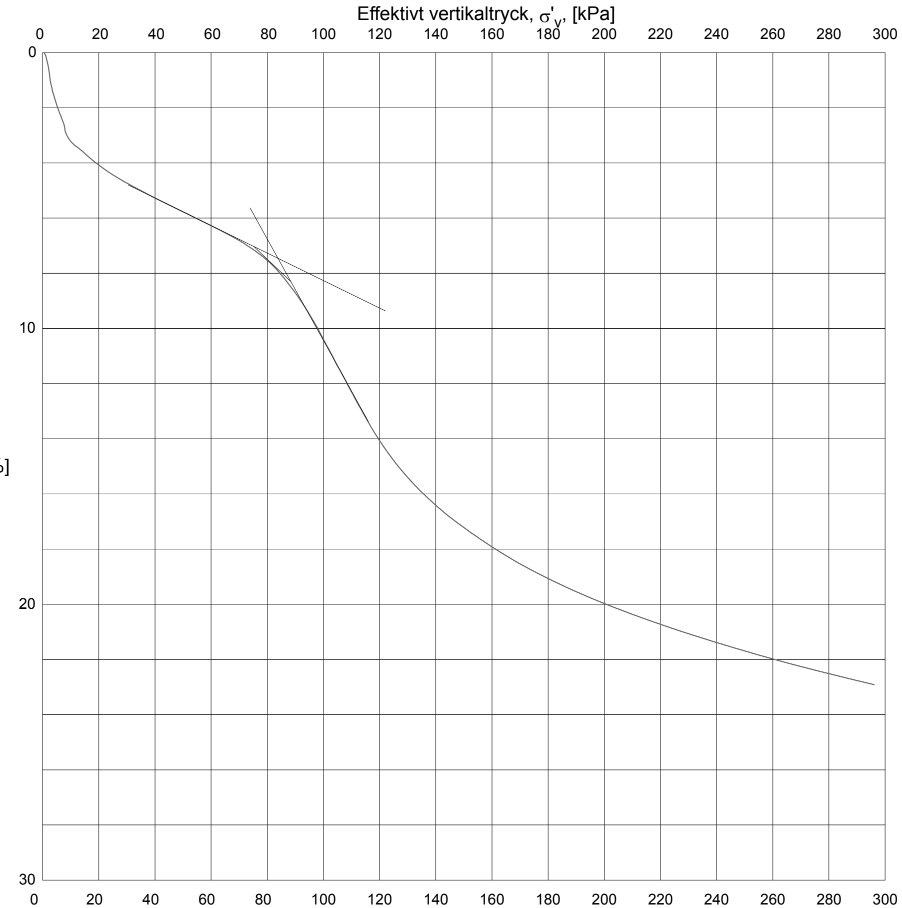
k_i, m/s	β_k
4,9E-10	3,2

Anm.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Södra Boo**		
Uppdragsnummer: 10192148	*Uppdragsgivare:* WSP Samhällbyggnad, Stockholm	*Datum/Sign:* 2014-05-16 *Löp-nr/Gransk.:* 27005
Sektion/borrhål: 14W44 *Densitet:* 1,69 t/m^3 *Vattenkvot:* 58 % *Benämning:* Varvig lera	*Djup:* 8,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

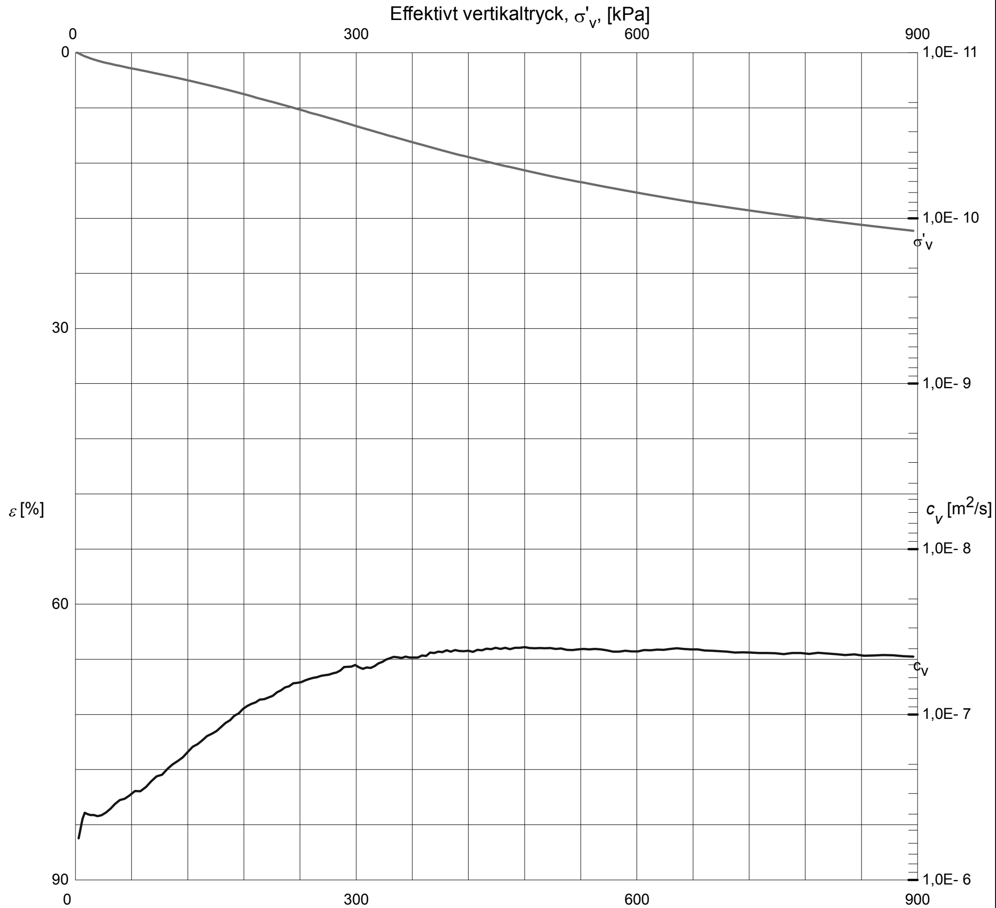
Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
75	544	106

Anm.

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 Båker *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Densitet:* 1,76 t/m^3 *Benämning:* Rostfläckig varvig lera	*Djup:* 2,0 m *Vattenkvot:* 51 % *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 4 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,71 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v, min}$, m^2/s	k_i, m/s	β_k
130	3343	314	10,2	4,0E-8	3,4E-10	4,4

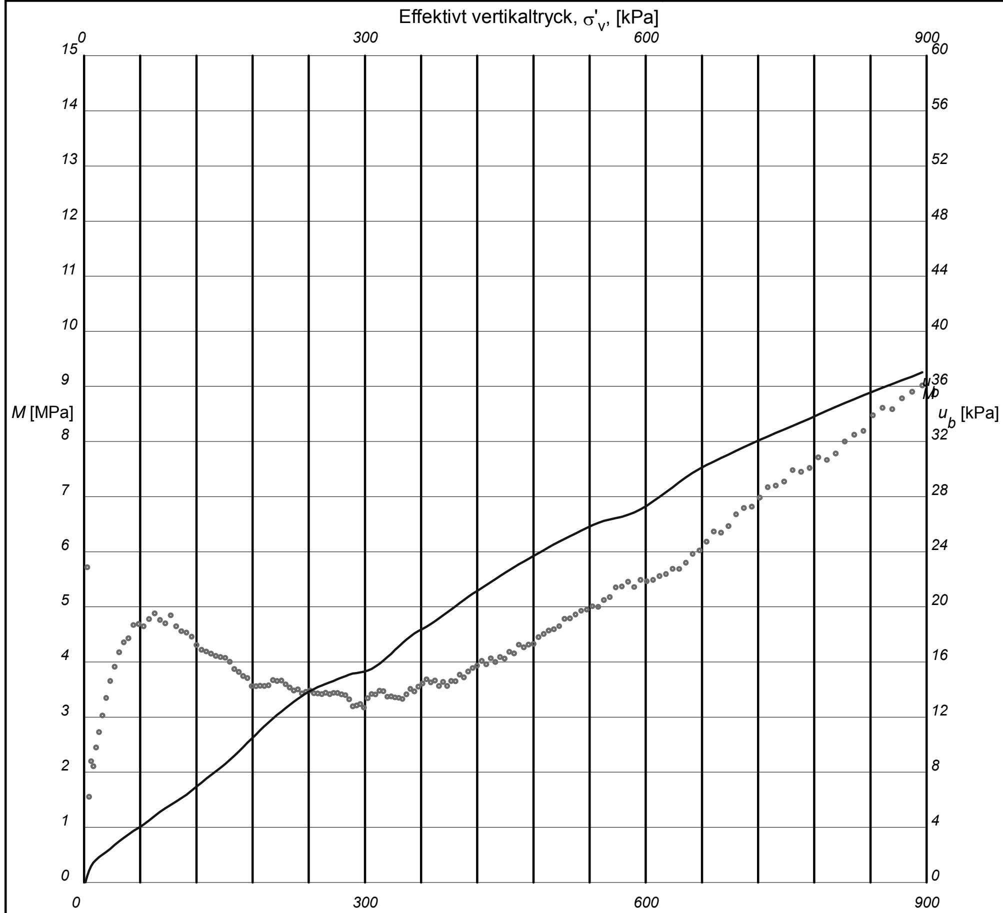
Anm.

Skalan i diagrammet avviker från den av SGF:s Laboratoriekommité satta rekommendation.

SWECO

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Densitet:* 1,76 t/m^3 *Benämning:* Rostfläckig varvig lera	*Djup:* 2,0 m *Vattenkvot:* 51 % *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 4 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,71 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

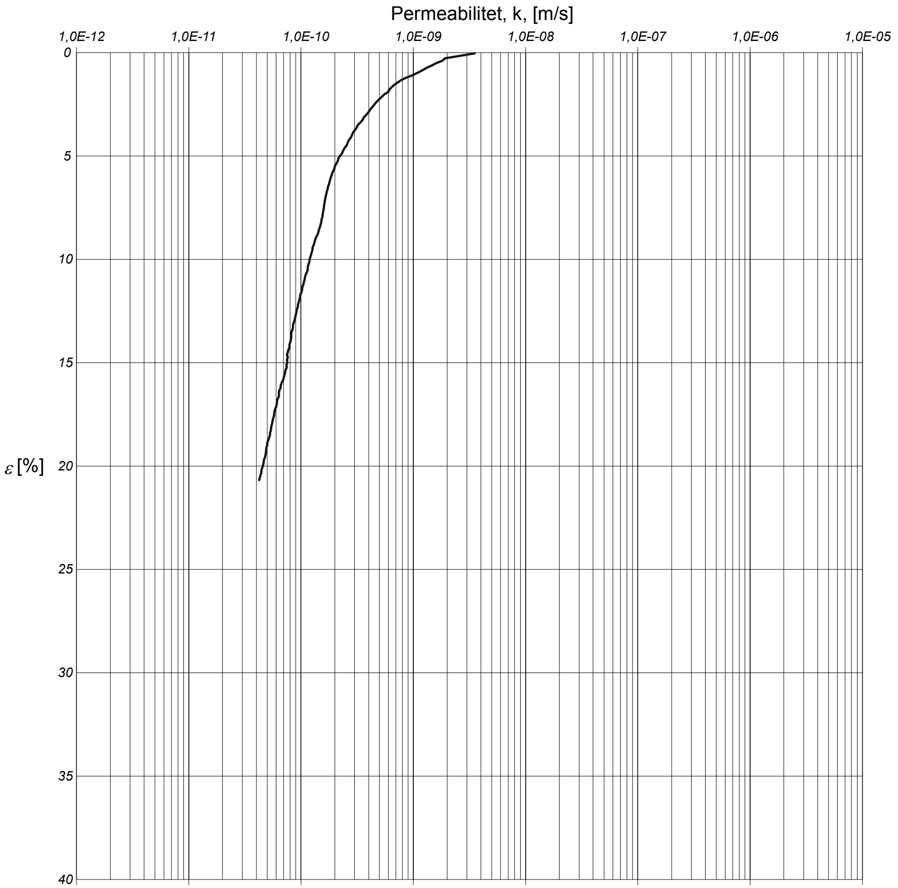
M'	σ'_L, kPa
10,2	314

Anm.

SWECO

Utvärdering av permeabilitet

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Densitet:* 1,76 t/m³ *Vattenkvot:* 51 % *Benämning:* Rostfläckig varvig lera	*Djup:* 2,0 m *Provningstemp.:* 20 °C	*Ödometer nr:* 4 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,71 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 °C.

k_i, m/s	β_k
3,4E-10	4,4

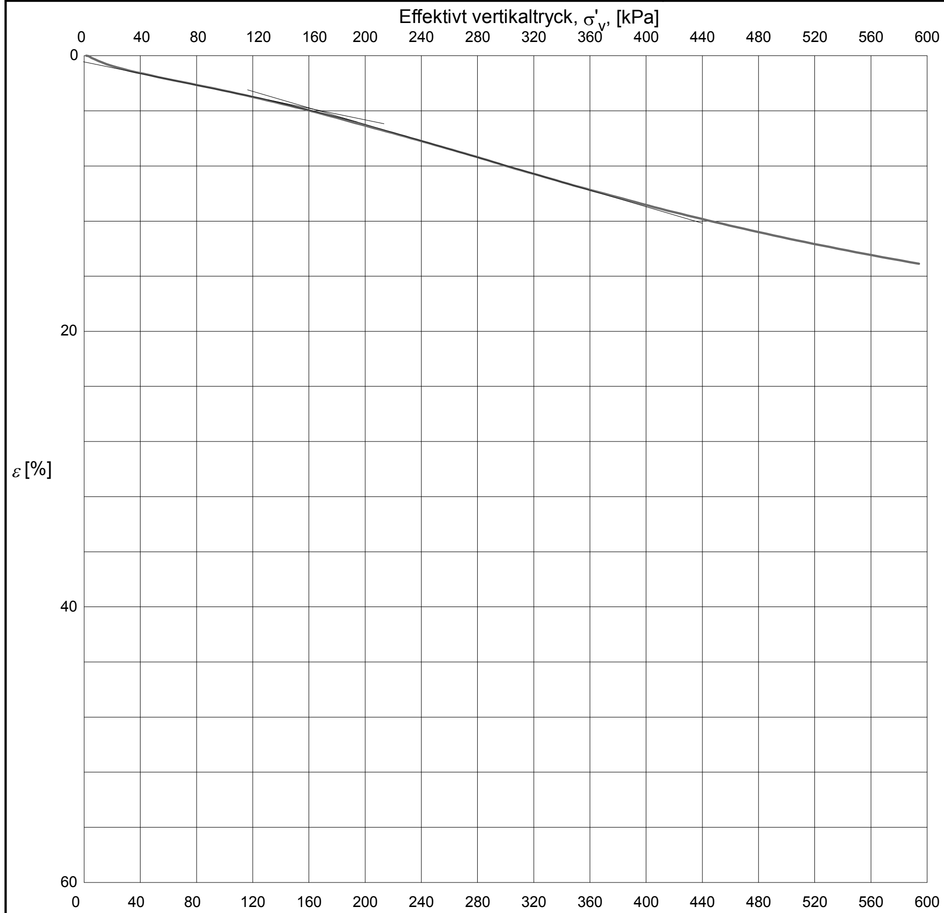
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Densitet:* 1,76 t/m³ *Vattenkvot:* 51 % *Benämning:* Rostfläckig varvig lera	*Djup:* 2,0 m *Provningstemp.:* 20 °C	*Ödometer nr:* 4 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,71 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
130	3343	314

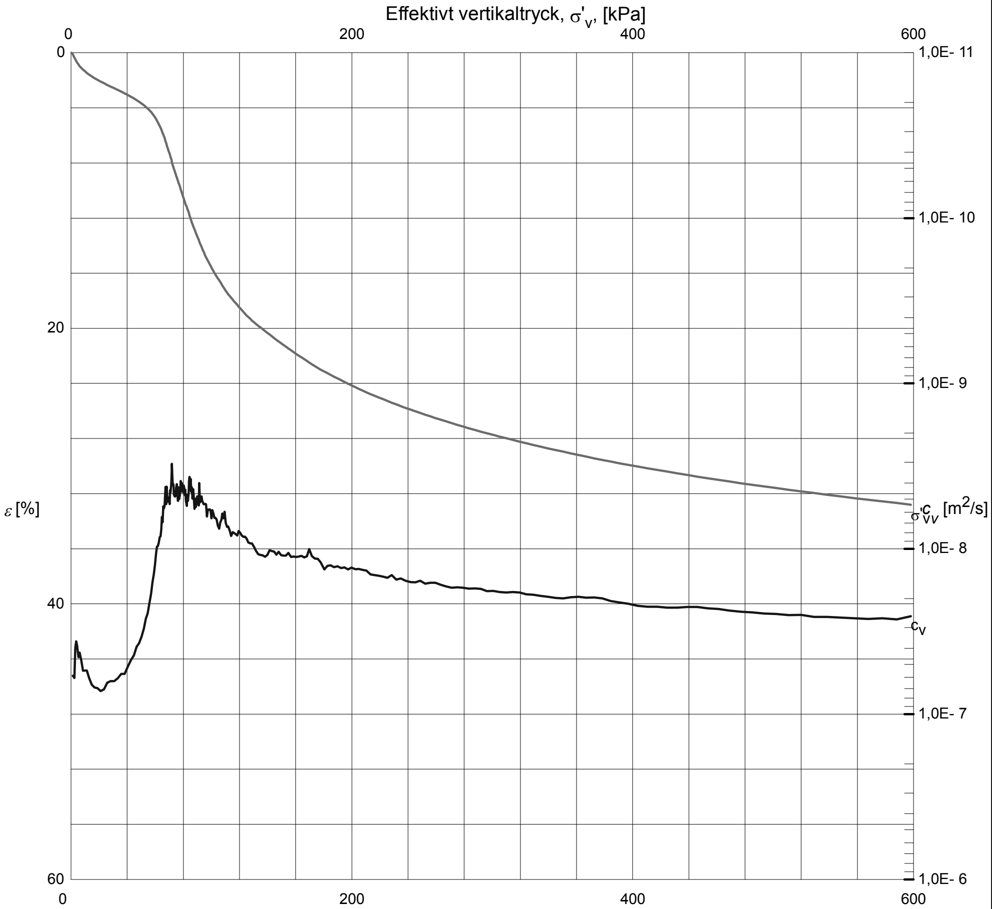
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 Båke *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Djup:* 3,0 m *Densitet:* 1,67 t/m^3 *Vattenkvot:* 61 % *Provningstemp.:* 20 ^oC *Benämning:* Något sulfidbandad varvig lera		*Ödometer nr:* 5 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\,min}$, m^2/s	k_i, m/s	β_k
52	313	73	16,2	4,2E-9	3,4E-10	3,0

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

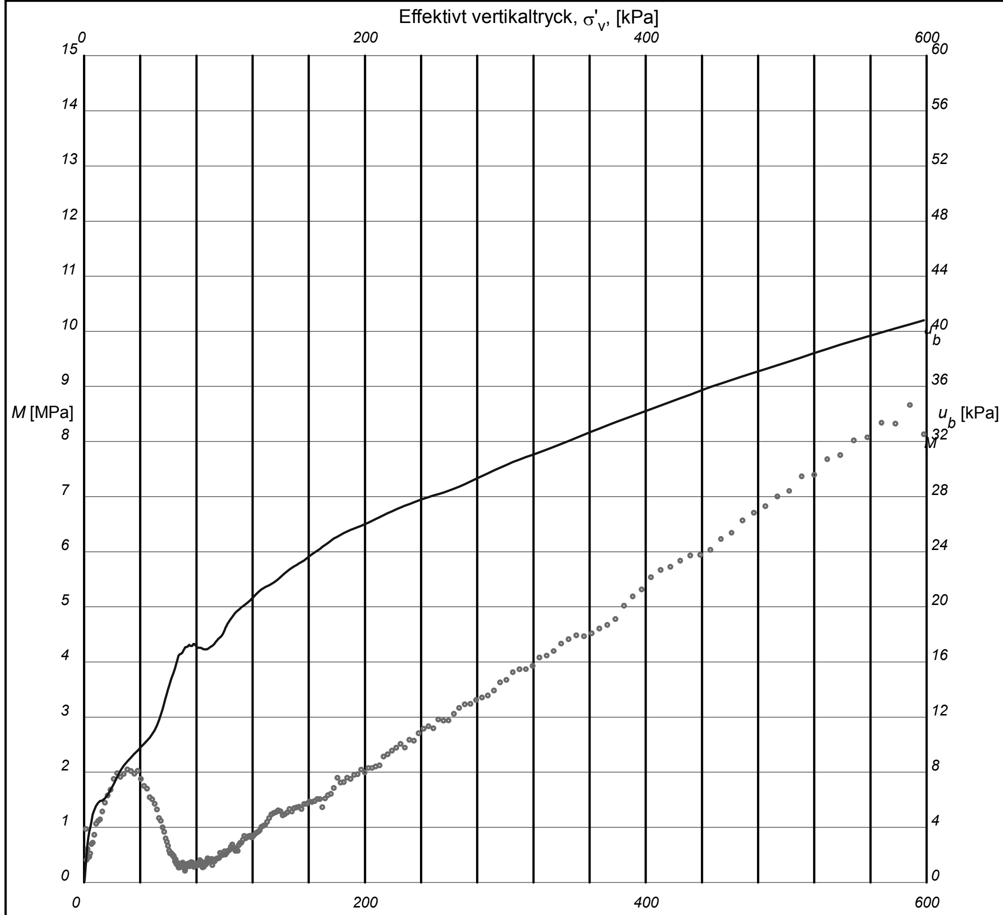
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2016\30056\a16_476.txt 2016-05-25

SWECO

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Djup:* 3,0 m *Densitet:* 1,67 t/m^3 *Vattenkvot:* 61 % *Provningstemp.:* 20 ^oC *Benämning:* Något sulfidbandad varvig lera		*Ödometer nr:* 5 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
16,2	73

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

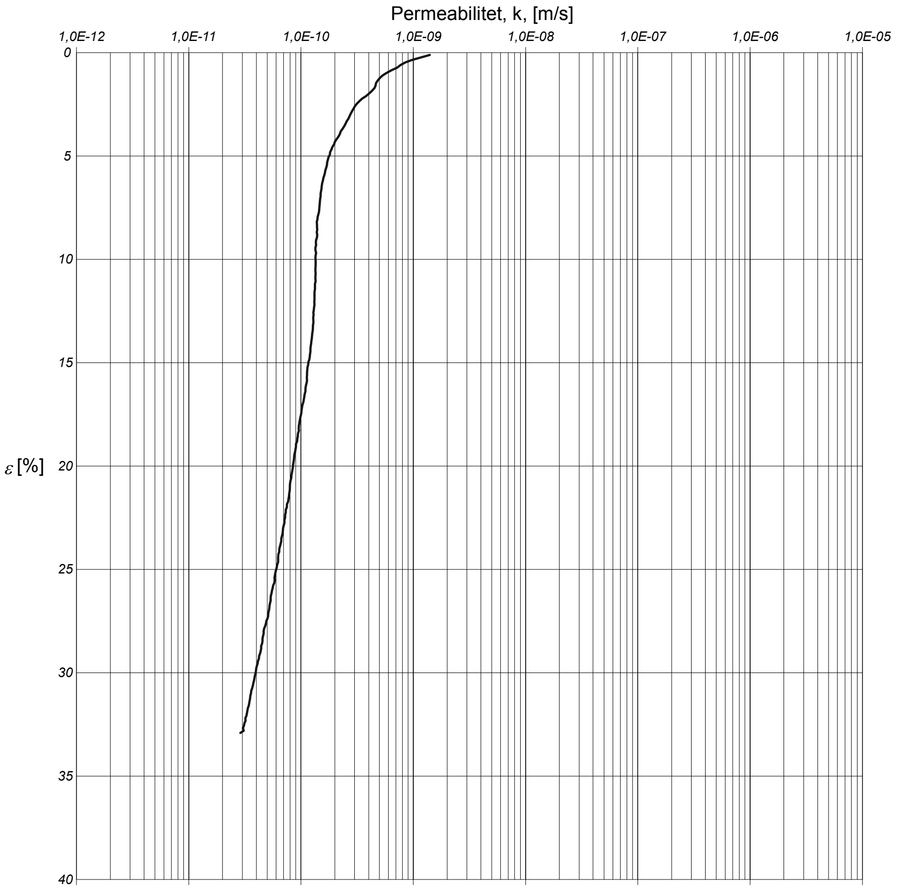
SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 Stockholm, Tel 08-695 60 00, Fax 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, Ingår i SWECO Civil AB

P:\2172\Uppdrag 2016\30056\a16_476.txt 2016-05-25

SWECO

Utvärdering av permeabilitet

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Djup:* 3,0 m *Densitet:* 1,67 t/m^3 *Vattenkvot:* 61 % *Provningstemp.:* 20 ^oC *Benämning:* Något sulfidbandad varvig lera		*Ödometer nr:* 5 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC.

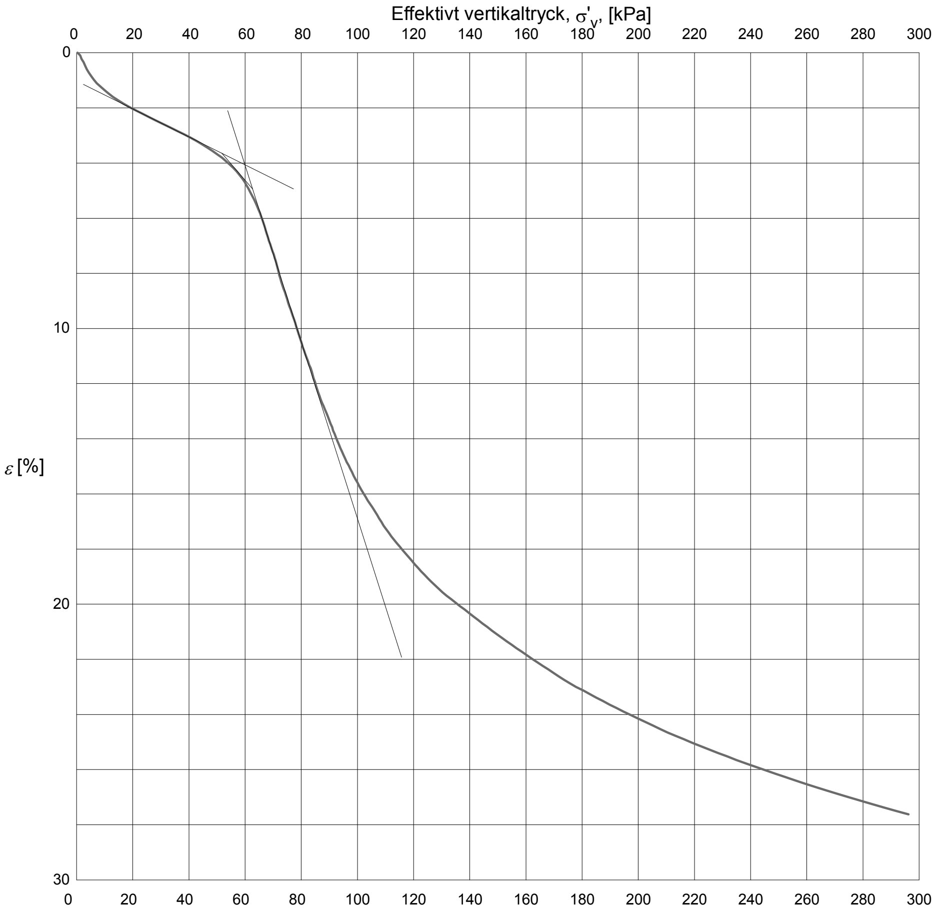
k_i, m/s	β_k
3,4E-10	3,0

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Djup:* 3,0 m *Densitet:* 1,67 t/m^3 *Vattenkvot:* 61 % *Provningstemp.:* 20 ^oC *Benämning:* Något sulfidbandad varvig lera		*Ödometer nr:* 5 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
52	313	73

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

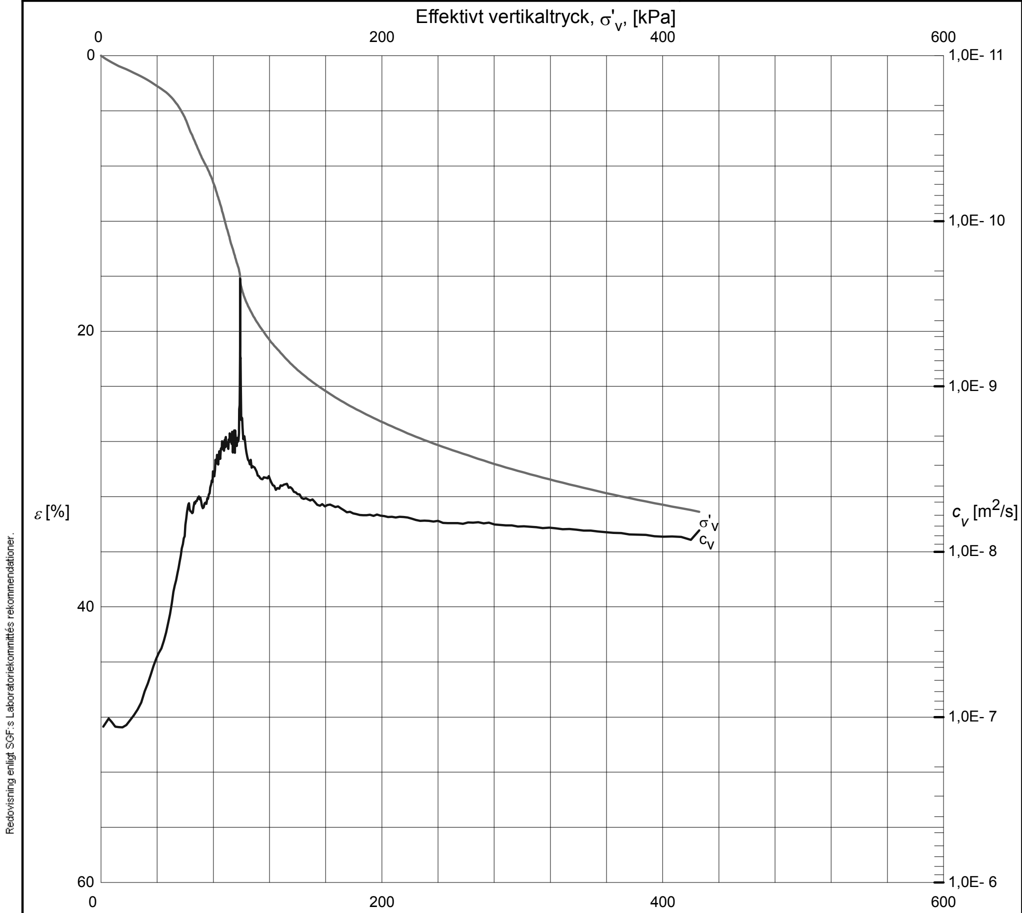
Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**

Uppdragsnummer:	Uppdragsgivare:	Datum/Sign: 2016-05-23
16U29452	Bjerking AB, Stockholm	Löp-nr/Gransk.: 30056

Sektion/borrhål: 16B40 *Djup:* 5,0 m
Densitet: 1,68 t/m^3 *Vattenkvot:* 54 % *Provningstemp.:* 20 ^{o}C
Benämning: Ngt sulfidb varvig lera m enst my tunna finsandssk

Ödometer nr: 1
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\,min}$, m^2/s	k_i, m/s	β_k
44	252	77	16,2	2,0E-9	2,7E-10	3,7

Anm.

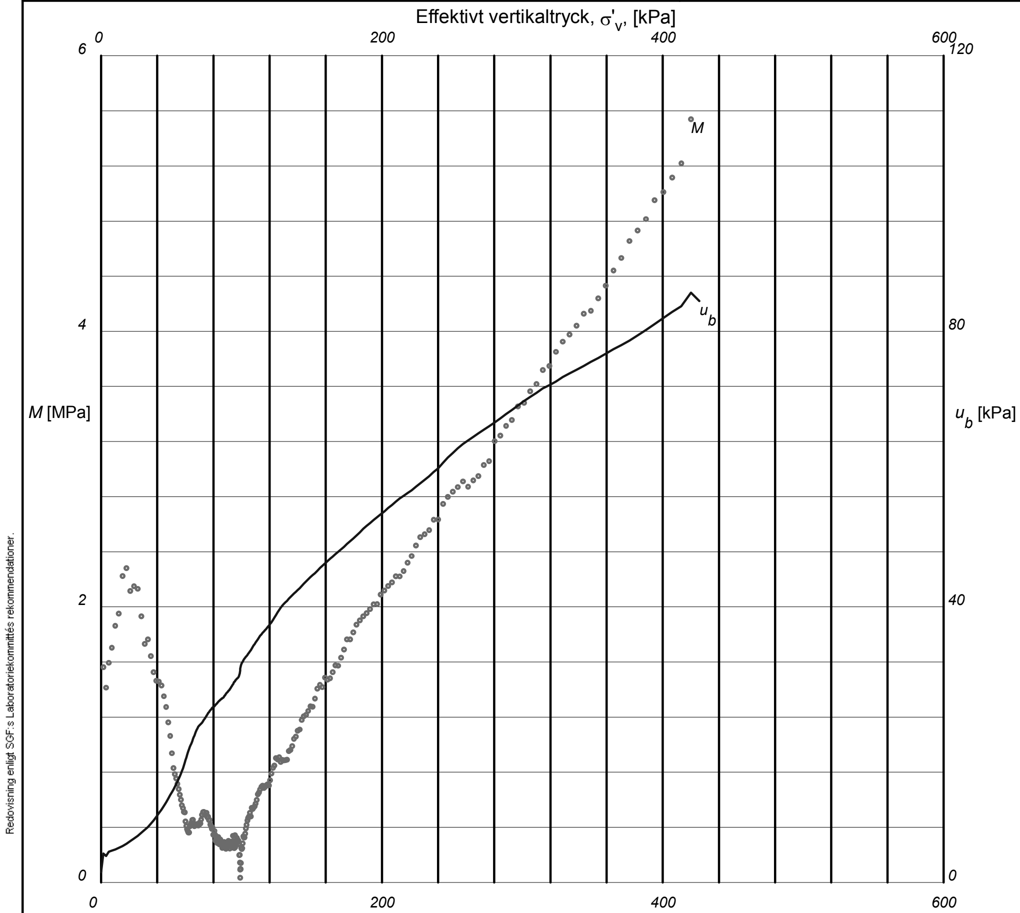
Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**

Uppdragsnummer:	Uppdragsgivare:	Datum/Sign: 2016-05-23
16U29452	Bjerking AB, Stockholm	Löp-nr/Gransk.: 30056

Sektion/borrhål: 16B40 *Djup:* 5,0 m
Densitet: 1,68 t/m^3 *Vattenkvot:* 54 % *Provningstemp.:* 20 ^{o}C
Benämning: Ngt sulfidb varvig lera m enst my tunna finsandssk

Ödometer nr: 1
Provdiameter: 50 mm
Provhöjd: 20 mm
Def.hastighet: 0,74 %/h



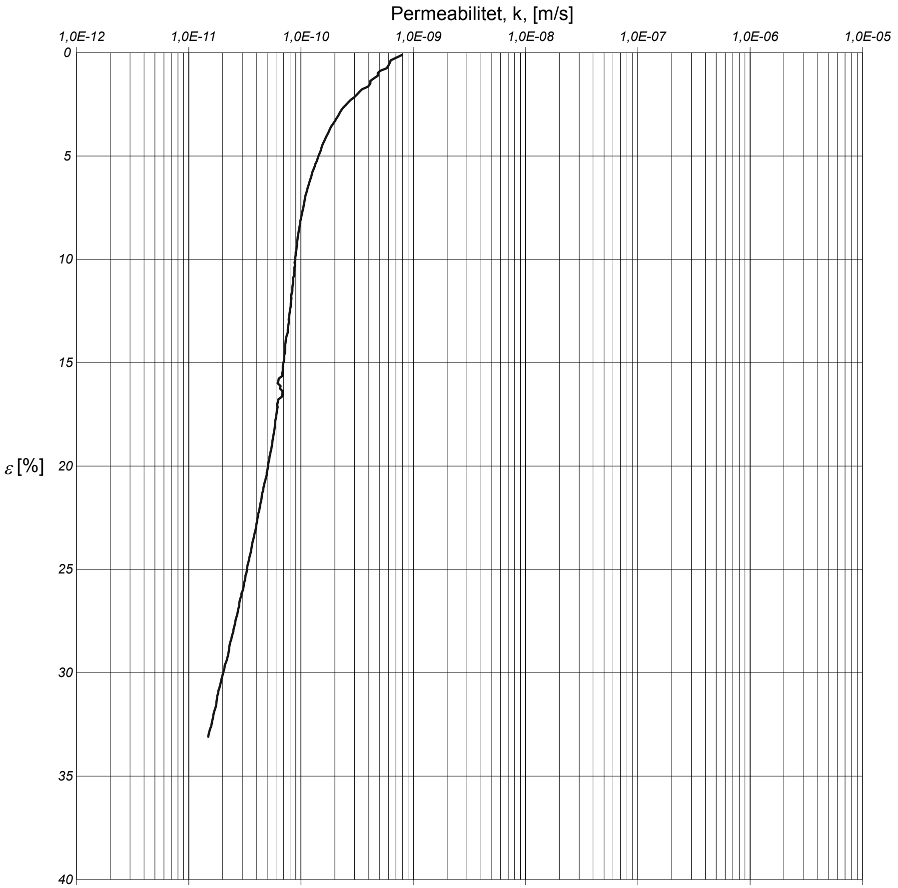
Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
16,2	77

Anm.

Utvärdering av permeabilitet

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Densitet:* 1,68 t/m^3 *Vattenkvot:* 54 % *Benämning:* Ngt sulfidb varvig lera m enst my tunna finsandssk	*Djup:* 5,0 m *Provningstemp.:* 20 ^oC	*Ödometer nr:* 1 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC.

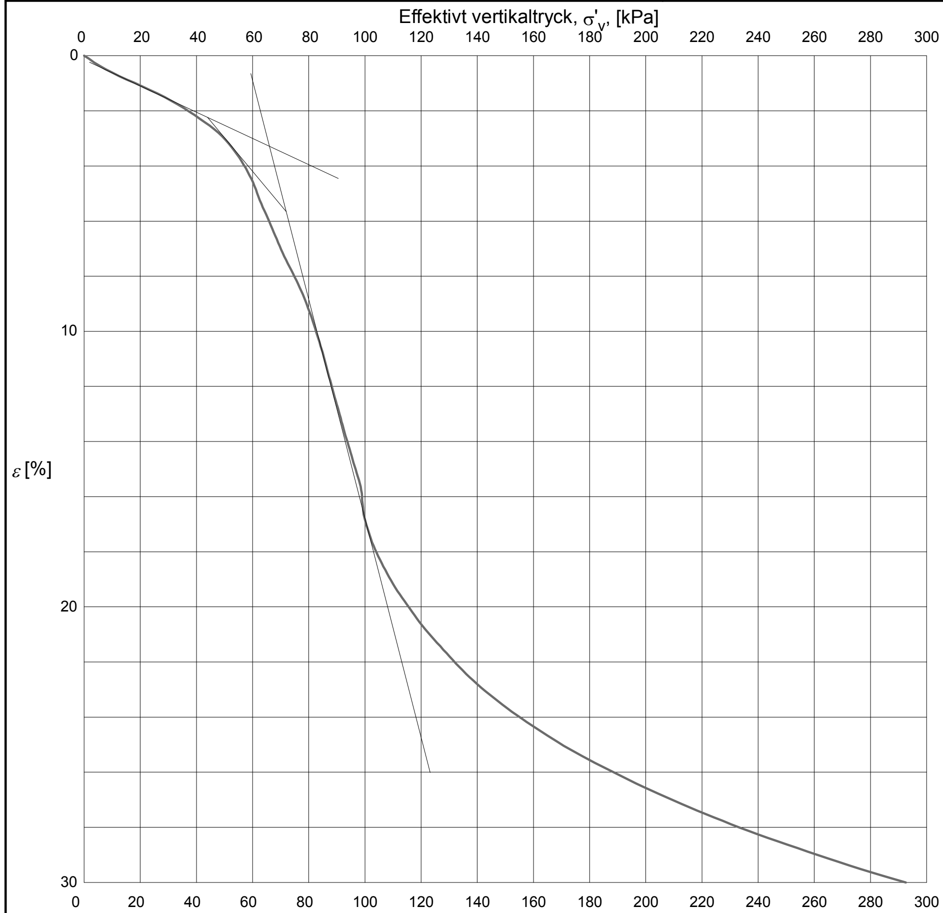
k_i, m/s	β_k
2,7E-10	3,7

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B40 *Densitet:* 1,68 t/m^3 *Vattenkvot:* 54 % *Benämning:* Ngt sulfidb varvig lera m enst my tunna finsandssk	*Djup:* 5,0 m *Provningstemp.:* 20 ^oC	*Ödometer nr:* 1 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

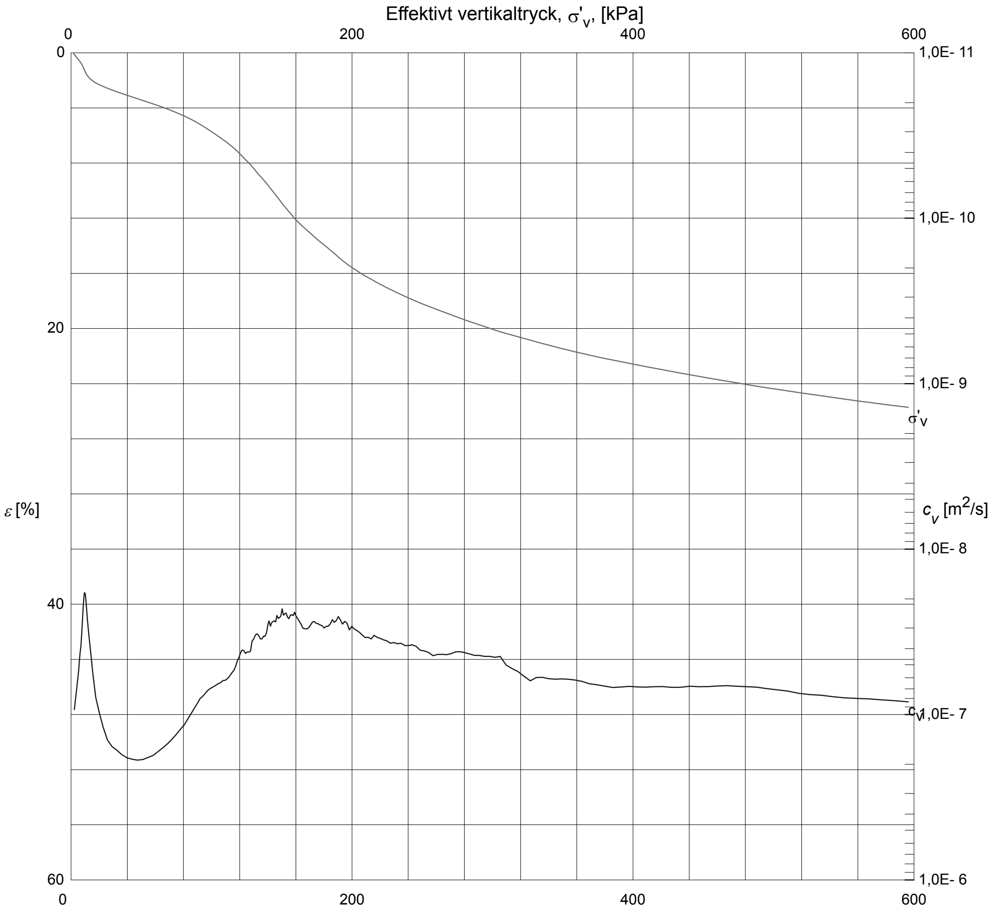
σ'_c, kPa	M_L, kPa	σ'_L, kPa
44	252	77

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46 *Djup:* 2,5 m *Densitet:* 1,75 t/m^3 *Vattenkvot:* 51 % *Provningstemp.:* 20 ^{o}C *Benämning:* Något sulfidbandad varvig lera, skredtecken		*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och *k* har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

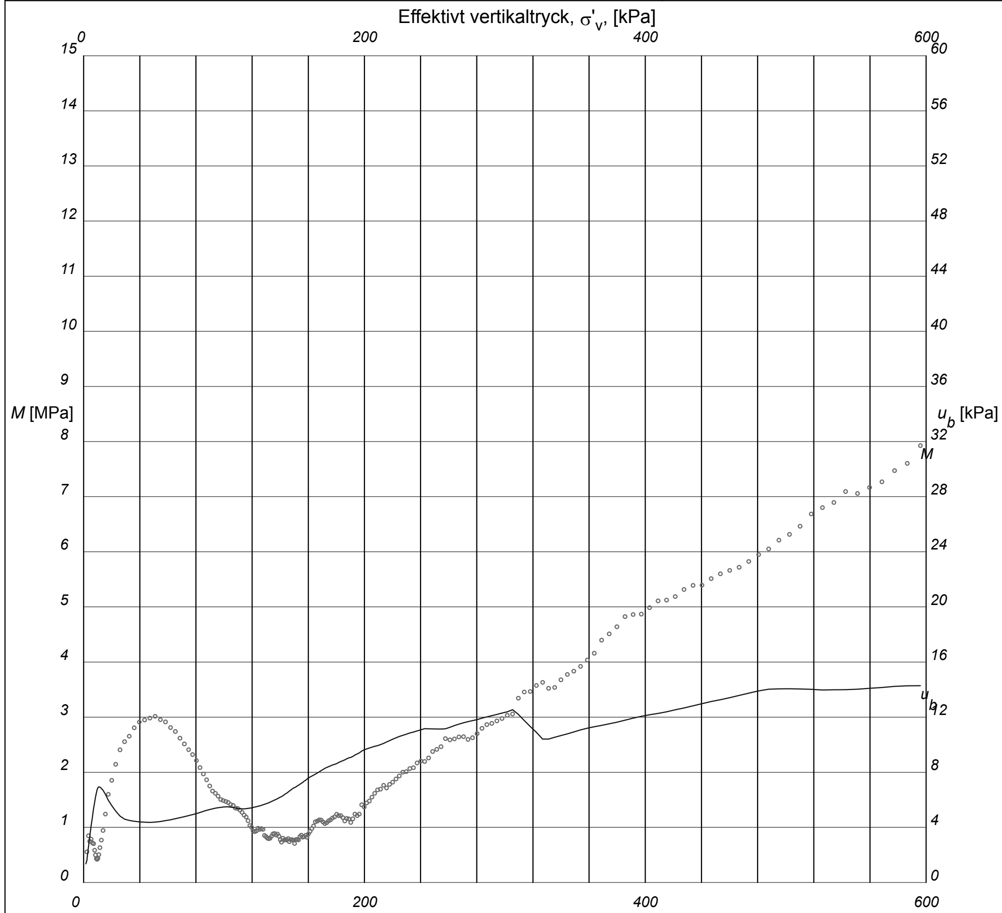
σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\,min}$, m^2/s	k_i, m/s	β_k
83	802	135	16,4	2,6E-8	7,8E-10	3,4

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46 *Djup:* 2,5 m *Densitet:* 1,75 t/m^3 *Vattenkvot:* 51 % *Provningstemp.:* 20 ^{o}C *Benämning:* Något sulfidbandad varvig lera, skredtecken		*Ödometer nr:* 3 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

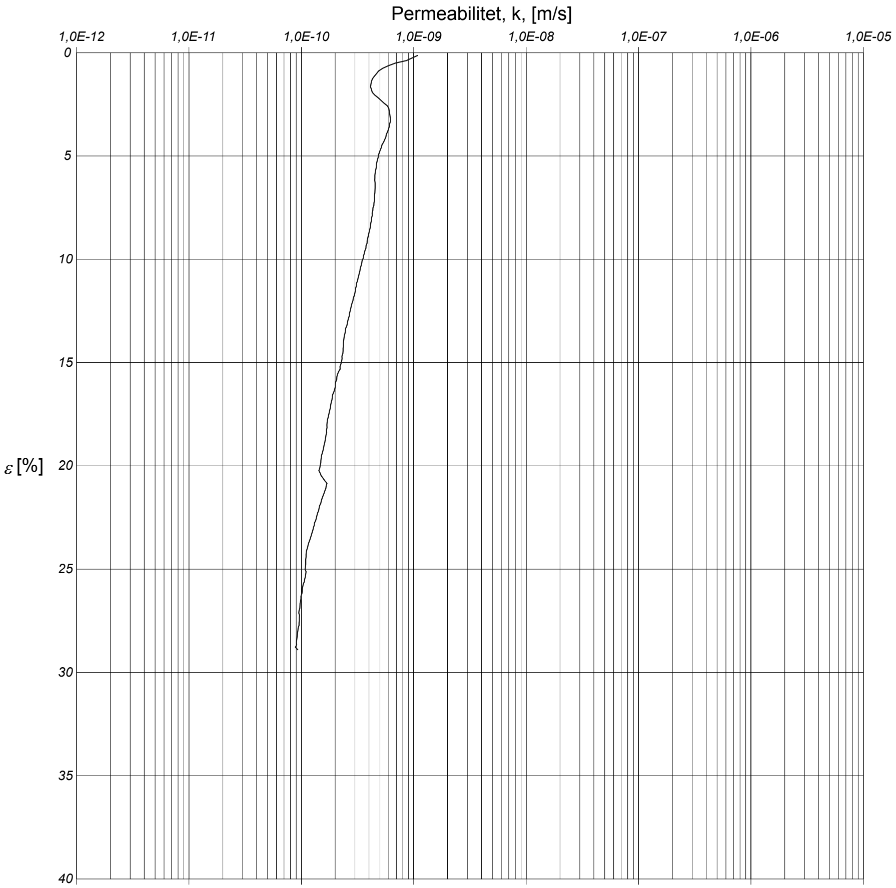
M'	σ'_L, kPa
16,4	135

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Utvärdering av permeabilitet

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46 *Densitet:* 1,75 t/m^3 *Vattenkvot:* 51 %	*Djup:* 2,5 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 3 *Provdiameter:* 50 mm
Benämning: Något sulfidbandad varvig lera, skredtecken		*Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



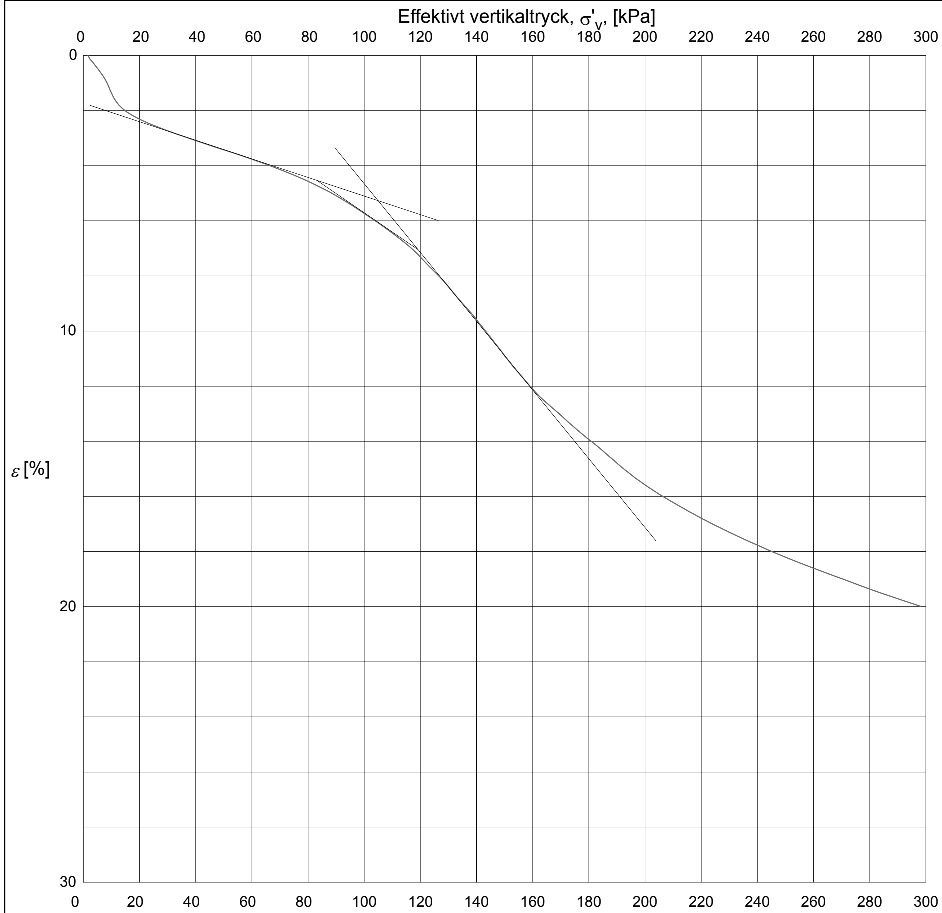
Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

k_i, m/s	β_k
7,8E-10	3,4

Anm.

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-23 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46 *Densitet:* 1,75 t/m^3 *Vattenkvot:* 51 %	*Djup:* 2,5 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 3 *Provdiameter:* 50 mm
Benämning: Något sulfidbandad varvig lera, skredtecken		*Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



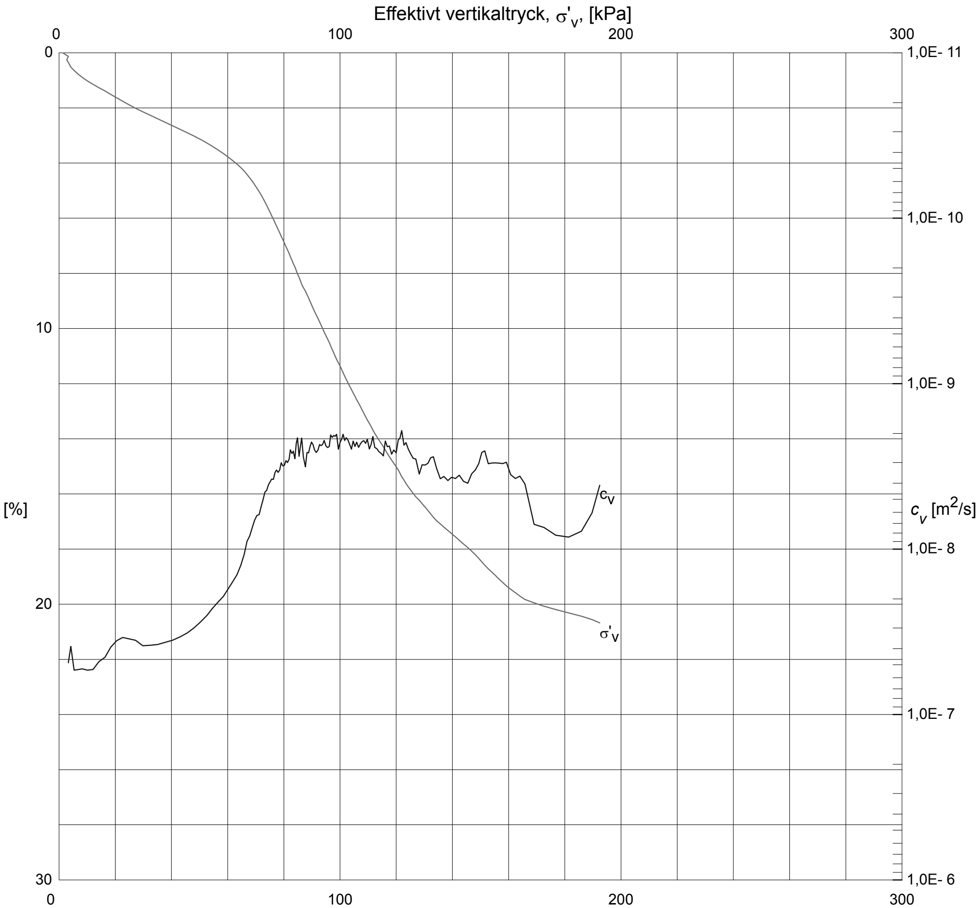
Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
83	802	135

Anm.

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	Uppdragsgivare: Bjerking AB, Stockholm	Datum/Sign: 2016-05-24 Löp-nr/Gransk.: 30056
Sektion/borrhål: 16B46 Djup: 4,0 m Densitet: 1,69 t/m^3 Vattenkvot: 56 % Provningstemp.: 20 ^oC Benämning: Något sulfidbandad varvig lera		Ödometer nr: 2 Provdiameter: 50 mm Provhöjd: 20 mm Def.hastighet: 0,73 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

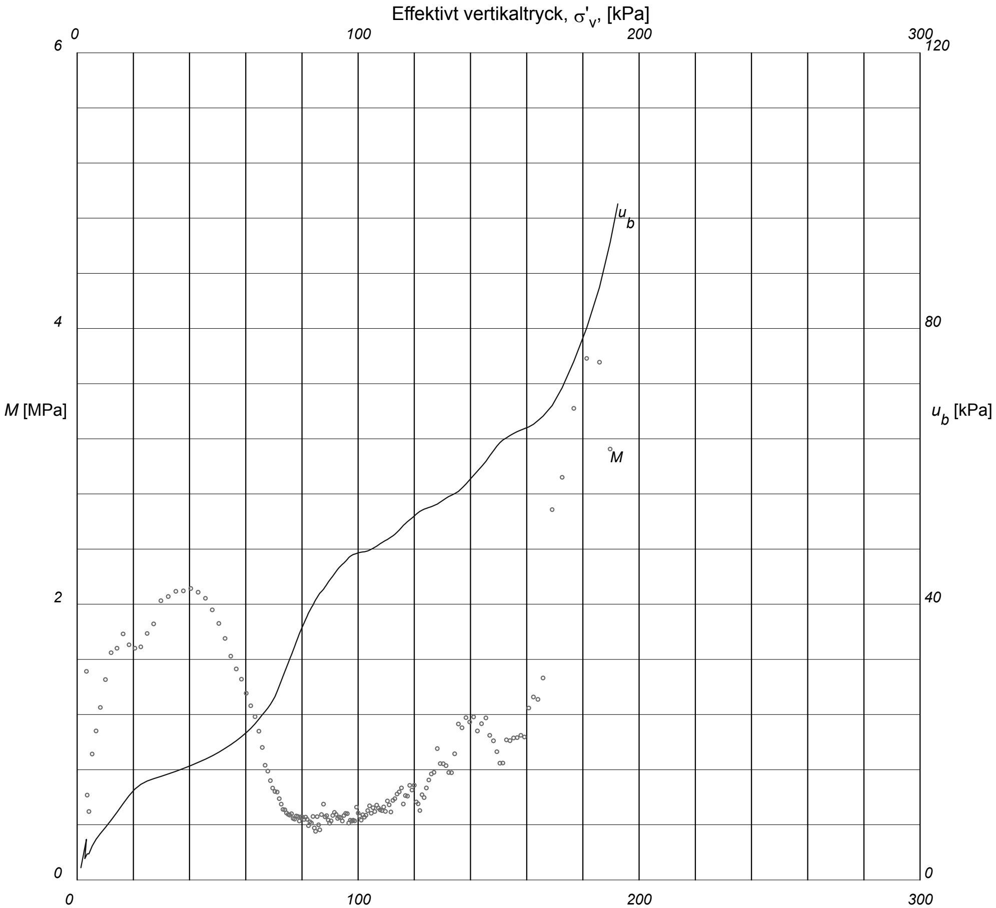
σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v, min}$, m^2/s	k_i, m/s	β_k
59	446	94	12,4	2,2E-9	9,5E-11	2,6

Anm.

Skalan i diagrammet avviker från den av SGF:s Laboratoriekommitté satta rekommendation.

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	Uppdragsgivare: Bjerking AB, Stockholm	Datum/Sign: 2016-05-24 Löp-nr/Gransk.: 30056
Sektion/borrhål: 16B46 Djup: 4,0 m Densitet: 1,69 t/m^3 Vattenkvot: 56 % Provningstemp.: 20 ^oC Benämning: Något sulfidbandad varvig lera		Ödometer nr: 2 Provdiameter: 50 mm Provhöjd: 20 mm Def.hastighet: 0,73 %/h



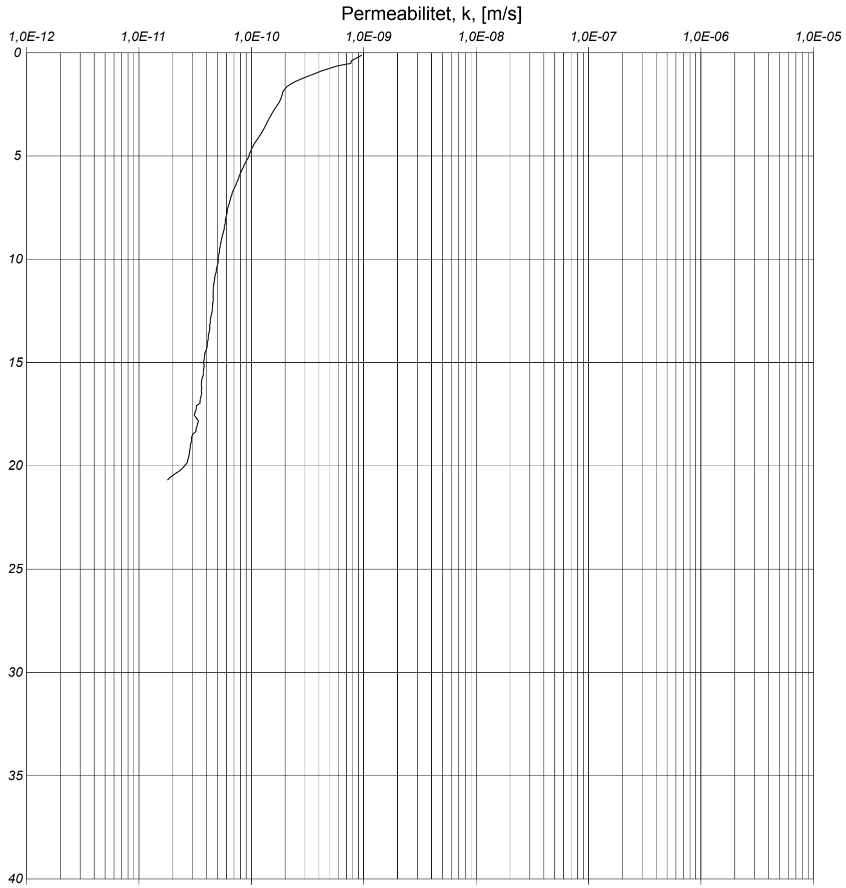
Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
12,4	94

Anm.

Utvärdering av permeabilitet

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-24 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46 *Densitet:* 1,69 t/m^3 *Benämning:* Något sulfidbandad varvig lera	*Djup:* 4,0 m *Vattenkvot:* 56 % *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

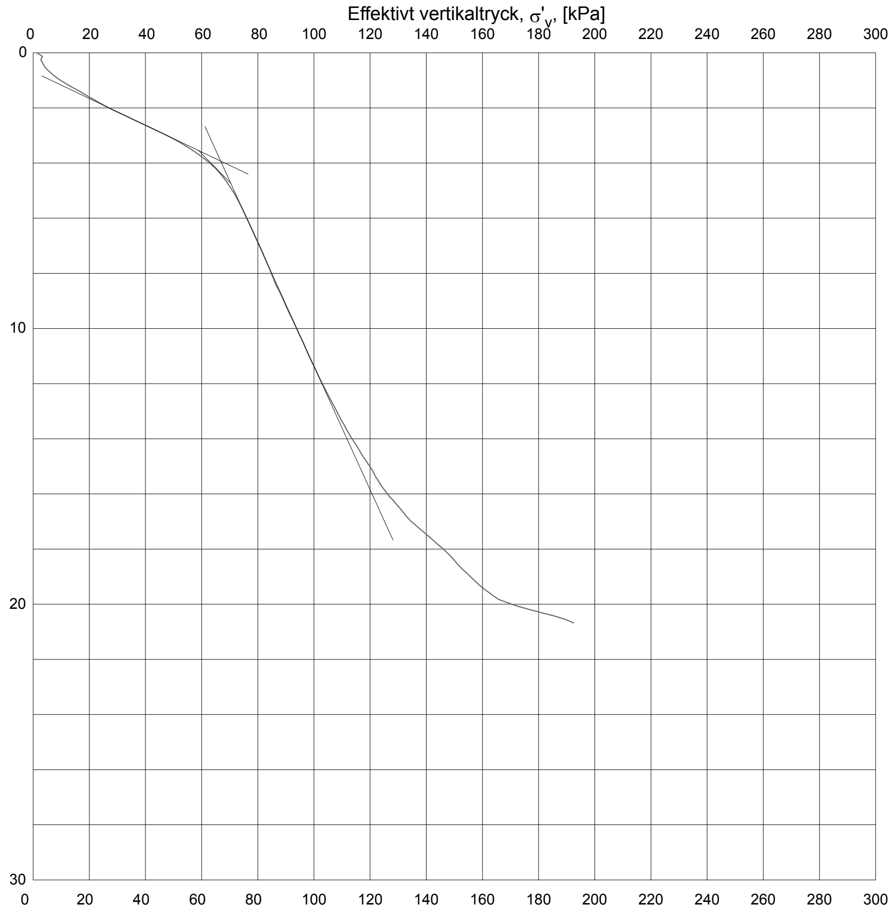
k_i, m/s	β_k
9,5E-11	2,6

Anm.

SWECO

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**		
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm	*Datum/Sign:* 2016-05-24 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46 *Densitet:* 1,69 t/m^3 *Benämning:* Något sulfidbandad varvig lera	*Djup:* 4,0 m *Vattenkvot:* 56 % *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,73 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
59	446	94

Anm.

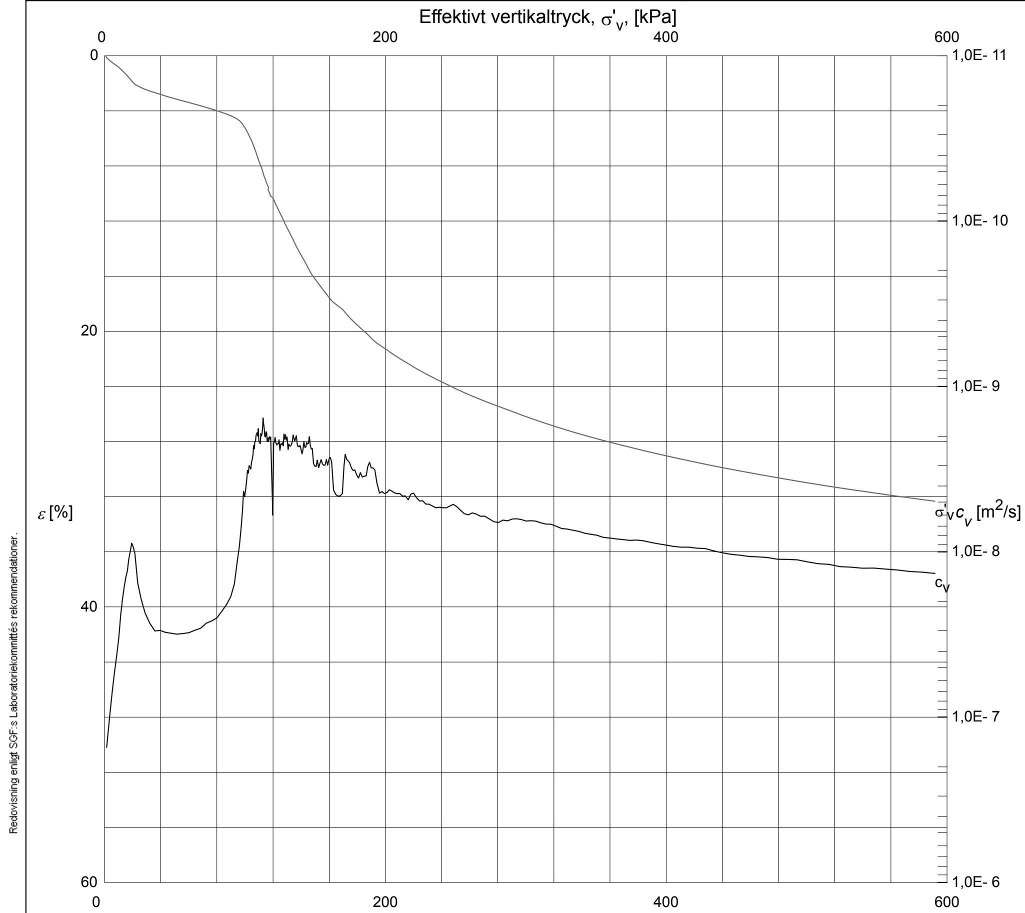
SWECO

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2016-05-24
16U29452	Bjerking AB, Stockholm	*Löp-nr/Gransk.:* 30056

Sektion/borrhål: 16B46 *Djup:* 6,0 m *Ödometer nr:* 4
Densitet: 1,79 t/m^3 *Vattenkvot:* 52 % *Provningstemp.:* 20 ^oC *Provdiameter:* 50 mm
Benämning: Ngt sulfidfläckig varvig lera m enst grusskikt *Provhöjd:* 20 mm
Def.hastighet: 0,72 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och *k* har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,min}$, m^2/s	k_i, m/s	β_k
93	349	111	14,7	2,0E-9	8,2E-11	2,1

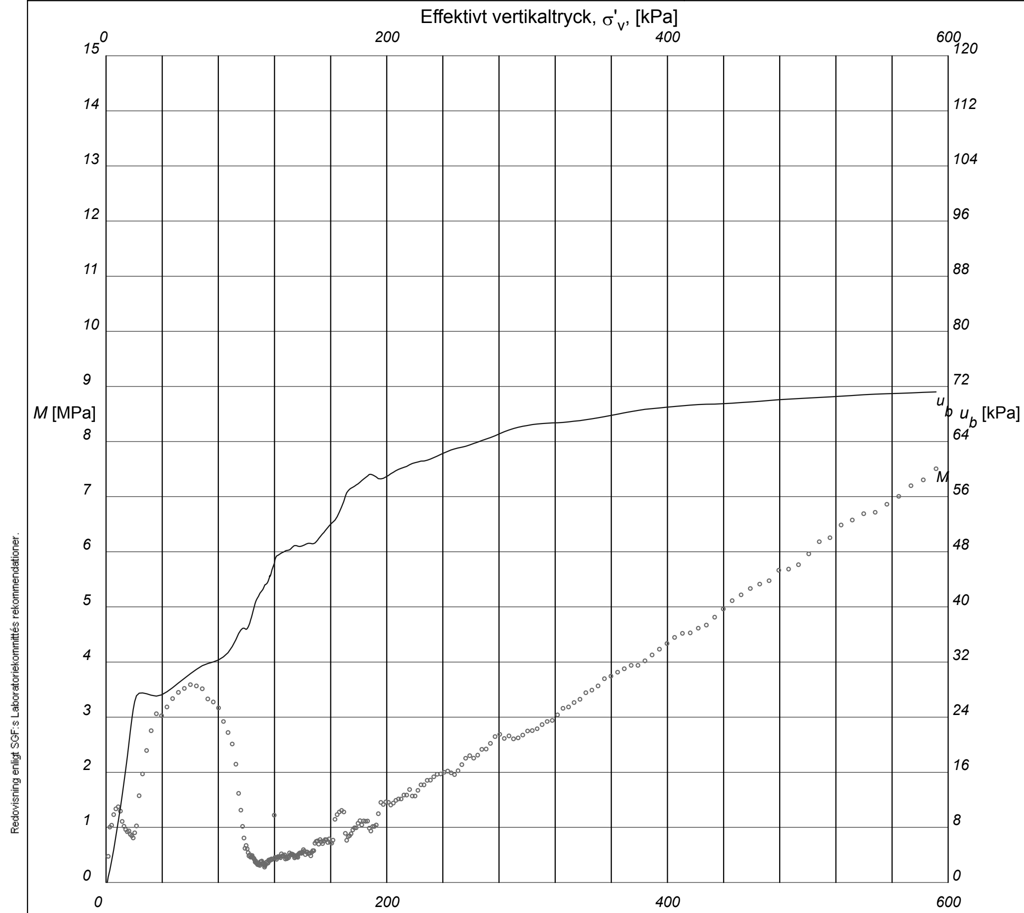
Anm.

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**

Uppdragsnummer:	*Uppdragsgivare:*	*Datum/Sign:* 2016-05-24
16U29452	Bjerking AB, Stockholm	*Löp-nr/Gransk.:* 30056

Sektion/borrhål: 16B46 *Djup:* 6,0 m *Ödometer nr:* 4
Densitet: 1,79 t/m^3 *Vattenkvot:* 52 % *Provningstemp.:* 20 ^oC *Provdiameter:* 50 mm
Benämning: Ngt sulfidfläckig varvig lera m enst grusskikt *Provhöjd:* 20 mm
Def.hastighet: 0,72 %/h



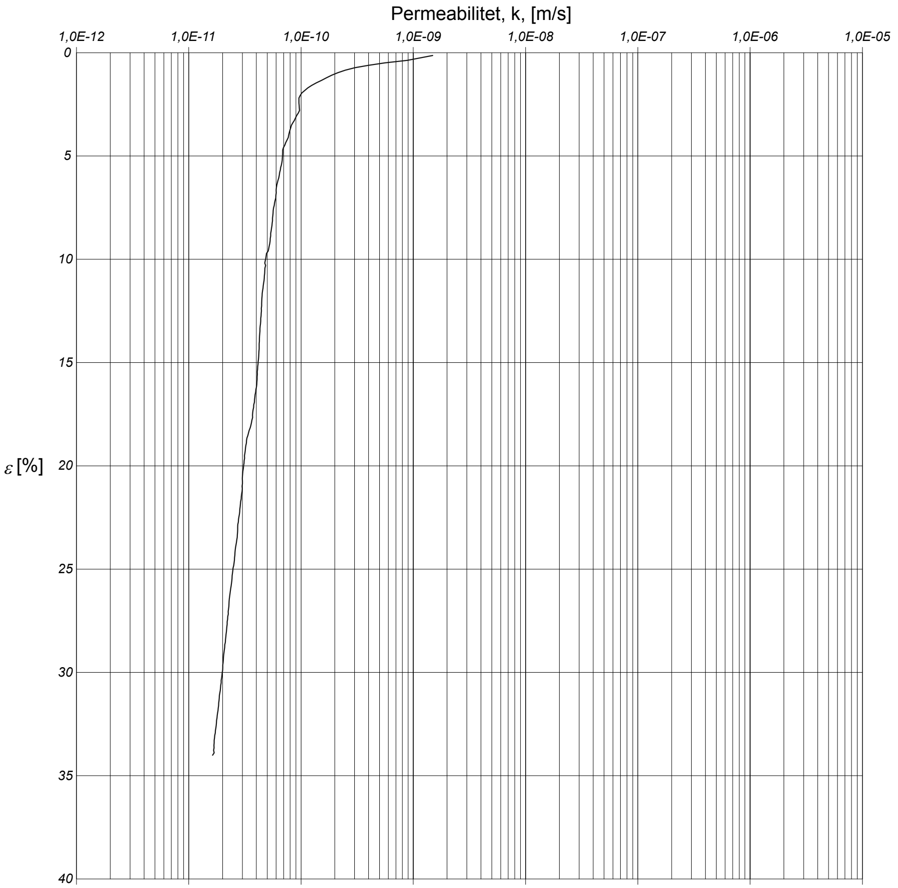
Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
14,7	111

Anm.

Utvärdering av permeabilitet

Projekt: **Dalvägen**			
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm		*Datum/Sign:* 2016-05-24 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46		*Djup:* 6,0 m	*Ödometer nr:* 4
Densitet: 1,79 t/m^3	*Vattenkvot:* 52 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Ngt sulfidfläckig varvig lera m enst grusskikt			*Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

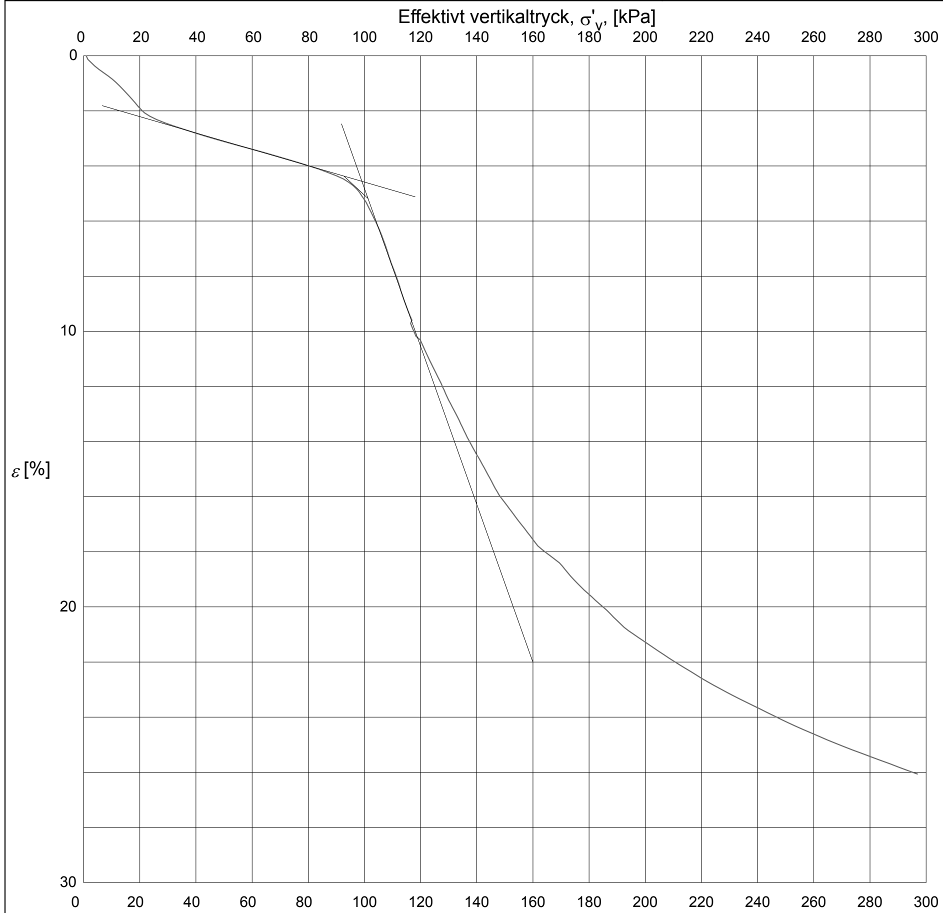
k_i , m/s	β_k
8,2E-11	2,1

Anm.

SWECO

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**			
Uppdragsnummer: 16U29452	*Uppdragsgivare:* Bjerking AB, Stockholm		*Datum/Sign:* 2016-05-24 *Löp-nr/Gransk.:* 30056
Sektion/borrhål: 16B46		*Djup:* 6,0 m	*Ödometer nr:* 4
Densitet: 1,79 t/m^3	*Vattenkvot:* 52 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Ngt sulfidfläckig varvig lera m enst grusskikt			*Provhöjd:* 20 mm *Def.hastighet:* 0,72 %/h



Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
93	349	111

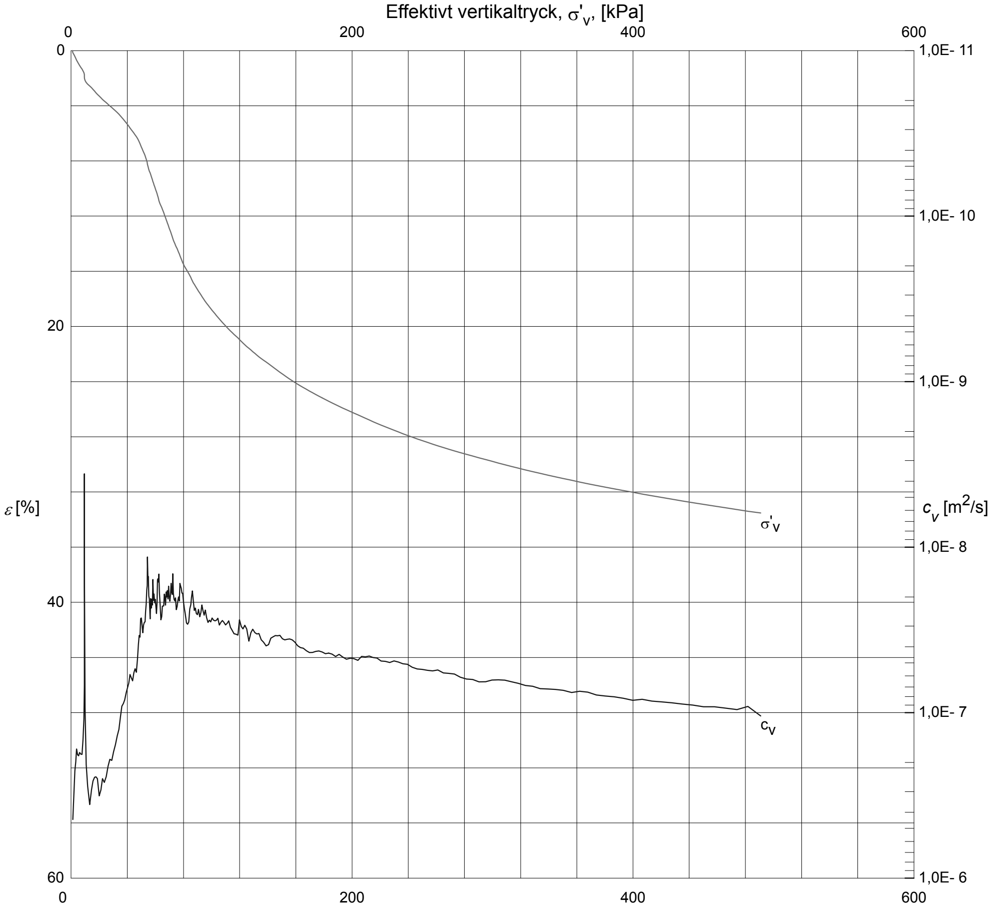
Anm.

SWECO

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**		
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30 *Densitet:* 1,76 t/m^3 *Benämning:* Lera	*Djup:* 2,0 m *Vattenkvot:* 50 % *Provningstemp.:* 20 ^oC	*Ödometer nr:* 1 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^oC. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\,min}$, m^2/s	k_i, m/s	β_k
42	339	67	15,0	1,8E-8	1,1E-9	2,6

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, ingår i SWECO Infrastructure AB

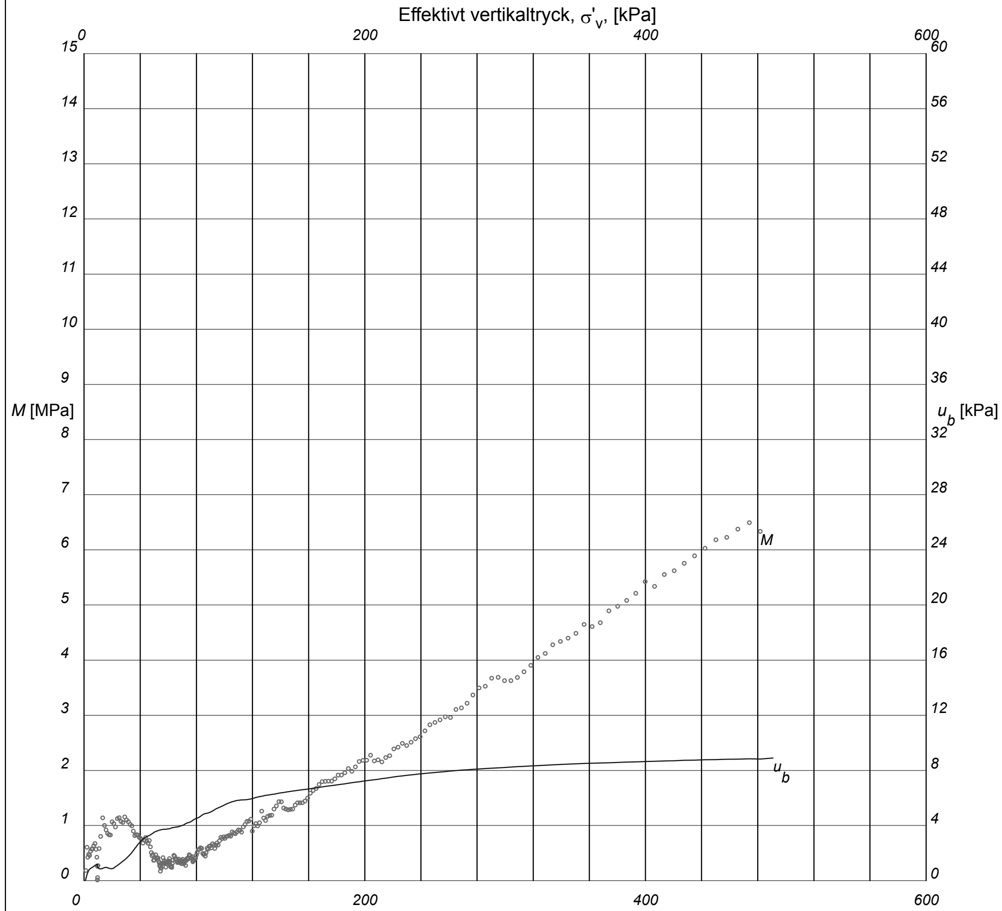
P:\2172\Uppdrag 2011\23720\a11_361.txt 2011-12-21

SWECO

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**		
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30 *Densitet:* 1,76 t/m^3 *Benämning:* Lera	*Djup:* 2,0 m *Vattenkvot:* 50 % *Provningstemp.:* 20 ^oC	*Ödometer nr:* 1 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
15,0	67

Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB, Gjörwellsgatan 22, Box 34044
100 26 STOCKHOLM, Tel: 08-695 60 00, Fax: 08-695 63 60
geolab@sweco.se, www.sweco.se/geolab, ingår i SWECO Infrastructure AB

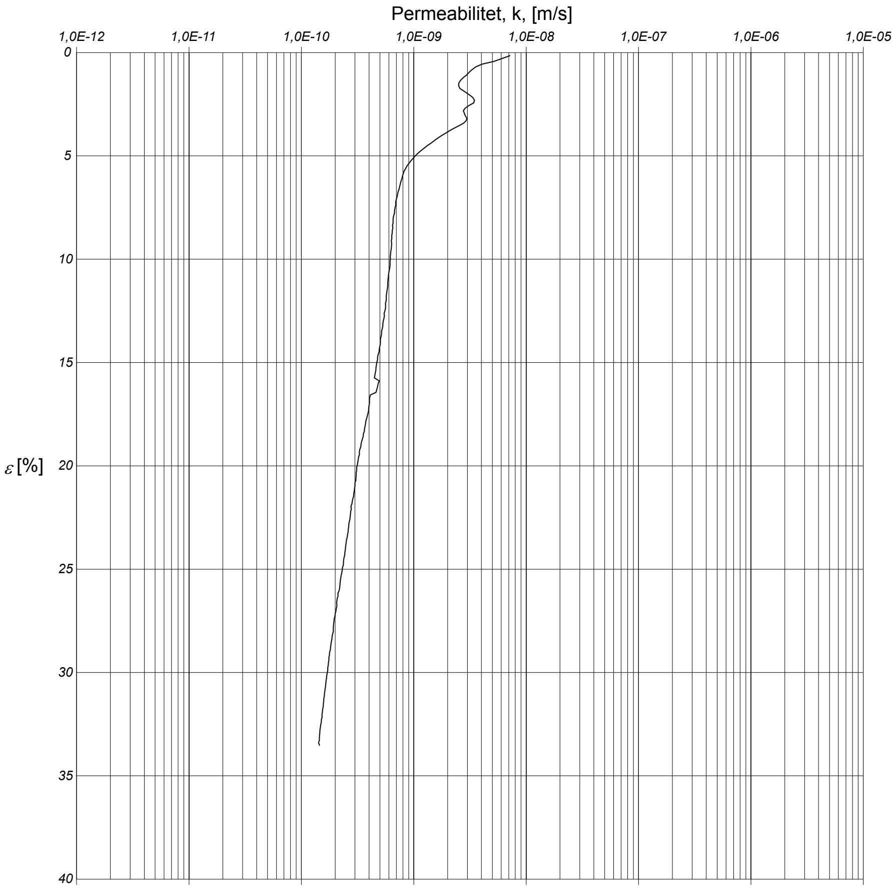
P:\2172\Uppdrag 2011\23720\a11_361.txt 2011-12-21

SWECO

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Dalvägen**			
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm		*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30		*Djup:* 2,0 m	*Ödometer nr:* 1
Densitet: 1,76 t/m^3	*Vattenkvot:* 50 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Lera			*Provhöjd:* 20 mm
			Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{O}C.

k_i , m/s	β_k
1,1E-9	2,6

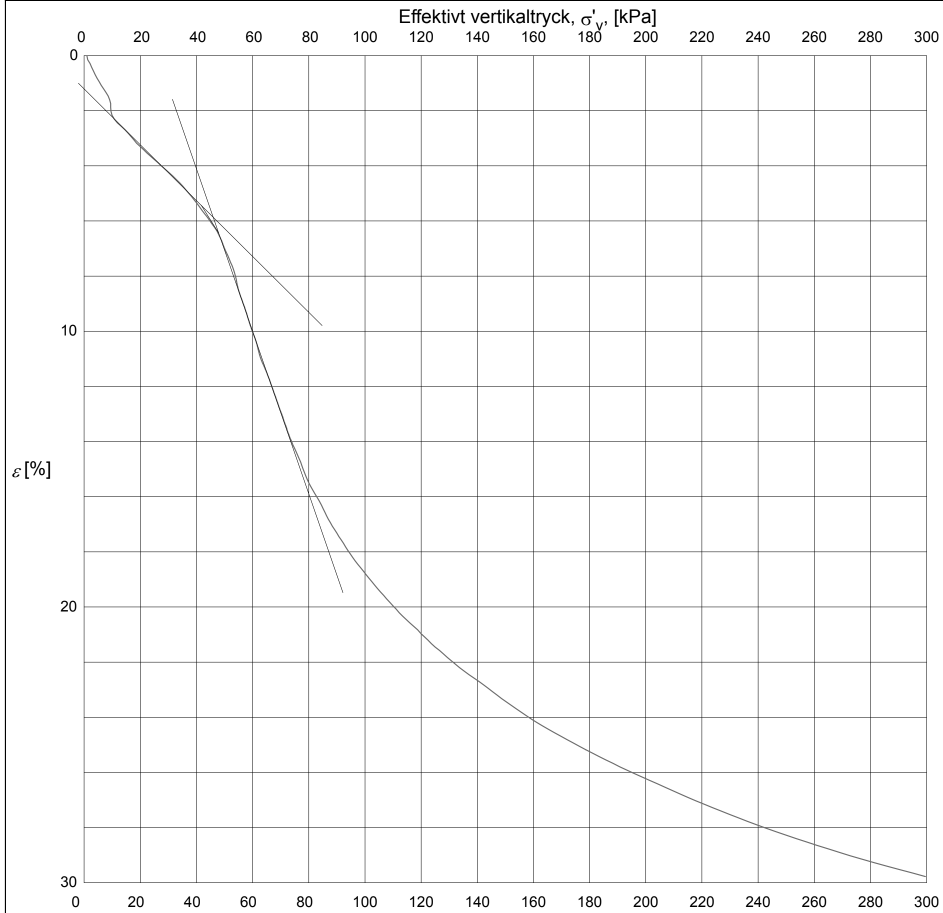
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**			
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm		*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30		*Djup:* 2,0 m	*Ödometer nr:* 1
Densitet: 1,76 t/m^3	*Vattenkvot:* 50 %	*Provningstemp.:* 20 ^{o}C	*Provdiameter:* 50 mm
Benämning: Lera			*Provhöjd:* 20 mm
			Def.hastighet: 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
42	339	67

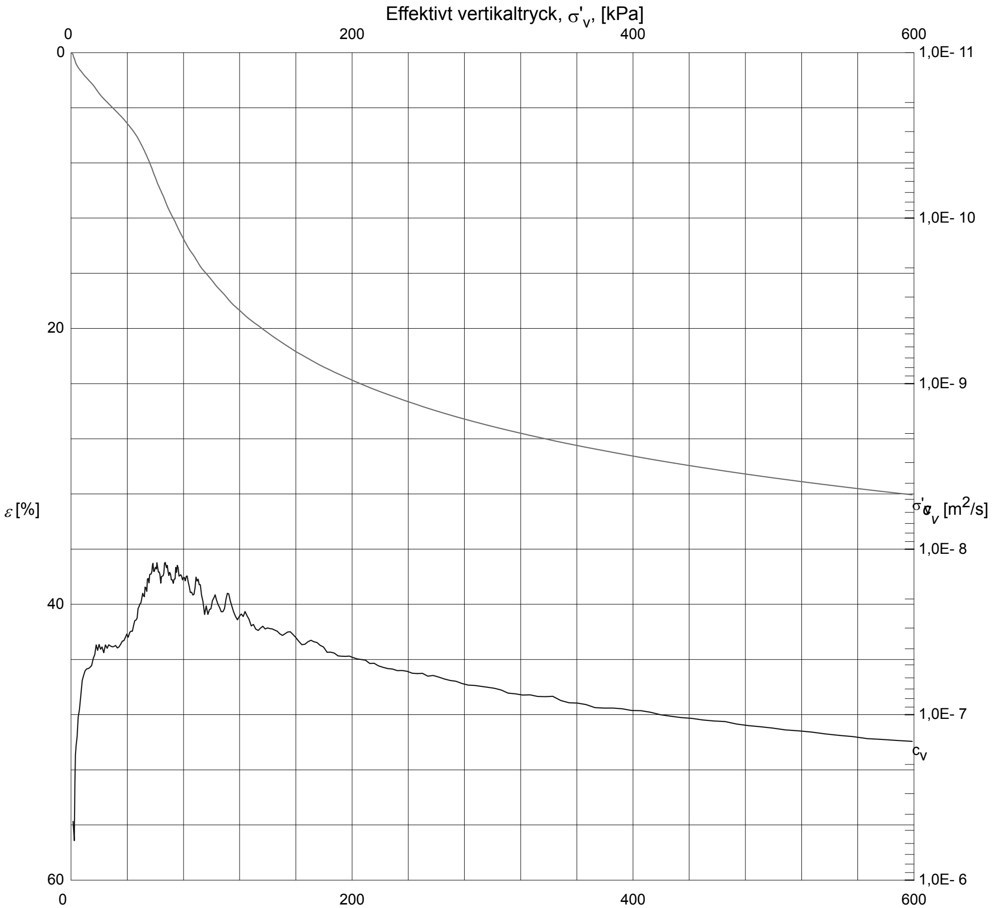
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Redovisning av ödometerförsök, CRS-försök

Projekt: **Dalvägen**		
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30 *Densitet:* 1,72 t/m^3 *Vattenkvot:* 53 % *Benämning:* Varvig lera	*Djup:* 3,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av c_v och *k* har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C. Utrustningens egendeformation är beaktad. För utvärdering se bilagda diagram sid 2 - 4.

σ'_c, kPa	M_L, kPa	σ'_L, kPa	M'	$c_{v,\,min}$, m^2/s	k_i , m/s	β_k
43	418	75	15,6	1,4E-8	4,6E-10	1,5

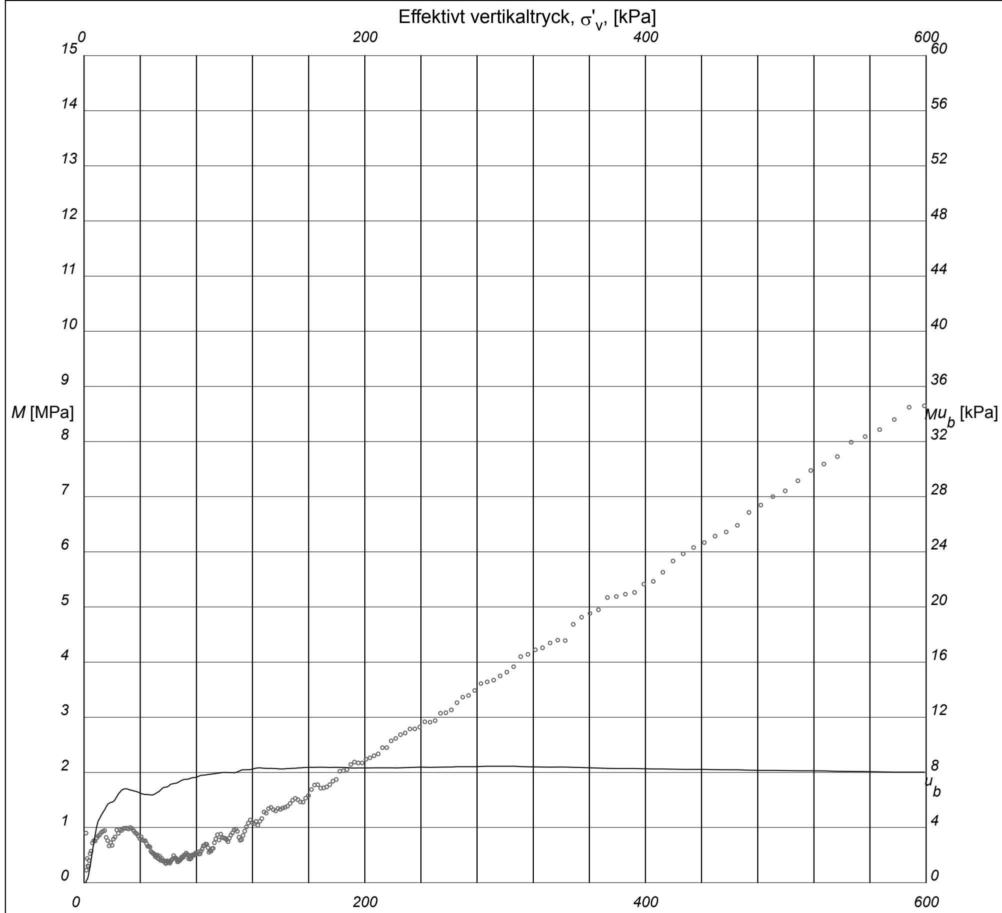
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av modultal och kontroll av portryck

Projekt: **Dalvägen**		
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30 *Densitet:* 1,72 t/m^3 *Vattenkvot:* 53 % *Benämning:* Varvig lera	*Djup:* 3,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

M'	σ'_L, kPa
15,6	75

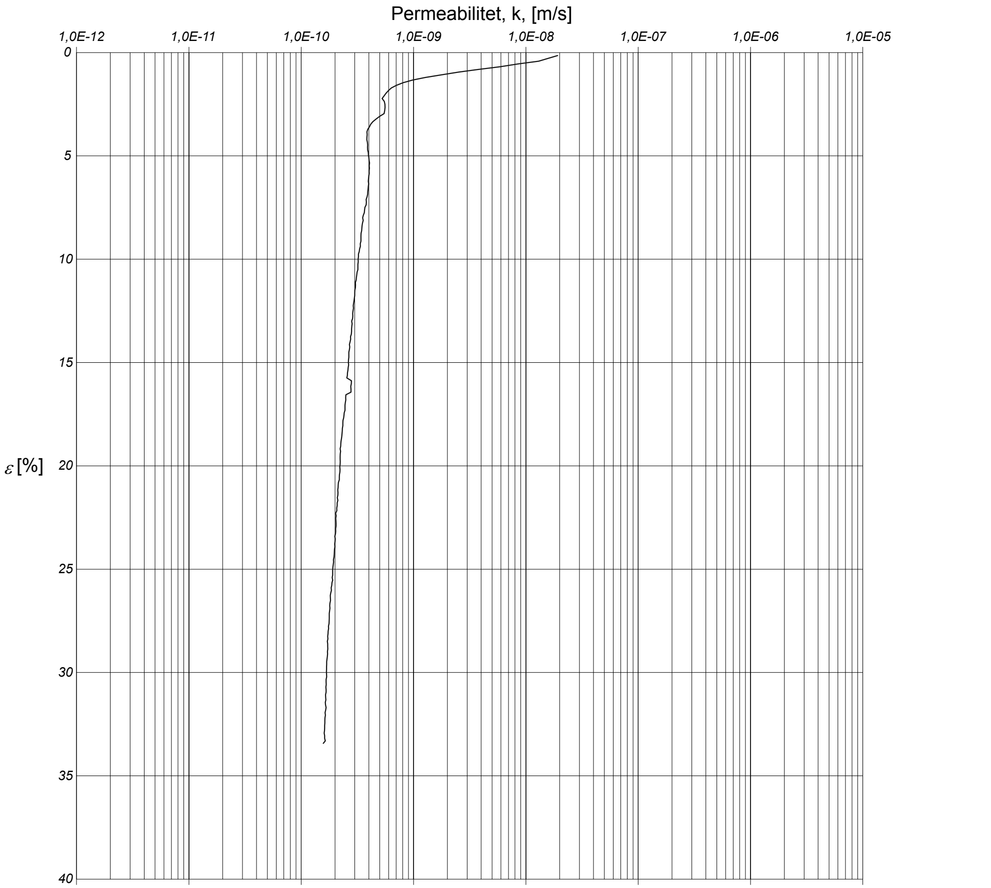
Anm.

Redovisning enligt SGF:s Laboratoriekommittés rekommendationer.

SWECO GEOLAB

Utvärdering av permeabilitet

Projekt: **Dalvägen**		
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30 *Densitet:* 1,72 t/m^3 *Vattenkvot:* 53 % *Benämning:* Varvig lera	*Djup:* 3,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Vid utvärdering av permeabiliteten k har korrektion utförts så att värdena motsvarar en temperatur av 7 ^{o}C.

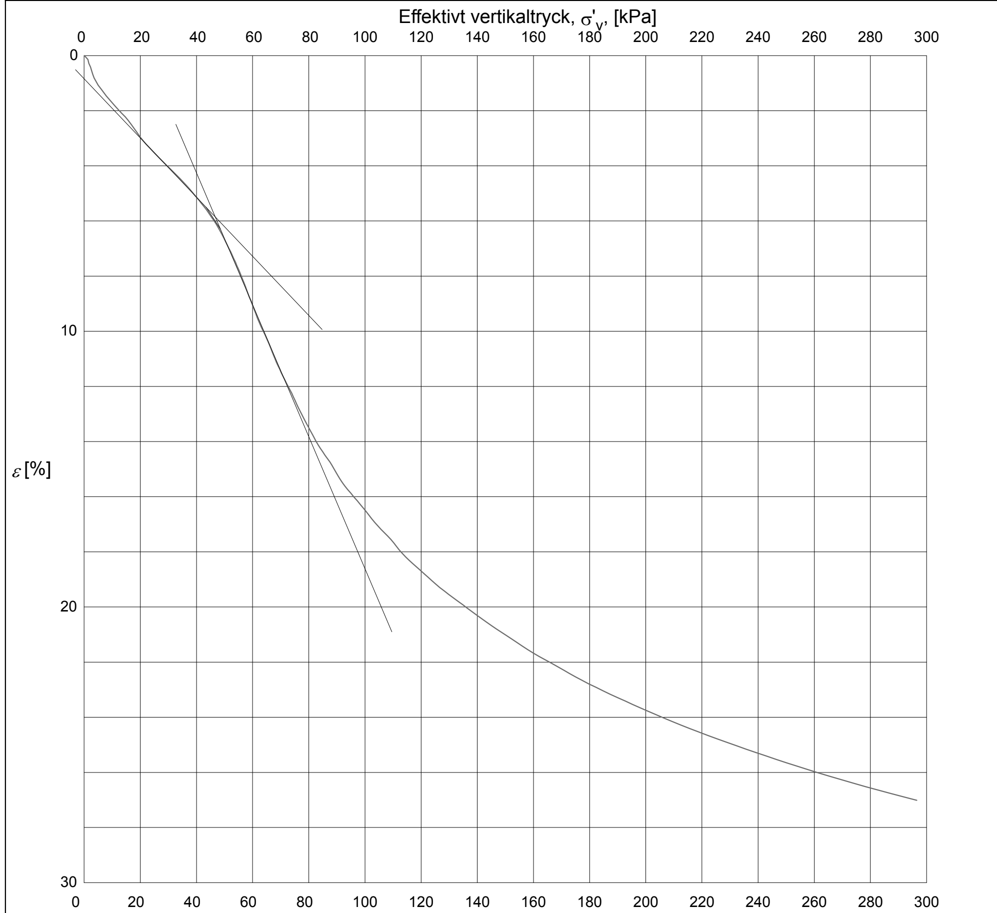
k_i, m/s	β_k
4,6E-10	1,5

Anm.

SWECO GEOLAB

Utvärdering av förkonsolideringstryck och linjär modul

Projekt: **Dalvägen**		
Uppdragsnummer:	*Uppdragsgivare:* WSP Samhällsbyggnad, Stockholm	*Datum/Sign:* 2011-12-19 *Löp-nr/Gransk.:* 23720
Sektion/borrhål: 30 *Densitet:* 1,72 t/m^3 *Vattenkvot:* 53 % *Benämning:* Varvig lera	*Djup:* 3,0 m *Provningstemp.:* 20 ^{o}C	*Ödometer nr:* 2 *Provdiameter:* 50 mm *Provhöjd:* 20 mm *Def.hastighet:* 0,74 %/h



Försöket är utfört och utvärderat enligt Svensk Standard SS 027126. Utrustningens egendeformation är beaktad.

σ'_c, kPa	M_L, kPa	σ'_L, kPa
43	418	75

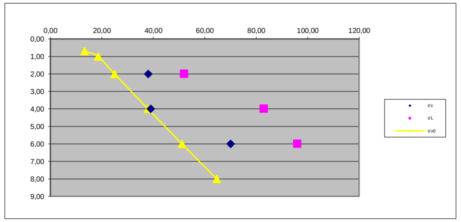
Anm.

Arkitekter Ingenjörer

PROJECT: Dalvägen		NR OF PAGES: 3	PAGE NR: 1
Calculation of the settlements		PROJECT NR: 16U29452	
14W44		SIGN: CGI	
		DATUM: 2016-05-27	

Depth	Soil type	h	Contribute to settlements? Yes or no	ρ	σ'_c	σ'_L	M_L	M_0	M'	a	τ^{fu}	W_L	τ^{fed}
0,7	F		no	1,9									
1	Cl dc	0	no	1,75									
2	Clay	2	yes	1,64	38	52	224	2401	14,9	36,97	9,7	49	9,15
4	Clay	2	yes	1,66	39	83	240	3375	18	69,67	10	43	10,00
6	Clay	2	yes	1,66	70	96	230	3315	20,4	84,73	12	47	11,53
8	Clay	2,3	yes	1,69	75	106	544	3713	20,2	79,07	15	49	14,14
9,3	Fr		no	1,9									
	Clay thickness	8,3											
Mean values:				1,743	55,5	84,25	309,5	3201	18,375	67,61	11,675		11,20479

Depth	Classification of clay	Classification of texture limits	k-value
0,7			
1			
2	Very loose clay	Mean plastic	262,5
4	Very loose clay	Mean plastic	337,5
6	Very loose clay	Mean plastic	287,5
8	Loose clay	Mean plastic	262,5
9,3			



Level of the ground water (m under ground) 1

Stress diagram

Depth	σ_{v0}	u	σ'_{v0}	σ'_c	σ'_L	OCR	Consolidation
0,70	13,30	0,00	13,30				
1,00	18,55	0,00	18,55				
2,00	34,95	10,00	24,95	38,00	52,00	1,523	Over consolidated
4,00	67,95	30,00	37,95	39,00	83,00	1,028	Normal/lightly overconsolidated
6,00	101,15	50,00	51,15	70,00	96,00	1,369	Normal/lightly overconsolidated
8,00	134,65	70,00	64,65	75,00	106,00	1,16	Normal/lightly overconsolidated
9,30	156,62	83,00	73,62				

Arkitekter Ingenjörer

PROJECT: Dalvägen		NR OF PAGES: 3	PAGE NR: 2
Calculation of the settlements		PROJECT NR: 16U29452	
14W44		SIGN: CGI	
		DATUM: 2016-05-27	

Properties of made ground

ρ made ground 1,9

Thickness	$\Delta\sigma_v$
0,5	9,5
1	19
1,5	28,5
2	38
3	57

Choose load distribution

- (●) Without load distribution in the layer
- ( ) 2:1-method

Results

Settlements in the clay:

Depth	Filling 0,5m	Filling 1m	Filling 1,5m	Filling 2m	Filling 3m
0,7					
1					
2	0,008	0,064	0,148	0,209	0,283
4	0,071	0,150	0,229	0,309	0,438
6	0,006	0,013	0,095	0,178	0,309
8	0,006	0,043	0,083	0,123	0,190
9,3					
S (m):	0,091	0,270	0,556	0,819	1,220

Arkitekter Ingenjörer	PROJECT Dalvägen	NR OF PAGES 3	PAGE NR 3
14W44	Calculation of the settlements	PROJECT NR 16U29452	
		SIGN CGI	
		DATUM 2016-05-27	

Consolidation

Double drainage?
yes

Clay thickness
8,3 m

Consolidation degree(%)
50

Consolidation coefficient
7,30E-09

After how long do you want to know the settlements
574 months

Cv min
1,3E-08
1,9E-09
3,9E-09
1E-08

Mean Cv 7,30E-09

After what time do you have 50 % settlements

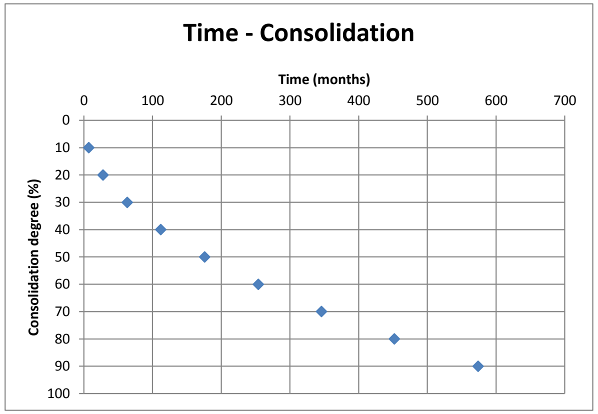
Time in months	Times in year
179	15

Consolidation after 574 months = 90 %

Settlements after 574 months

Filling	Settlements	
0,5	0,081	m
1	0,242	m
1,5	0,498	m
2	0,734	m
3	1,093	m

% consolid.	time (months)
10	7
20	28
30	63
40	112
50	176
60	254
70	346
80	452
90	574

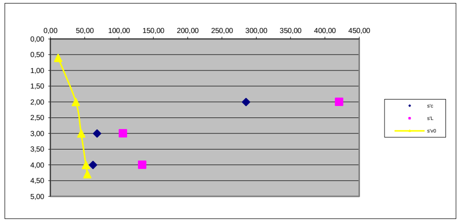


Arkitekter Ingenjörer

	PROJECT	NR OF PAGES	PAGE NR
	Dalvägen	3	1
14W17	Calculation of the settlements	PROJECT NR 16U29452	
		SIGN CGI	
		DATUM 2016-05-27	

Depth	Soil type	h	Contribute to settlements? Yes or no	ρ	σ'_c	σ'_L	M_L	M_0	M'	a	τ_{fu}	W_L	τ_{fed}
0,6	F		no	1,9									
2	Cl dc	1,9	yes	1,84	285	421	4041	11386	13,2	114,9	46	49	43,37
3	Clay	1	yes	1,69	68	106	223	2803	17,7	93,4	12	50	11,21
4	Clay	0,8	yes	1,7	62	134	760	2798	16,9	89,03	15	60	12,91
4,3	Fr		no	1,9									
	Clay thickness	3,7											
Mean values:				1,806	138,3	220,3	1675	5662	15,9333	99,1	24,33333333		22,49946

Depth	Classification of clay	Classification of texture limits	k-value
0,6			
2	Mean dense clay	Mean plastic	262,5
3	Very loose clay	Mean plastic	250
4	Loose clay	High plastic	216,7
4,3			



Level of the ground water (m under ground) 2

Stress diagram

Depth	σ_{v0}	u	σ'_{v0}	σ'_c	σ'_L	OCR	Consolidation
0,60	11,40	0,00	11,40				
2,00	37,16	0,00	37,16	285,00	421,00	7,67	Over consolidated
3,00	54,81	10,00	44,81	68,00	106,00	1,518	Over consolidated
4,00	71,76	20,00	51,76	62,00	134,00	1,198	Normal/lightly overconsolidated
4,30	76,86	23,00	53,86				

Arkitekter Ingenjörer

	PROJECT	NR OF PAGES	PAGE NR
	Dalvägen	3	2
14W17	Calculation of the settlements	PROJECT NR 16U29452	
		SIGN CGI	
		DATUM 2016-05-27	

Properties of made ground

ρ made ground 1,9

Thickness	$\Delta\sigma_v$
0,5	9,5
1	19
1,5	28,5
2	38
3	57

Choose load distribution

- (●) Without load distribution in the layer
- ( ) 2:1-method

Results

Settlements in the clay:

Filling 0,5m Depth		Filling 1m Depth		Filling 1,5m Depth		Filling 2m Depth		Filling 3m Depth	
0,6		0,6		0,6		0,6		0,6	
2	0,002	2	0,003	2	0,005	2	0,006	2	0,010
3	0,003	3	0,007	3	0,032	3	0,075	3	0,160
4	0,003	4	0,012	4	0,022	4	0,032	4	0,052
4,3		4,3		4,3		4,3		4,3	
S (m):	0,008		0,022		0,059		0,113		0,222

Consolidation

Double drainage?
yes

Clay thickness
3,7 m

Consolidation degree(%)
50

Consolidation coefficient
1,39E-08

Cv min
2,3E-08
4,2E-09
4,3E-09

Mean Cv 1,39E-08

After how long do you want to know the settlements
60 months

After what time do you have 50 % settlements

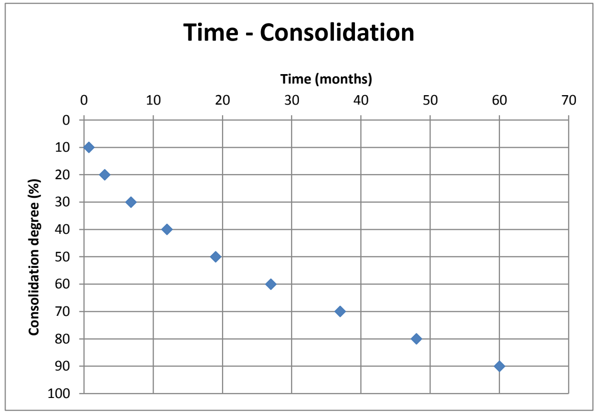
Time in months	Times in year
19	2

Consolidation after 60 months = 90 %

Settlements after 60 months

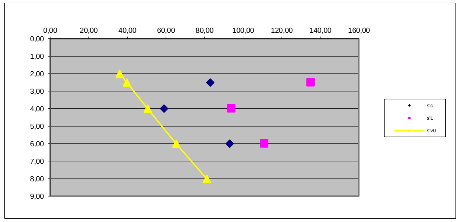
Filling	Settlements	
0,5	0,007	m
1	0,020	m
1,5	0,053	m
2	0,101	m
3	0,199	m

% consolid.	time (months)
10	0,7
20	3
30	6,8
40	12
50	19
60	27
70	37
80	48
90	60



Depth	Soil type	h	Contribute to settlements? Yes or no	ρ	σ'c	σ'L	ML	M0	M'	a	τfu	WL	τfed
2	Cl dc		no	1,8									
2,5	Clay	1,25	yes	1,75	83	135	802	4973	16,4	86,1	19	48	18,08
4	Clay	1,75	yes	1,69	59	94	446	3783	12,4	58,03	13	46	12,61
6	Clay	3	yes	1,79	93	111	349	4245	14,7	87,26	12	42	12,13
8	Fr		no	1,9									
	Clay thickness	6											
Mean values:				1,786	78,33	113,3	532,3	4334	14,5	77,13	14,66666667		14,27384

Depth	Classification of clay	Classification of texture limits	k-value
2			
2,5	Loose clay	Mean plastic	275
4	Loose clay	Mean plastic	300
6	Very loose clay	Mean plastic	350
8			



Level of the ground water (m under ground) 2

Stress diagram

Depth	σ_{v0}	u	σ'_{v0}	σ'_c	σ'_L	OCR	Consolidation
2,00	36,00	0,00	36,00				
2,50	44,75	5,00	39,75	83,00	135,00	2,088	Overconsolidated
4,00	70,55	20,00	50,55	59,00	94,00	1,167	Normal/lightly overconsolidated
6,00	105,35	40,00	65,35	93,00	111,00	1,423	Normal/lightly overconsolidated
8,00	141,15	60,00	81,15				

Properties of made ground

ρ made ground: 1,9

Choose load distribution

- (●) Without load distribution in the layer
- ( ) 2:1-method

Thickness	$\Delta\sigma_v$
0,5	9,5
1	19
1,5	28,5
2	38
3	57

Results

Settlements in the clay:

Filling 0,5m Depth		Filling 1m Depth		Filling 1,5m Depth		Filling 2m Depth		Filling 3m Depth	
2		2		2		2		2	
2,5	0,002	2,5	0,005	2,5	0,007	2,5	0,010	2,5	0,032
4	0,008	4	0,045	4	0,083	4	0,120	4	0,186
6	0,007	6	0,013	6	0,027	6	0,109	6	0,254
8		8		8		8		8	
S (m):	0,017		0,064		0,117		0,238		0,473

	PROJECT	NR OF PAGES	PAGE NR
Arkitekter Ingenjörer	Dalvägen	3	3
	Calculation of the settlements	PROJECT NR 16U29452	
16B46		SIGN CGI	
		DATUM 2016-05-27	

Consolidation

Double drainage?
yes

Clay thickness
6 m

Consolidation degree(%)
50

Consolidation coefficient
7,06E-09

Cv min
2,6E-08
2,2E-09
2E-09

Mean Cv 7,06E-09

After how long do you want to know the settlements
309 months

After what time do you have 50 % settlements

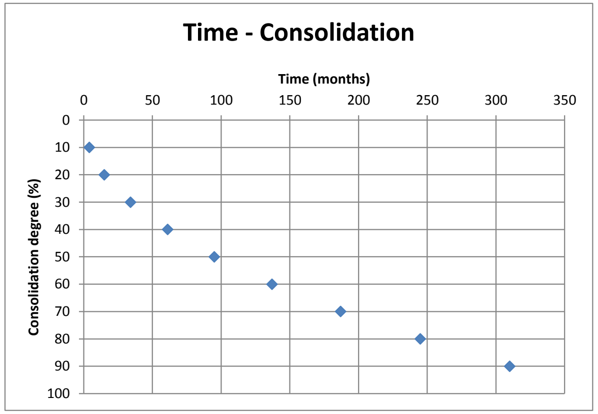
Time in months	Times in year
97	8

Consolidation after 309 months = 89 %

Settlements after 309 months

Filling	Settlements	
0,5	0,015	m
1	0,057	m
1,5	0,104	m
2	0,213	m
3	0,423	m

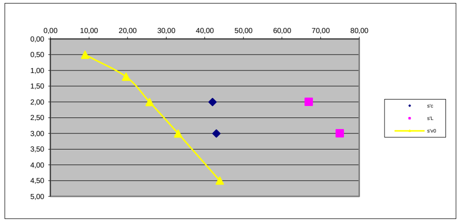
% consolid.	time (months)
10	4
20	15
30	34
40	61
50	95
60	137
70	187
80	245
90	310



Arkitekter Ingenjörer	PROJECT Dalvägen	NR OF PAGES 3	PAGE NR 1
30	Calculation of the settlements	PROJECT NR 16U29452	
		SIGN CGI	
		DATUM 2016-06-02	

Depth	Soil type	h	Contribute to settlements? Yes or no	ρ	σ'_c	σ'_L	M_L	M_0	M'	a	τ_{fu}	W_L	τ_{red}
0,5	F		no	1,8									
1,2	Cl dc	0	no	1,8									
2	Clay	1,3	yes	1,76	42	67	339	2895	15	44,4	9	44	8,91
3	Clay	2	yes	1,72	43	75	418	3062	15,6	48,21	10	45	9,80
4,5	Fr		no	1,9									
	Clay thickness	3,3											
Mean values:				1,796	42,5	71	378,5	2978	15,3	46,3	9,5		9,352435

Depth	Classification of clay	Classification of texture limits	k-value
0,5			
1,2			
2	**Very loose clay**	**Mean plastic**	325
3	**Very loose clay**	**Mean plastic**	312,5
4,5			



Level of the ground water (m under ground) 1

Stress diagram

Depth	σ_{v0}	u	σ'_{v0}	σ'_c	σ'_L	OCR	Consolidation
0,50	9,00	0,00	9,00				
1,20	21,60	2,00	19,60				
2,00	35,68	10,00	25,68	42,00	67,00	1,636	**Over consolidated**
3,00	53,08	20,00	33,08	43,00	75,00	1,3	**Normal/lightly overconsolidated**
4,50	78,88	35,00	43,88				

Arkitekter Ingenjörer	PROJECT Dalvägen	NR OF PAGES 3	PAGE NR 2
30	Calculation of the settlements	PROJECT NR 16U29452	
		SIGN CGI	
		DATUM 2016-06-02	

Properties of made ground

ρ made ground 1,9

Choose load distribution

- (●) Without load distribution in the layer
- ( ) 2:1-method

Thickness	$\Delta\sigma_v$
0,5	9,5
1	19
1,5	28,5
2	38
3	57

Results

Settlements in the clay:

Filling 0,5m Depth		Filling 1m Depth		Filling 1,5m Depth		Filling 2m Depth		Filling 3m Depth	
0,5		0,5		0,5		0,5		0,5	
1,2		1,2		1,2		1,2		1,2	
2	0,004	2	0,018	2	0,054	2	0,090	2	0,149
3	0,006	3	0,050	3	0,095	3	0,141	3	0,217
4,5		4,5		4,5		4,5		4,5	
S (m):	0,010		0,068		0,149		0,231		0,366

Consolidation

Double drainage?
yes

Clay thickness
3,3 m

Consolidation degree(%)
50

Cv min
1,8E-08
1,4E-08

Consolidation coefficient
1,56E-08

Mean Cv 1,56E-08

After how long do you want to know the settlements
12 months

After what time do you have 50 % settlements

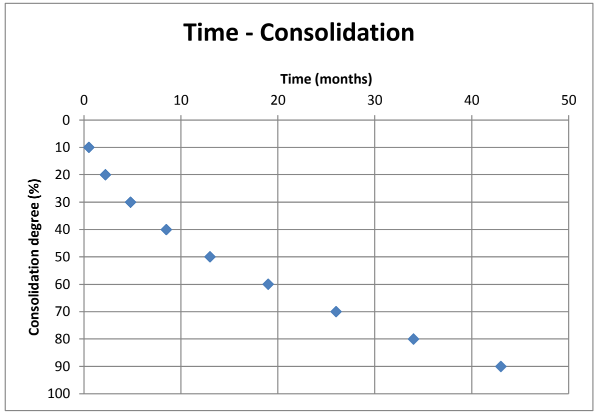
Time in months	Times in year
13	1

Consolidation after 12 months = 48 %

Settlements after 12 months

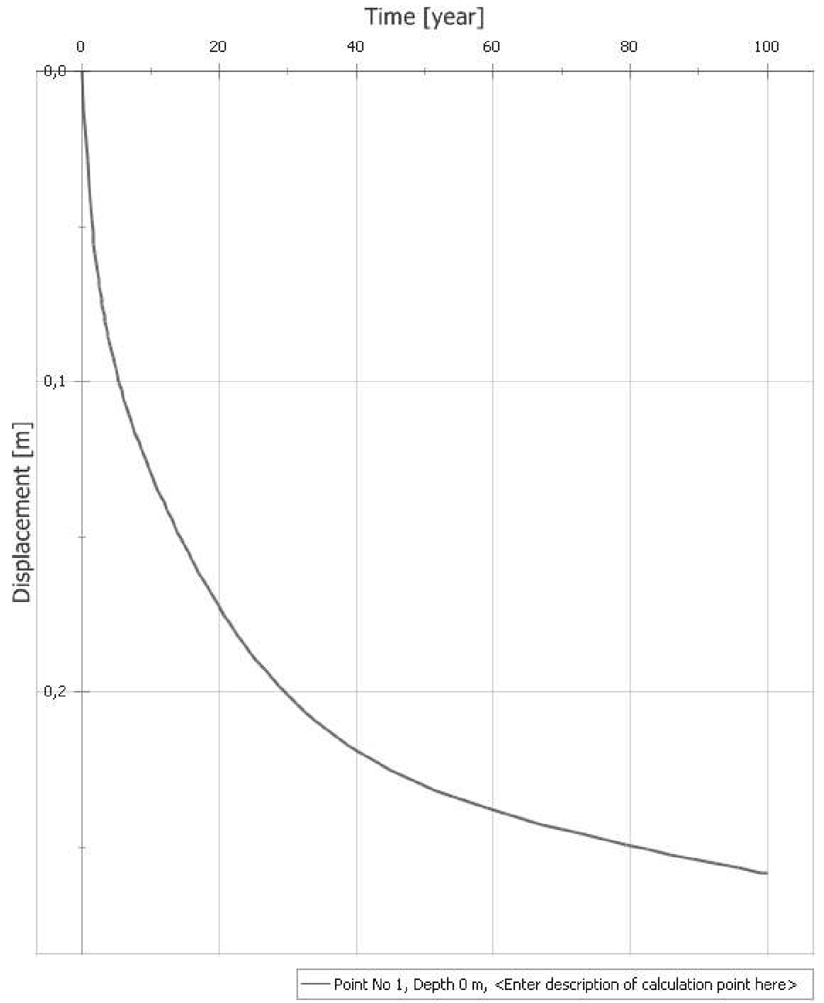
Filling	Settlements	
0,5	0,005	m
1	0,032	m
1,5	0,071	m
2	0,110	m
3	0,174	m

% consolid.	time (months)
10	0,5
20	2,2
30	4,8
40	8,5
50	13
60	19
70	26
80	34
90	43



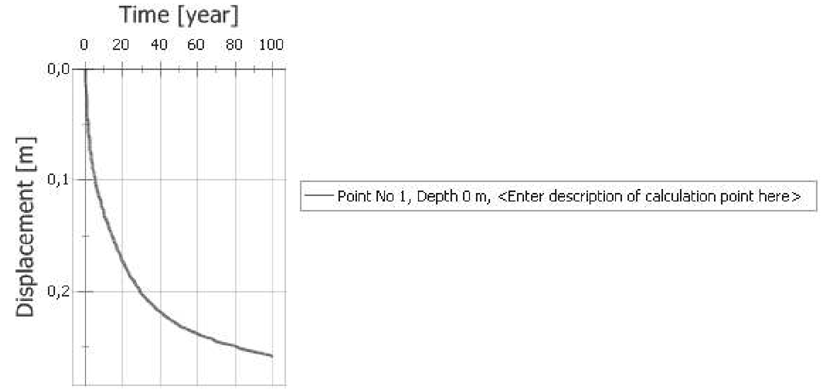
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

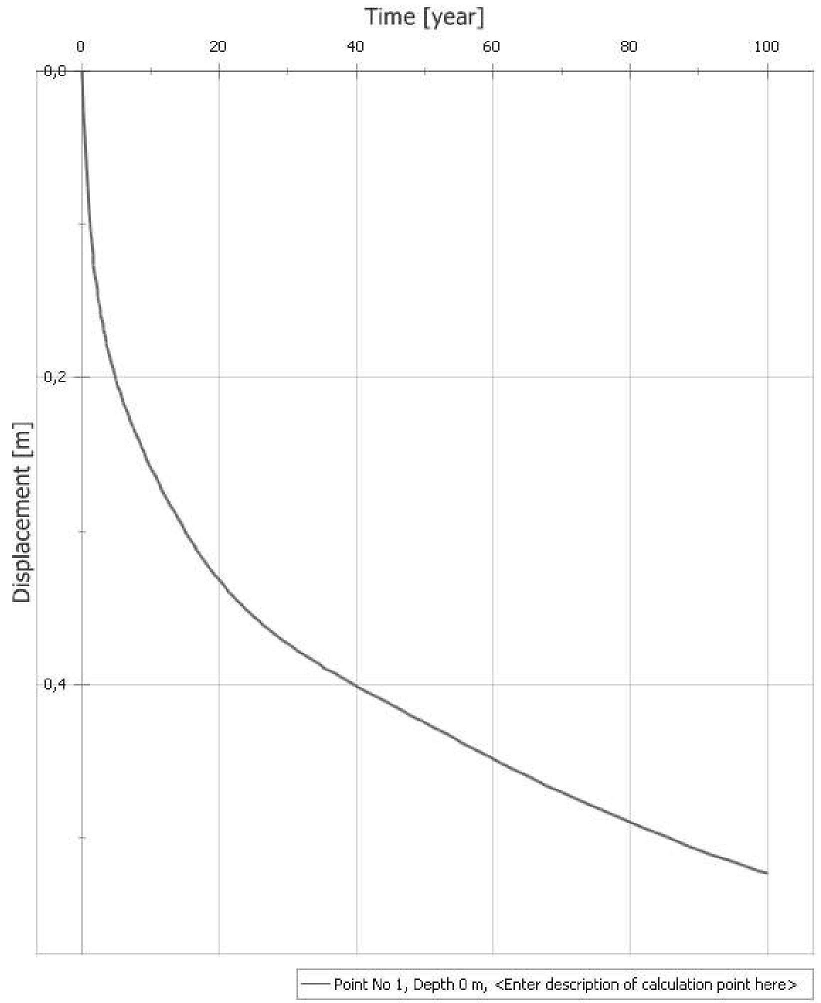
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,258	100,0000

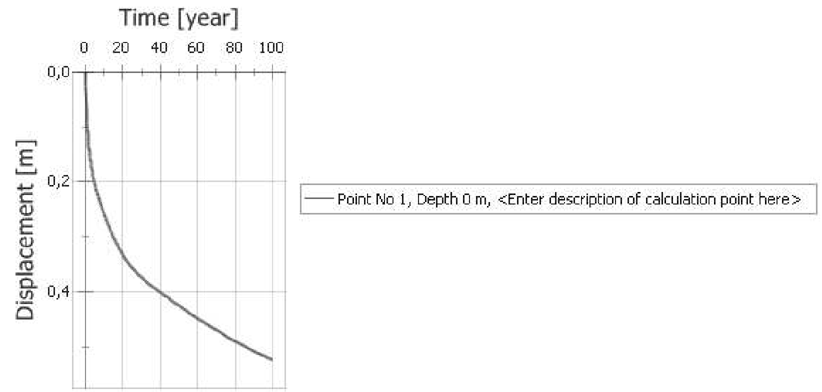
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

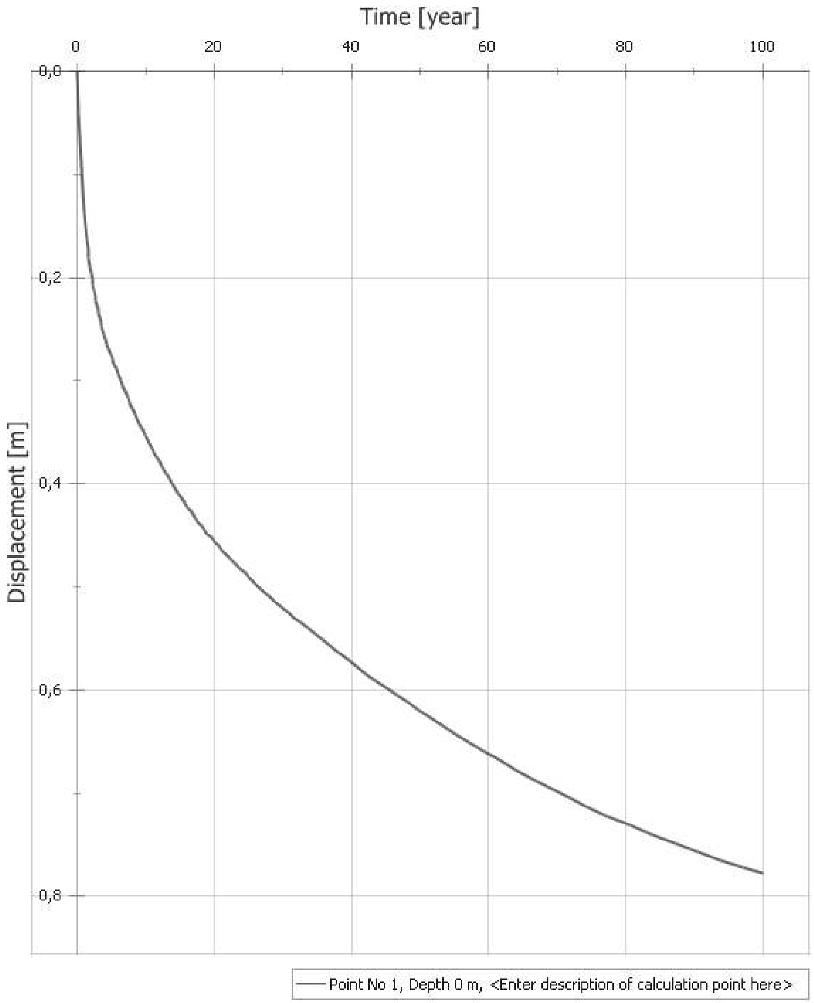
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,523	100,0000

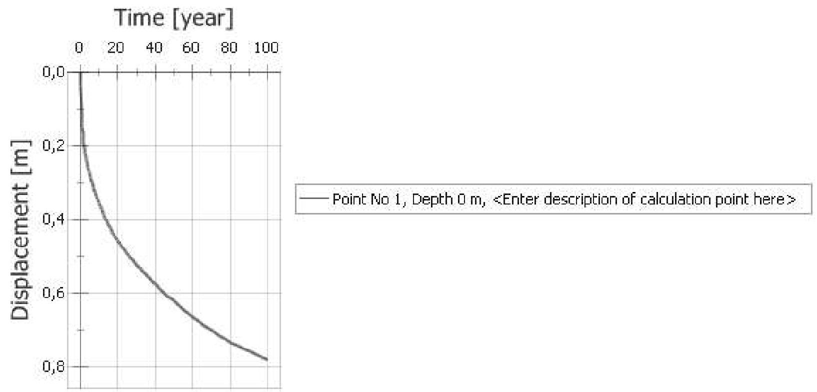
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

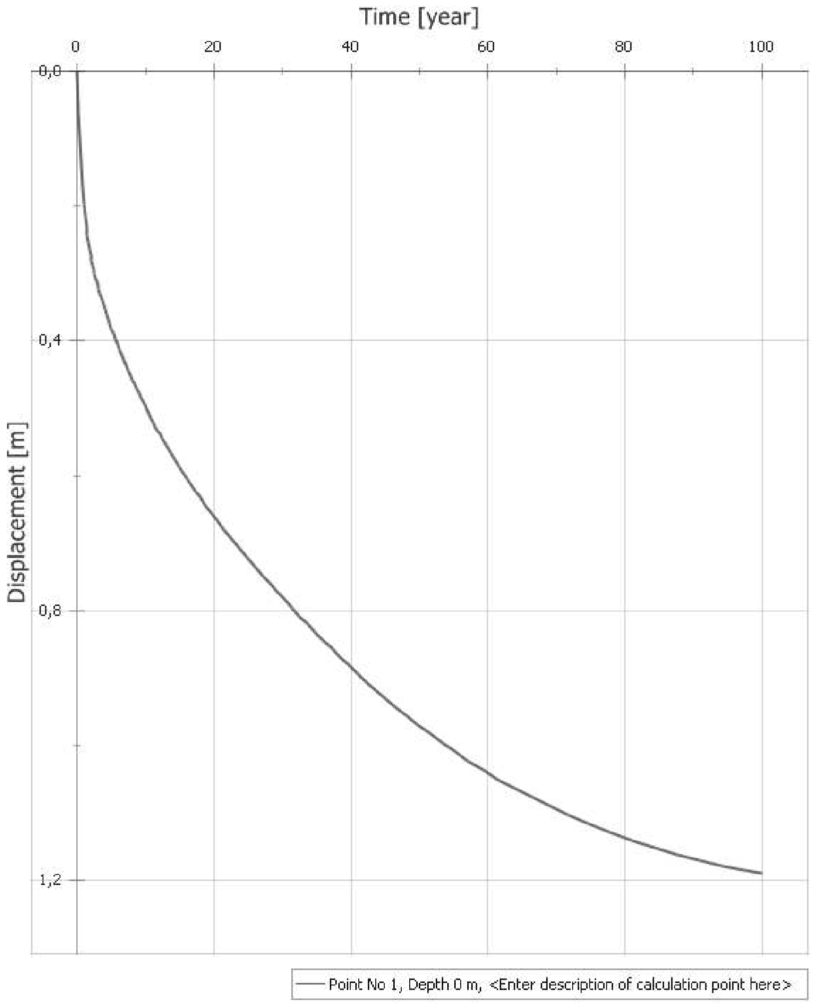
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,778	100,0000

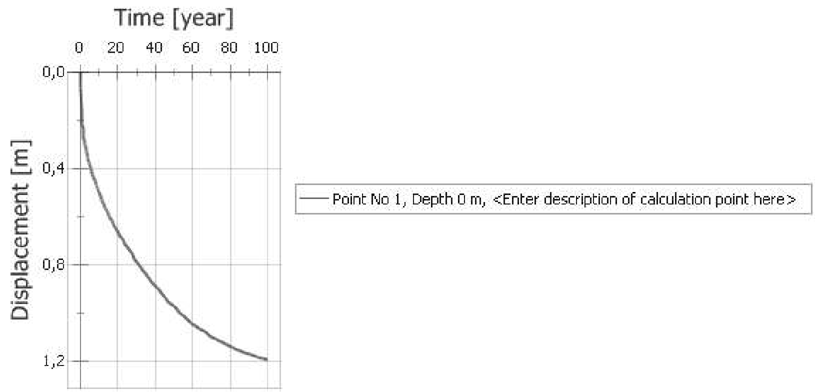
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

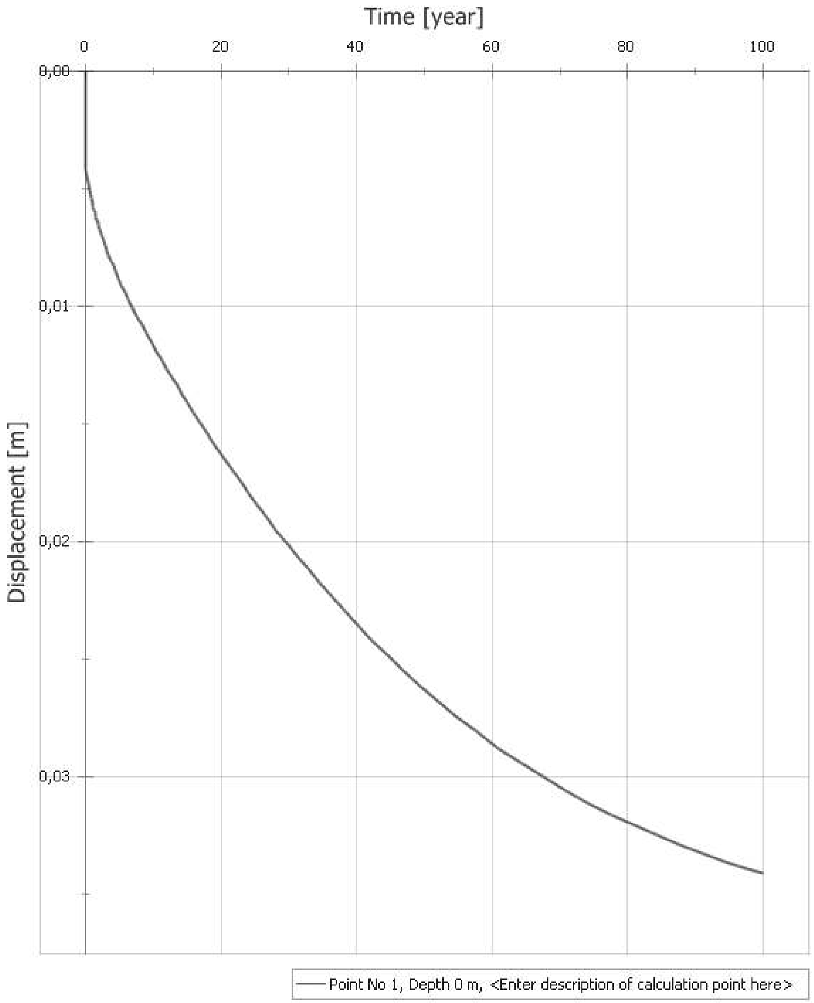
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	1,190	100,0000

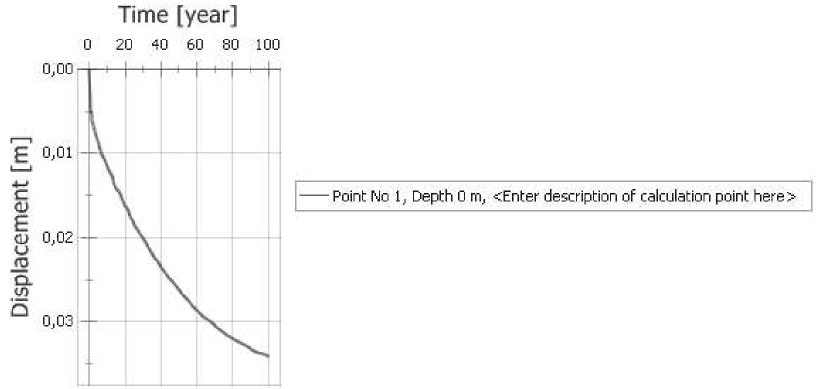
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

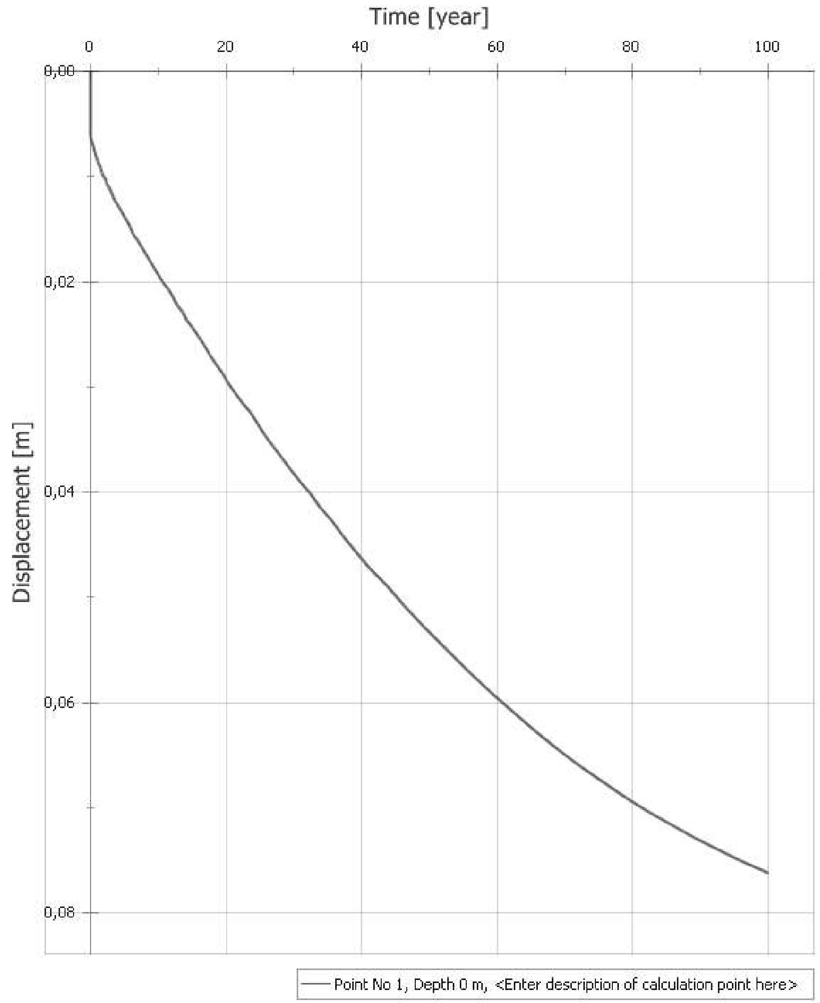
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,034	100,0000

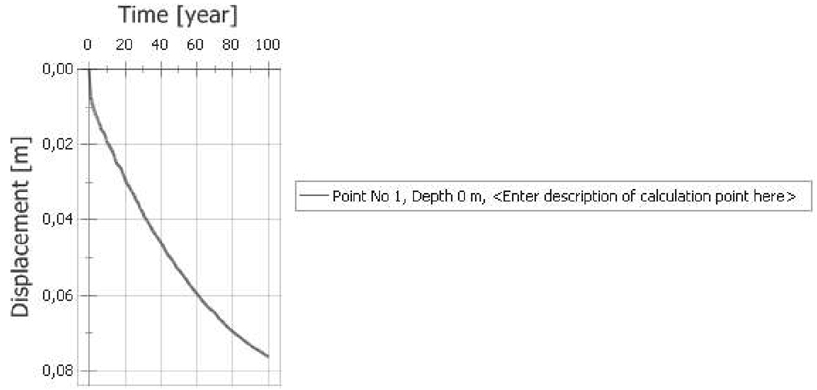
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

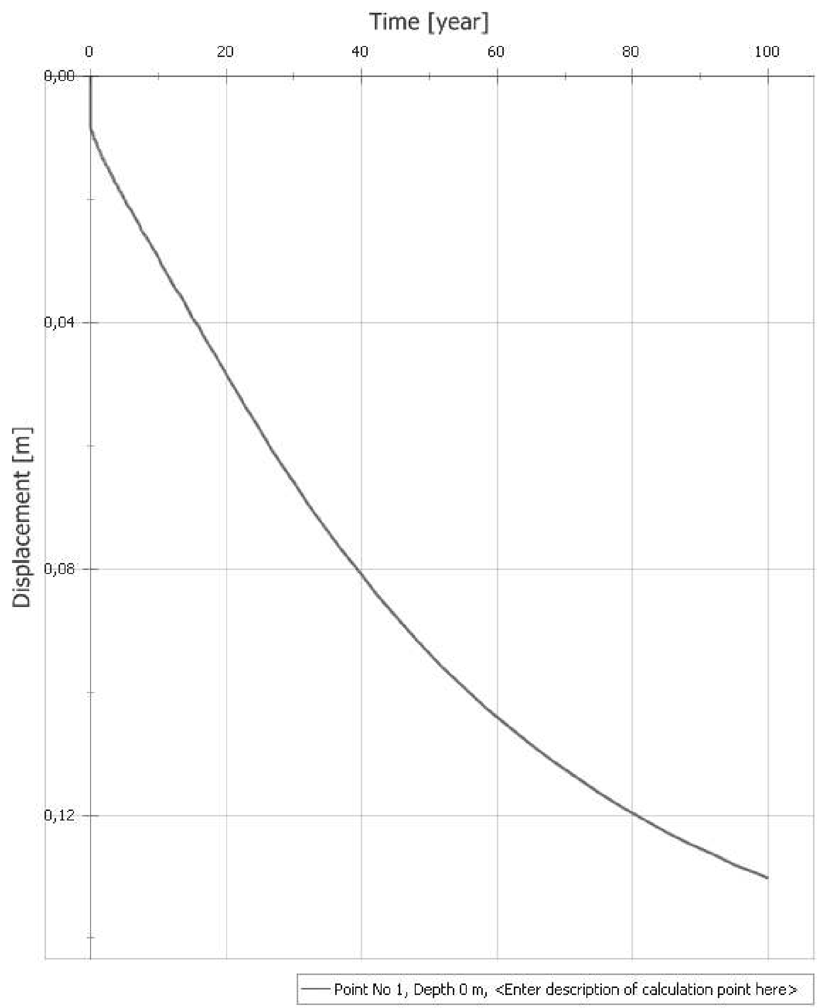
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,076	100,0000

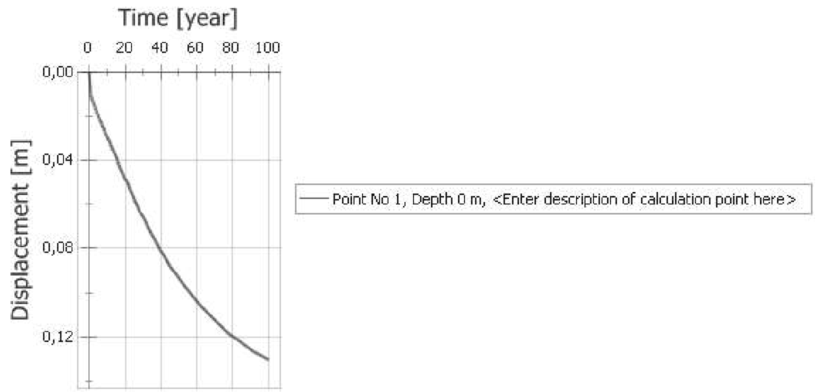
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

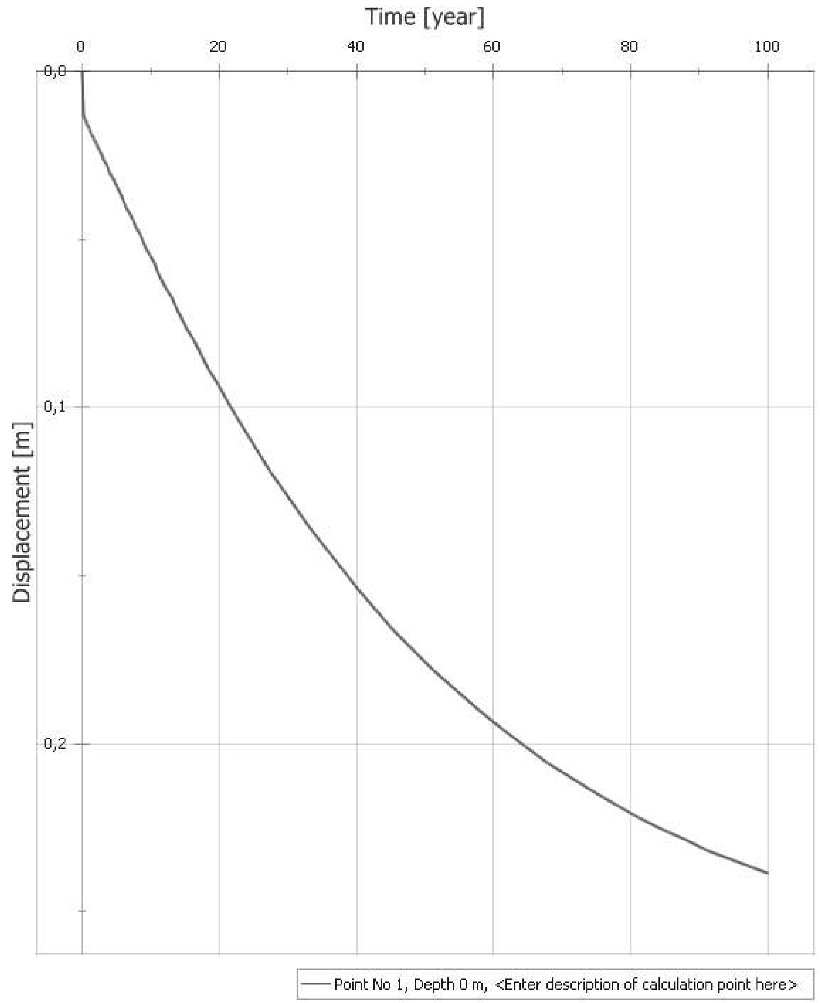
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,130	100,0000

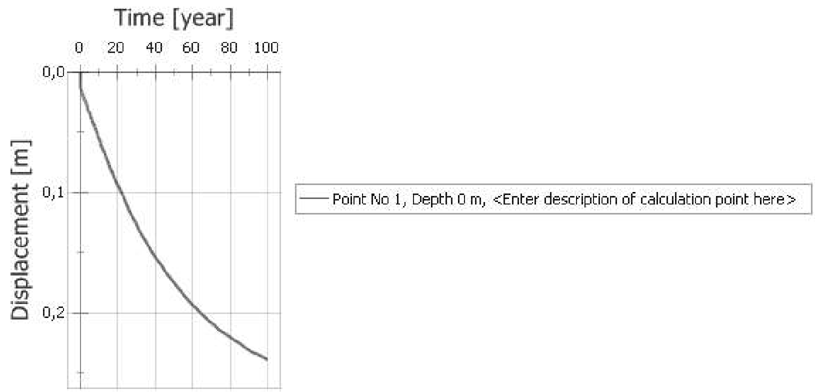
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

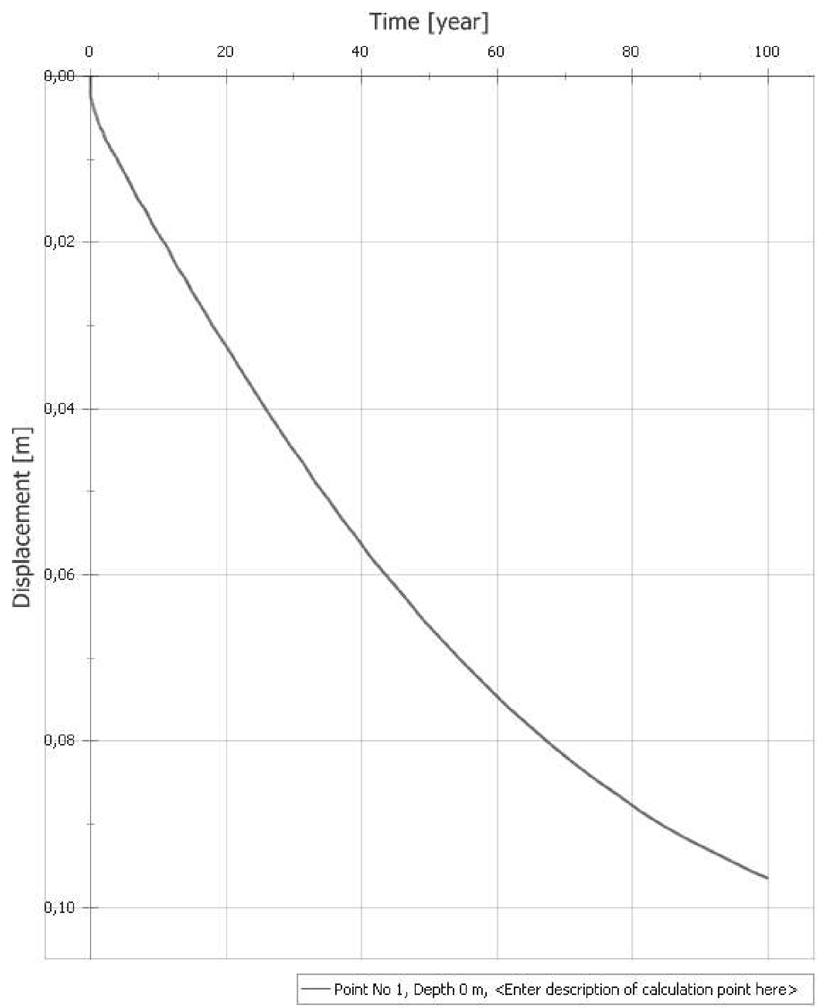
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,239	100,0000

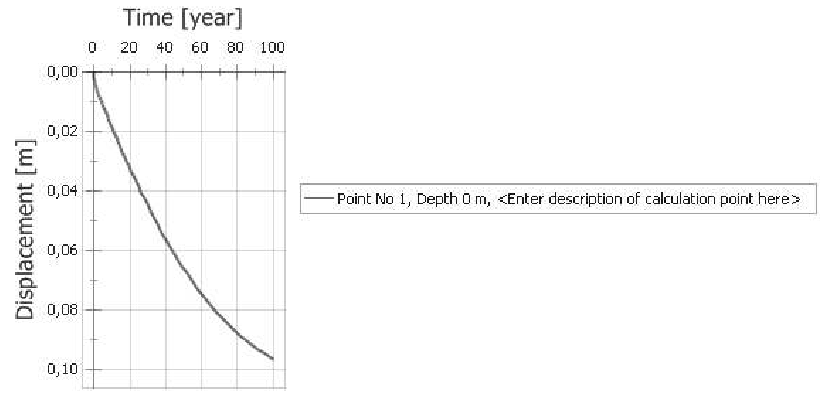
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

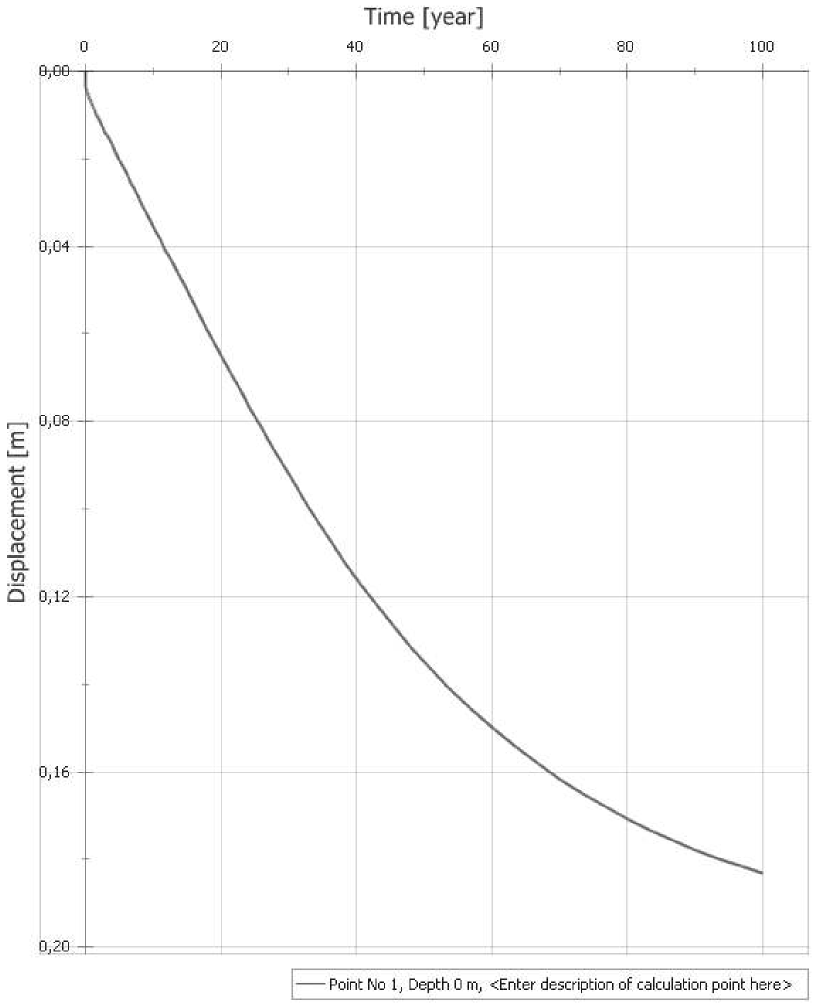
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,097	100,0000

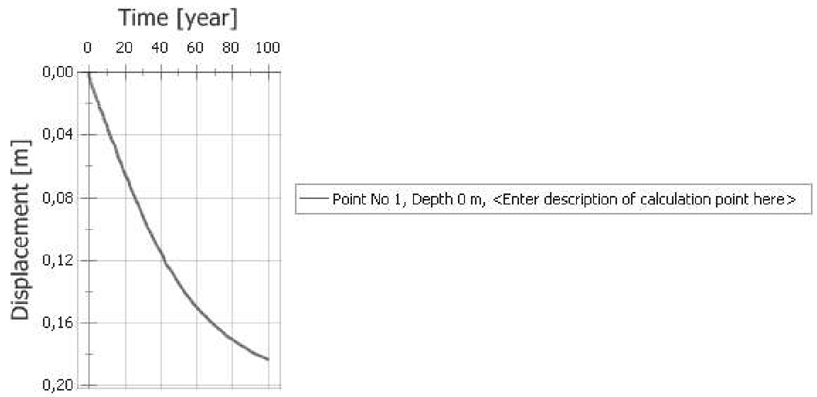
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

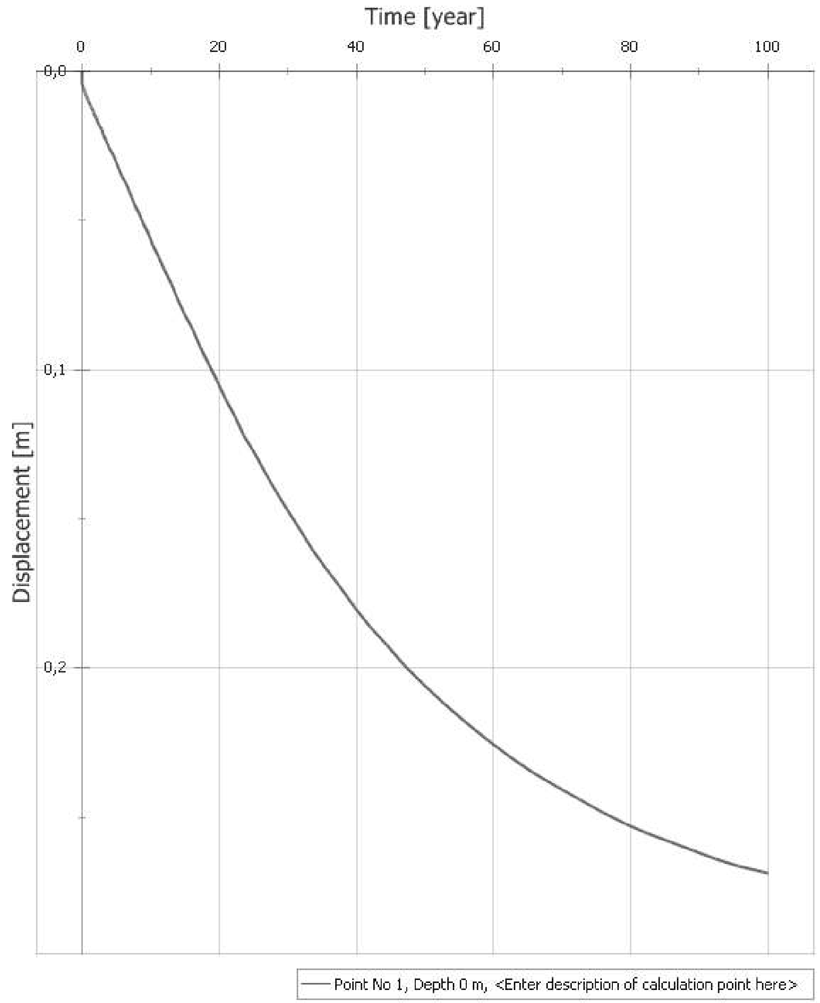
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,183	100,0000

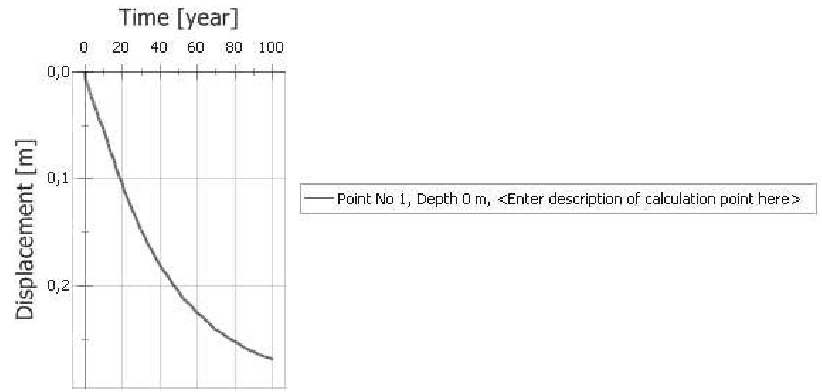
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

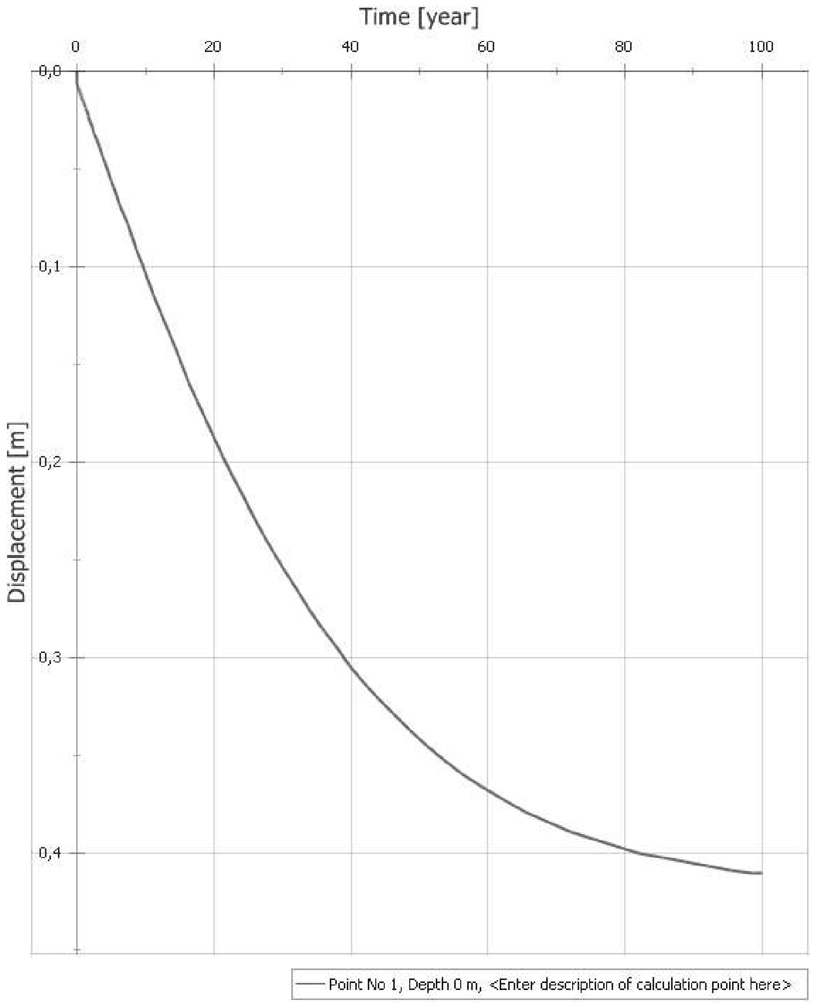
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,269	100,0000

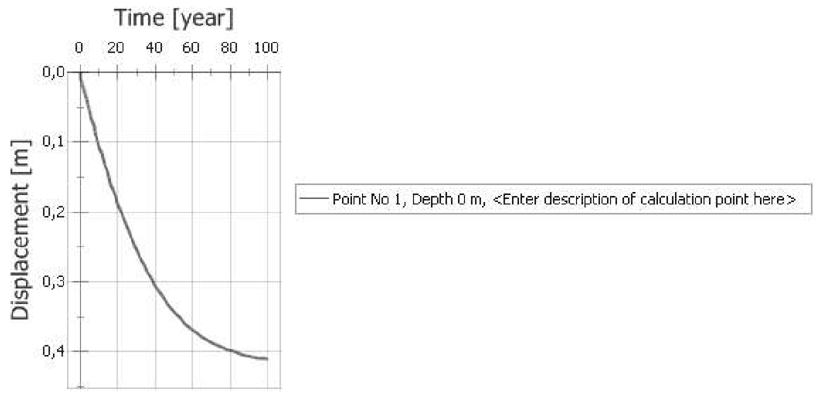
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

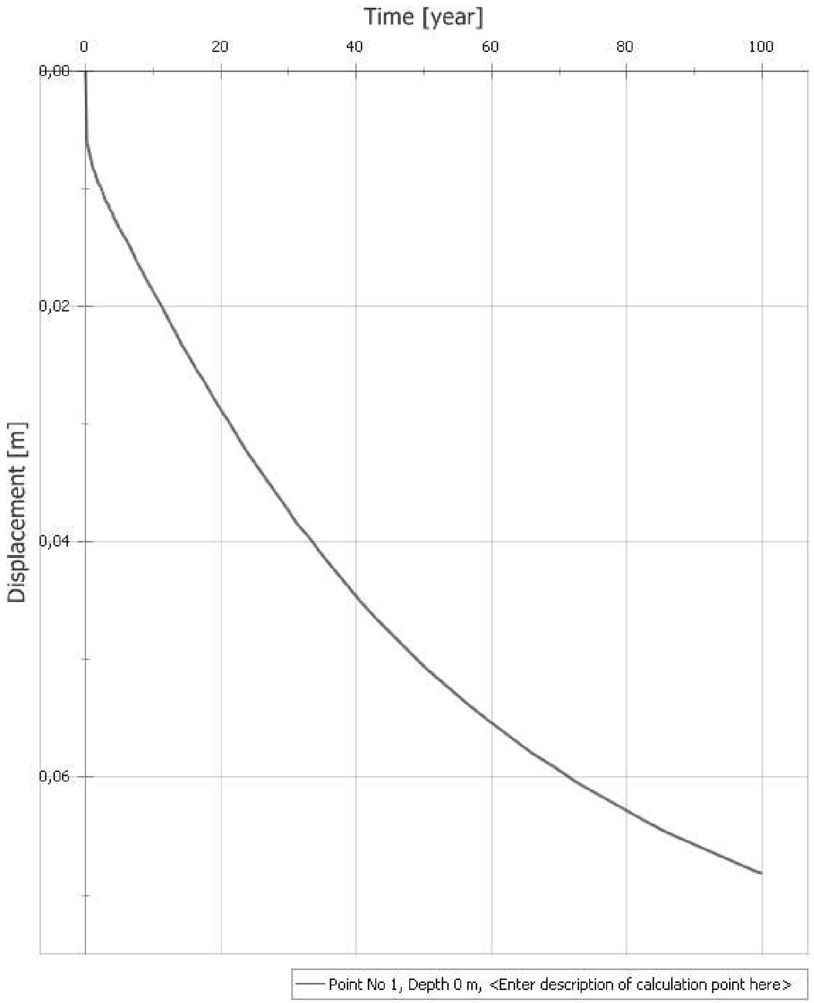
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,411	100,0000

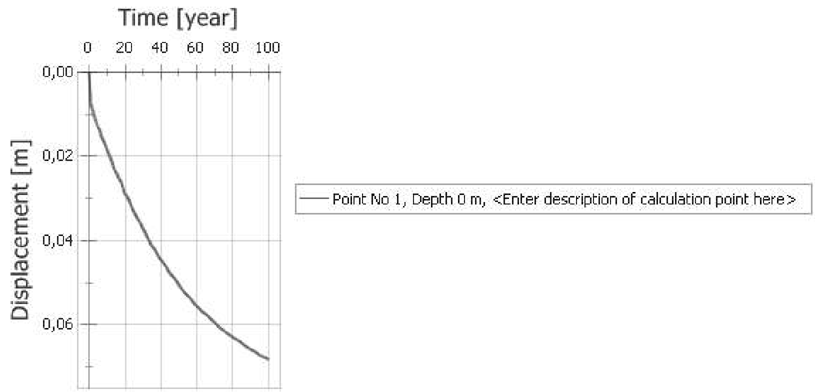
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

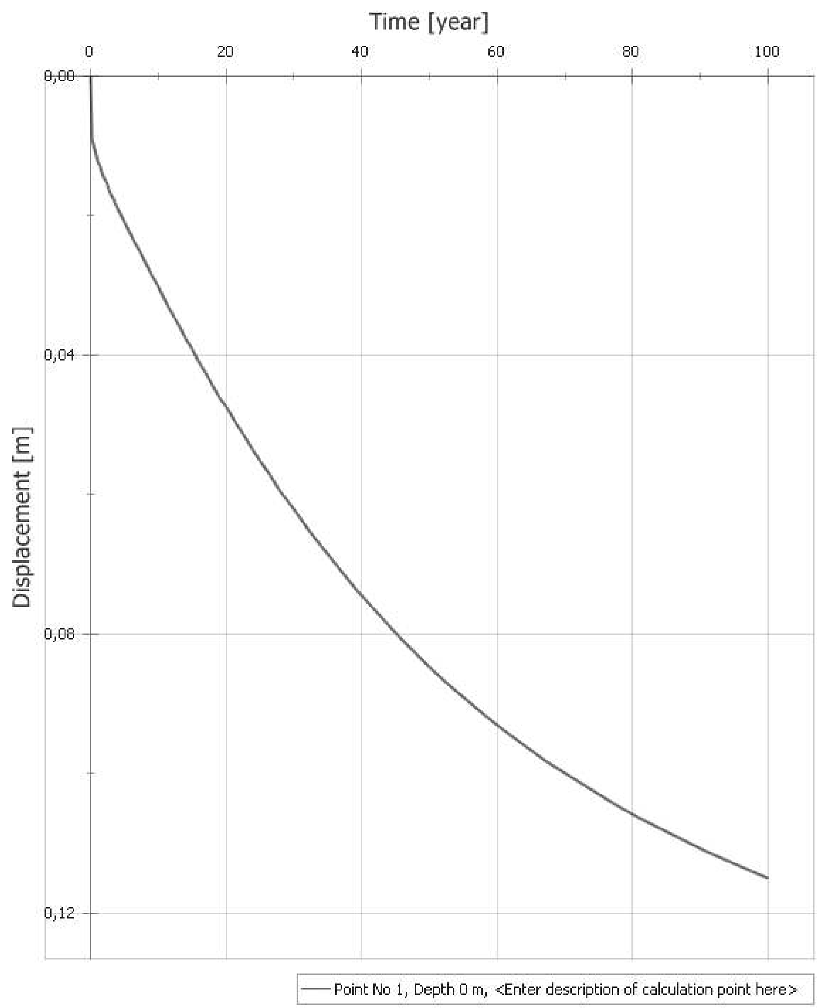
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,068	100,0000

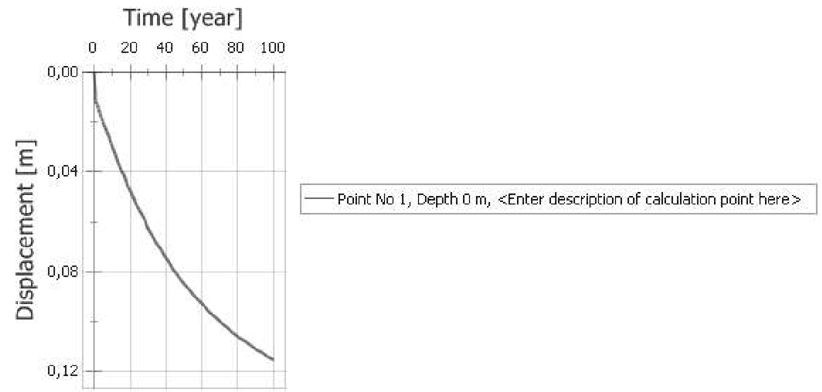
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

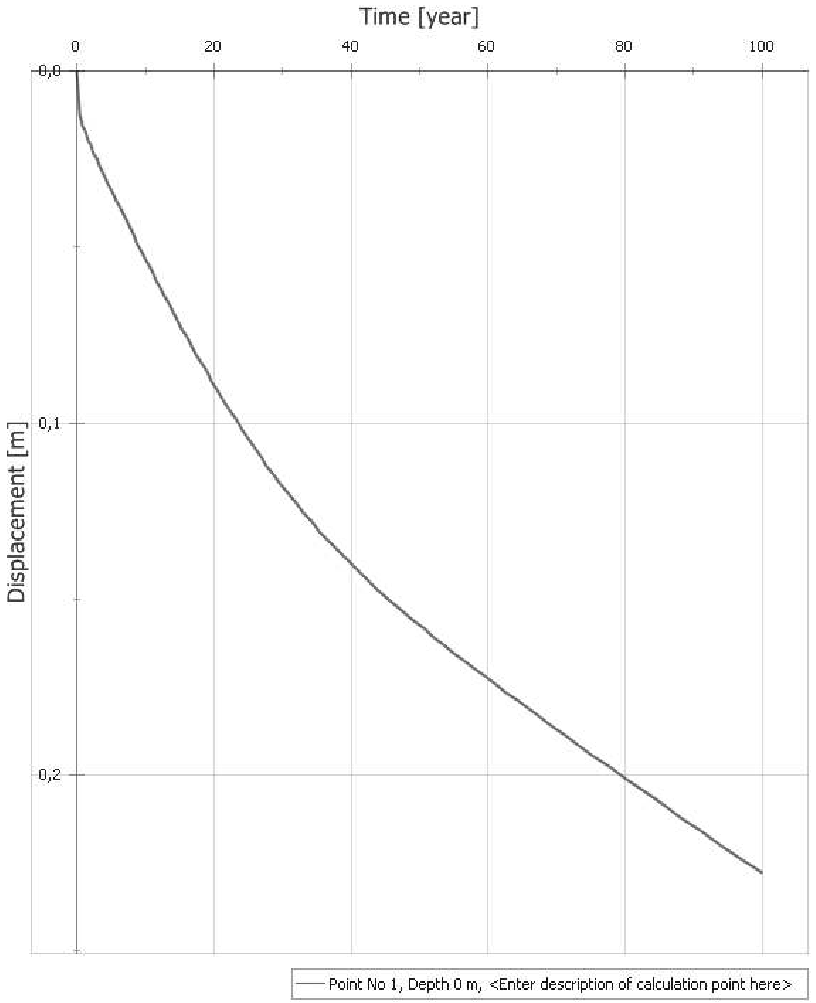
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,115	100,0000

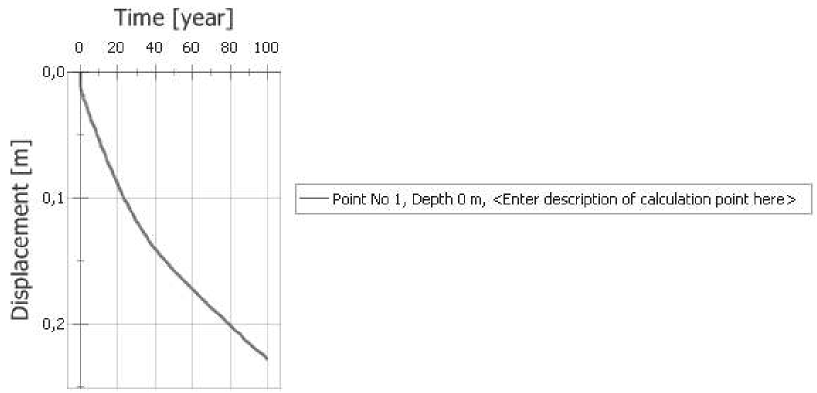
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

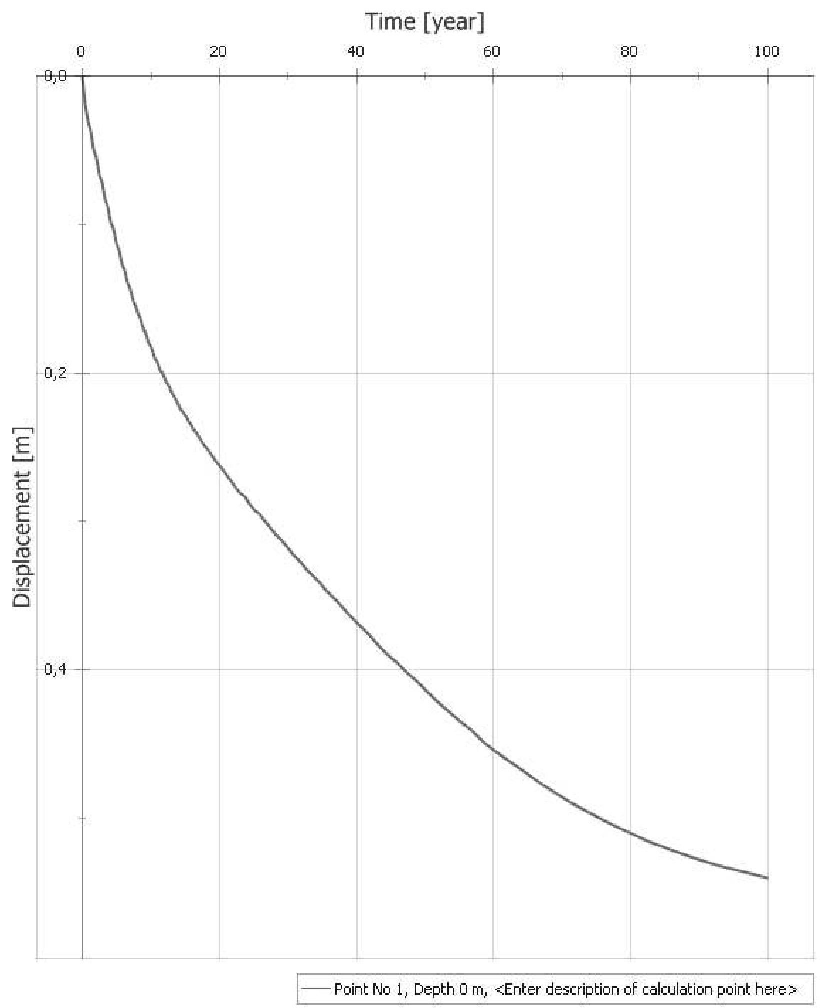
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,228	100,0000

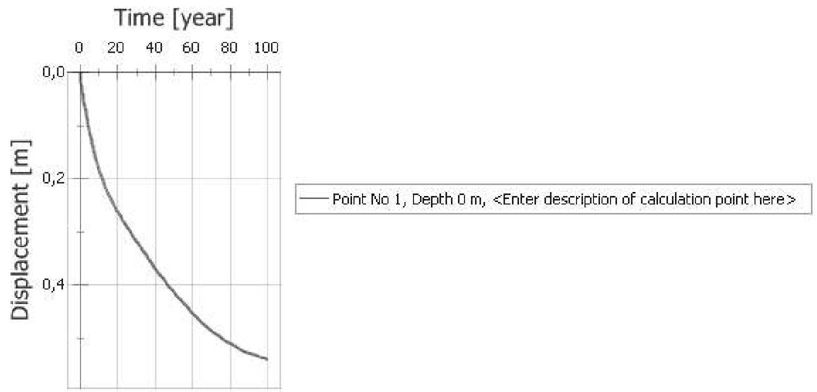
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

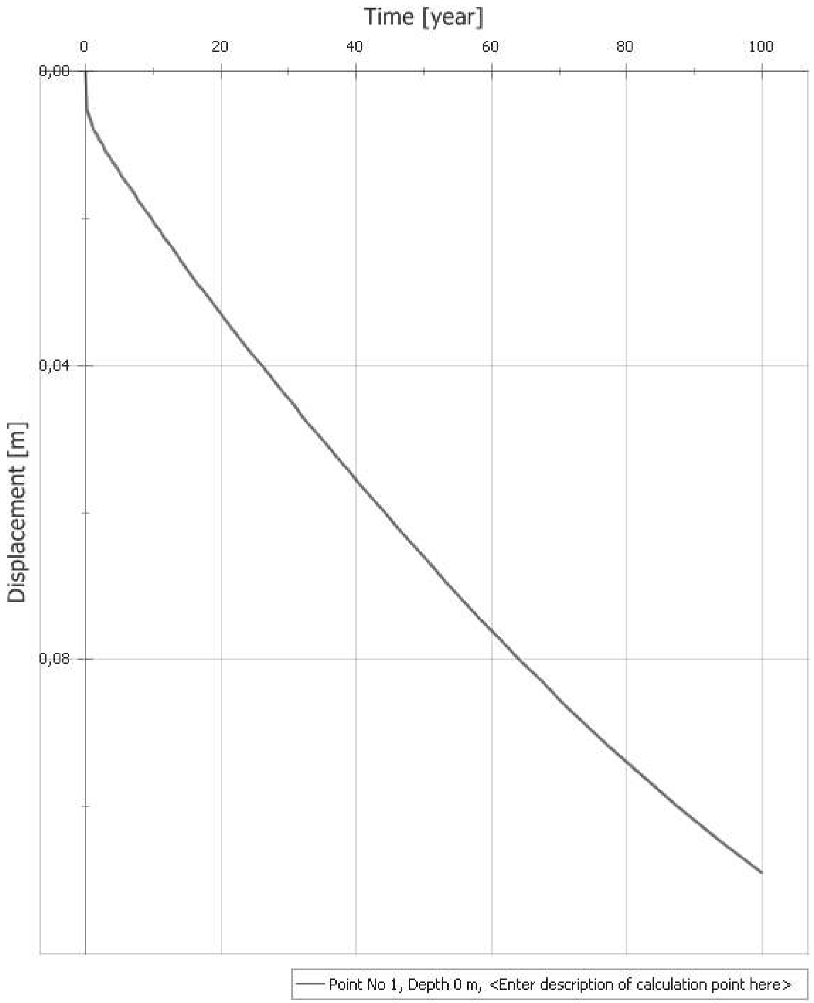
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,540	100,0000

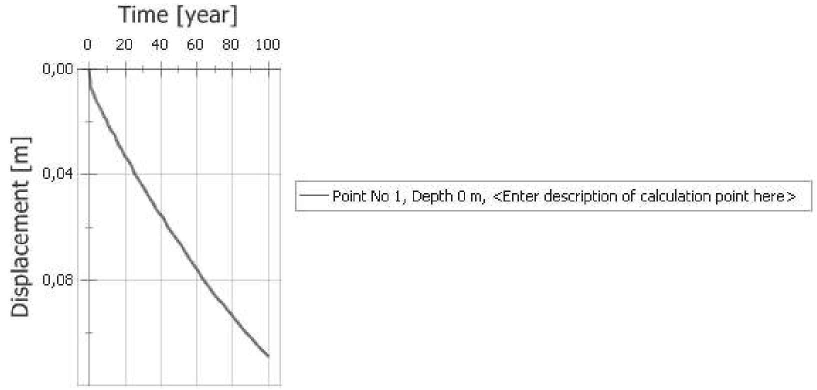
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

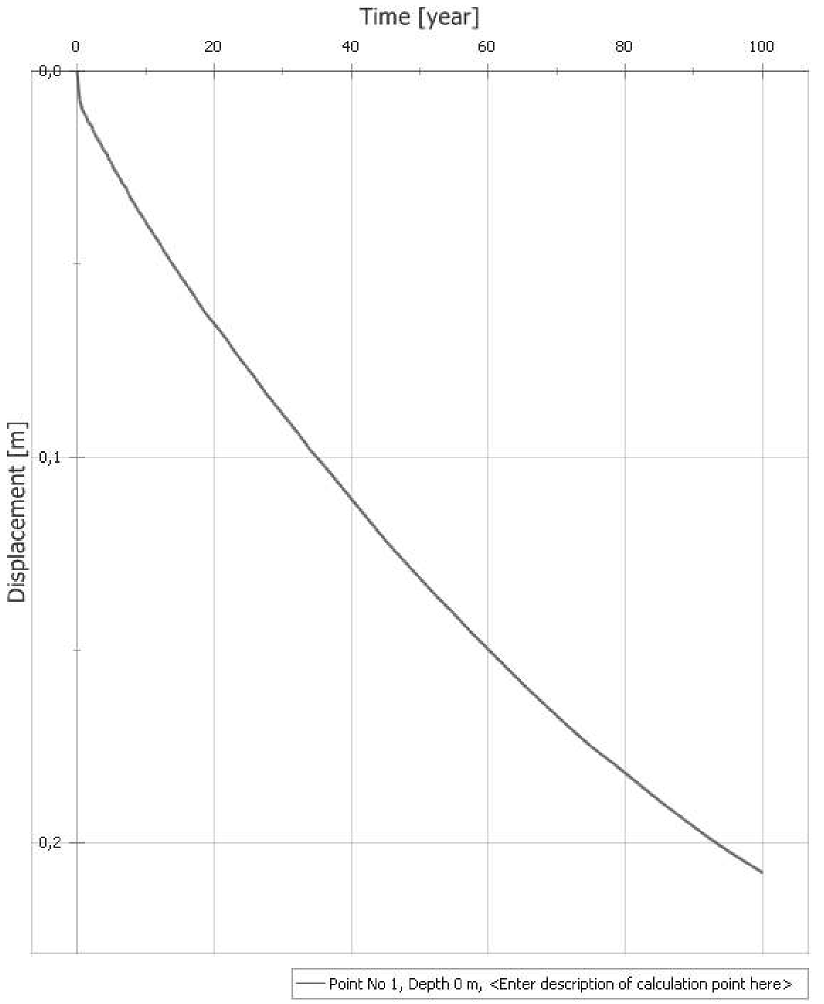
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,109	100,0000

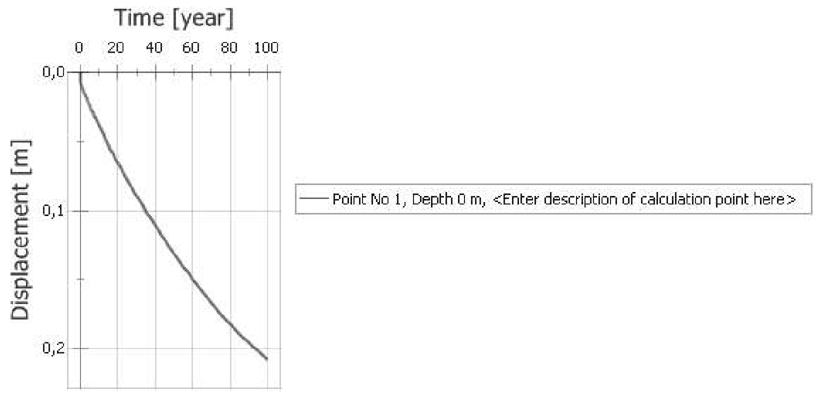
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

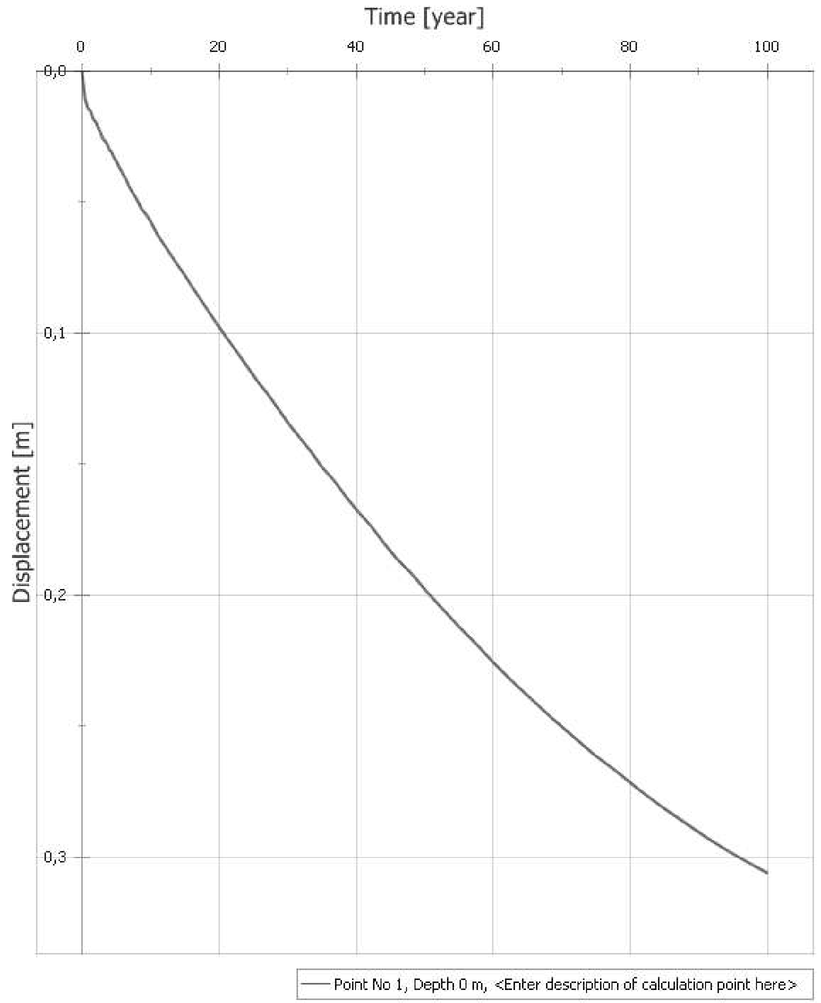
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,208	100,0000

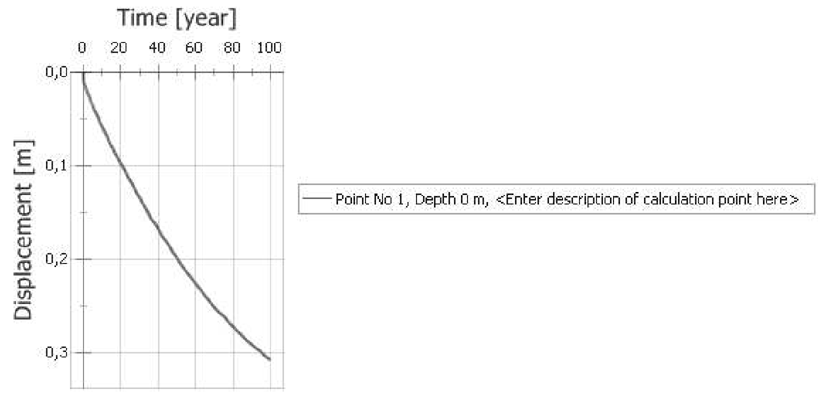
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

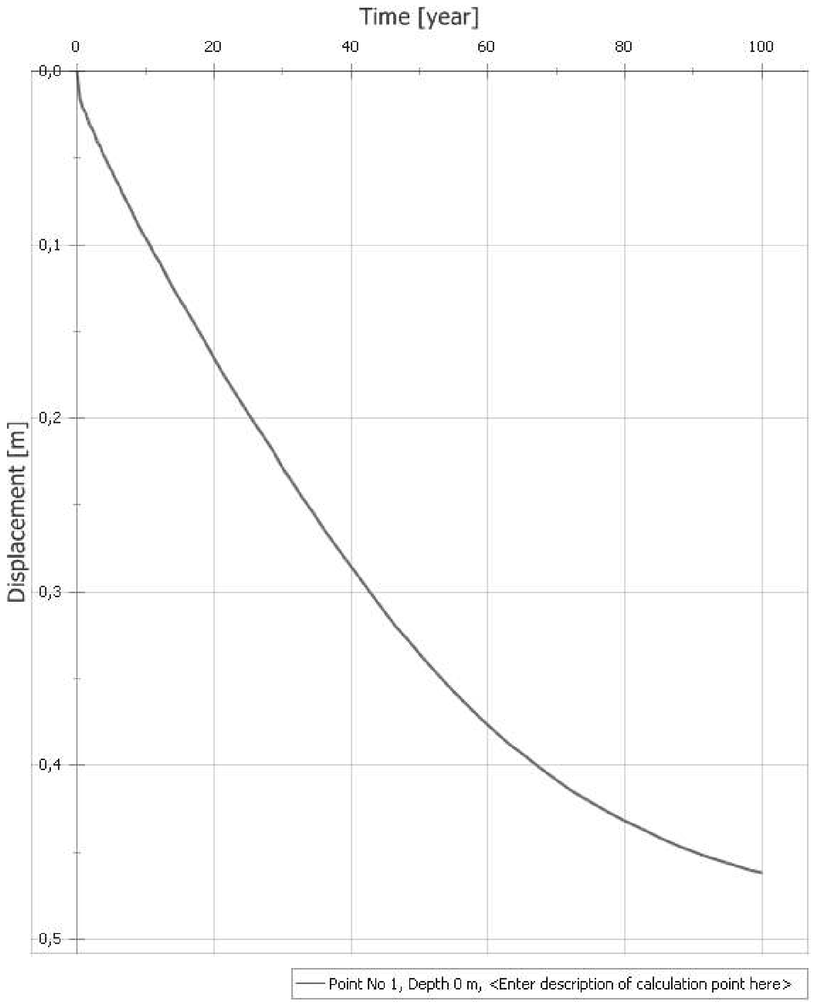
Point No 1, <Enter description of calculation point here>



Depth [m]	Displacement [m]	Time [years]
0,00	0,306	100,0000

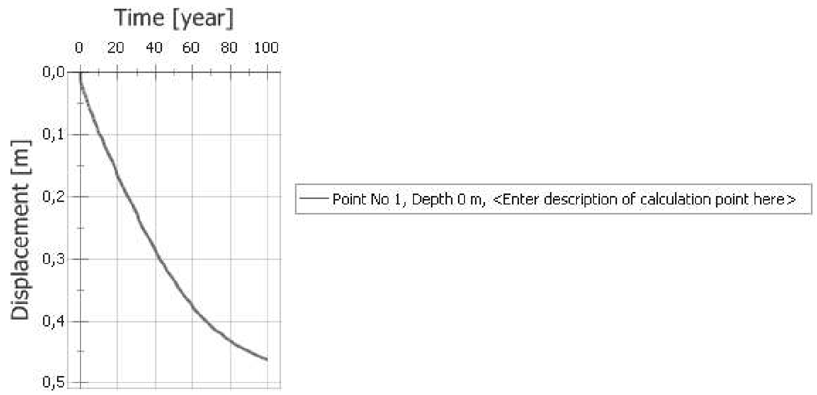
Displacement versus Time - Graph

Displacement versus Time - Graph for Point No 1, <Enter description of calculation point here>



Summary

Point No 1, <Enter description of calculation point here>



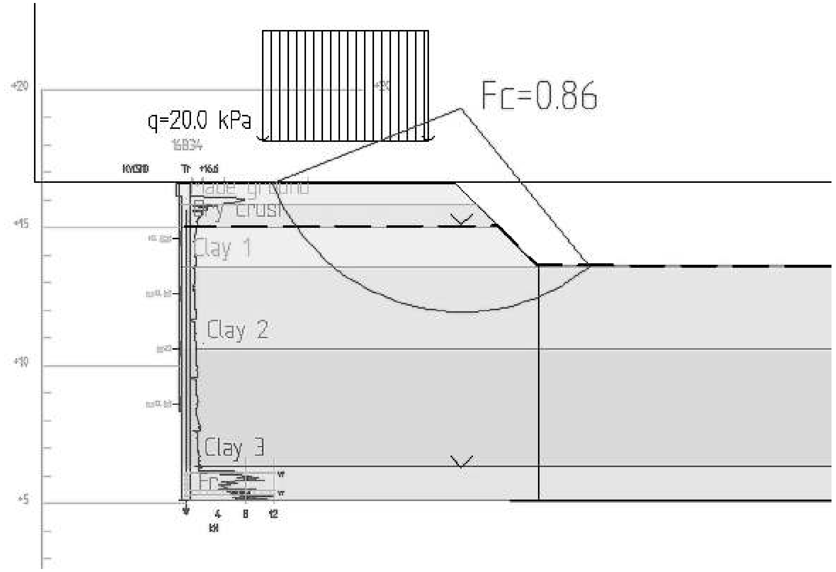
Depth [m]	Displacement [m]	Time [years]
0,00	0,462	100,0000

Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,50			8,9	1,00	1,00	1,00
Clay 2	16,80			12,3	1,00	1,00	1,00
Clay 3	17,10			18,6	1,00	1,00	1,00
Fr	19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

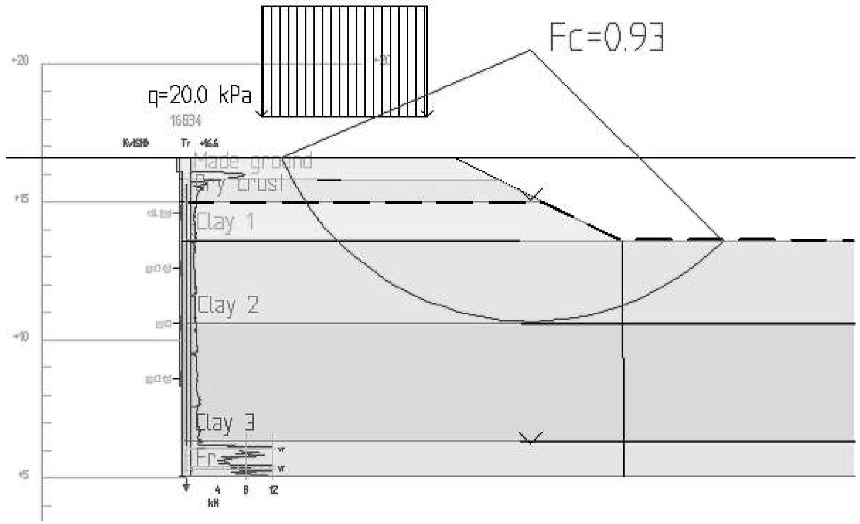


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,50			8,9	1,00	1,00	1,00
Clay 2	16,80			12,3	1,00	1,00	1,00
Clay 3	17,10			18,6	1,00	1,00	1,00
Fr	19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

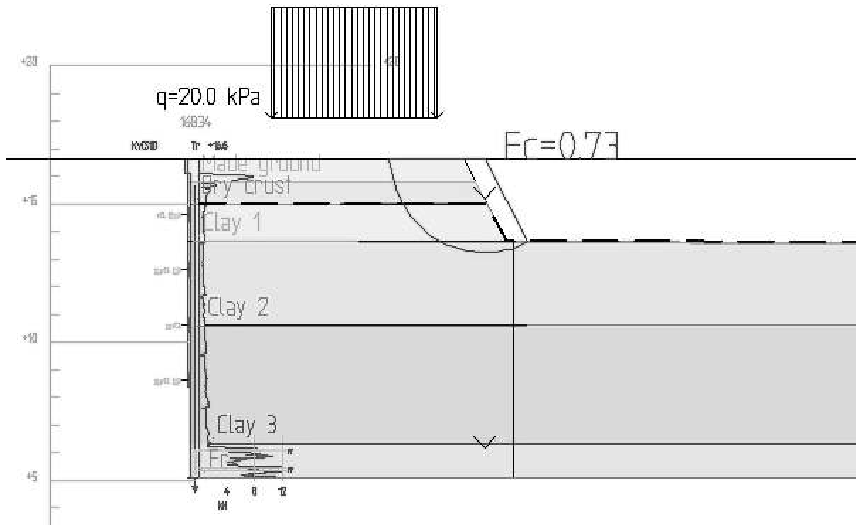


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,50			8,9	1,00	1,00	1,00
Clay 2	16,80			12,3	1,00	1,00	1,00
Clay 3	17,10			18,6	1,00	1,00	1,00
Fr	19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

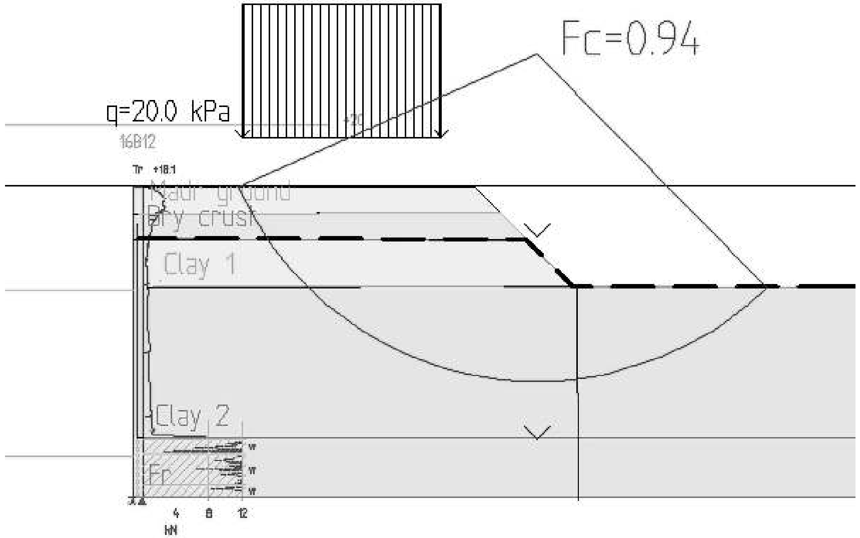


Soil

Materials

Material	ρ [kN/m^3]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Madr ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			10,3	1,00	1,00	1,00
Clay 2	16,90			13,0	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

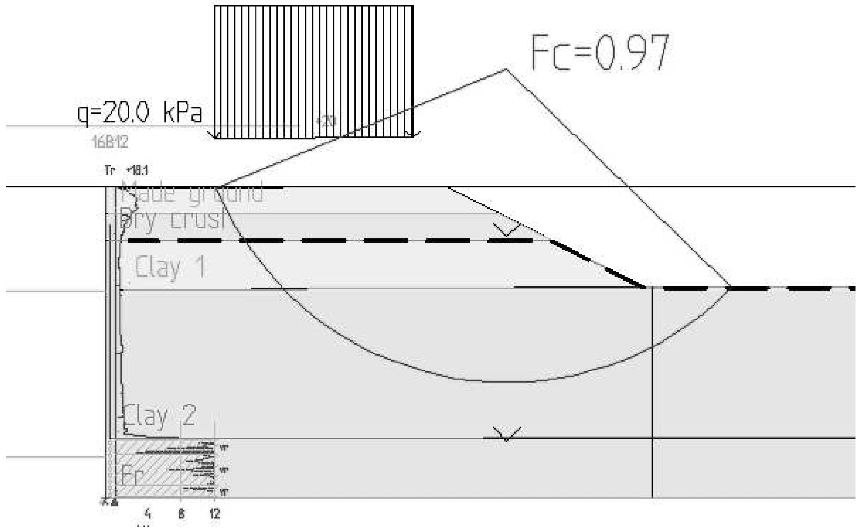


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			10,3	1,00	1,00	1,00
Clay 2	16,90			13,0	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

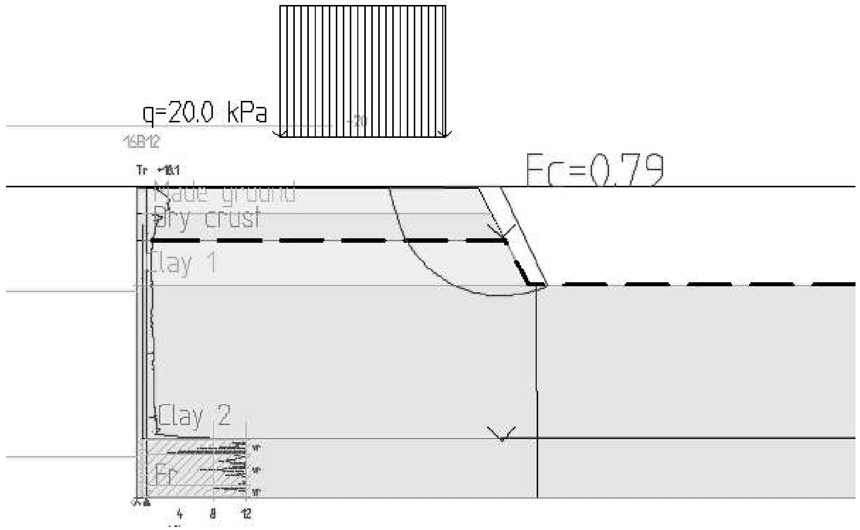


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			10,3	1,00	1,00	1,00
Clay 2	16,90			13,0	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

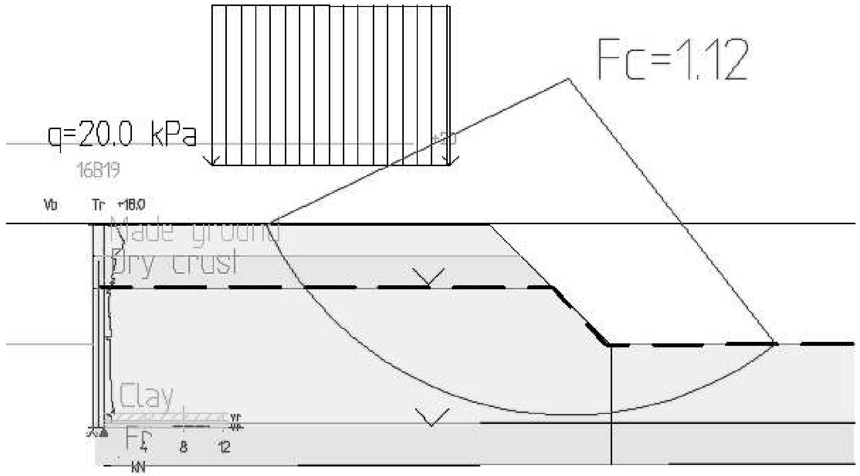


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		18,20			15,0	1,00	1,00	1,00
Fr		19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

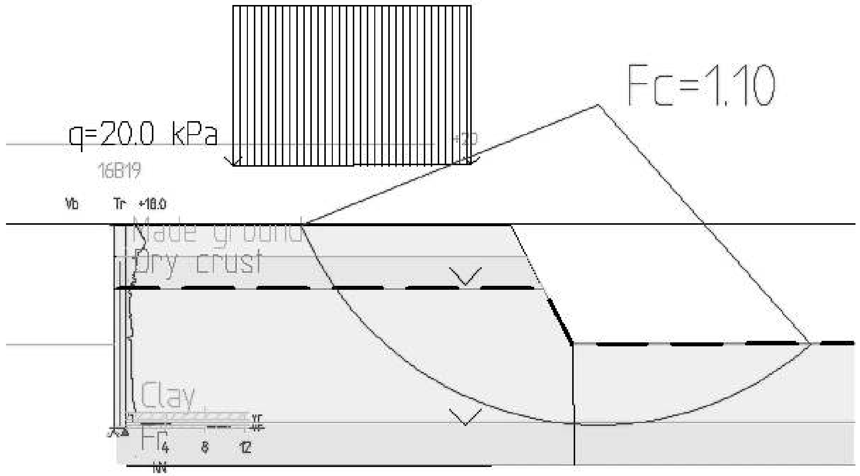


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay	18,20			15,0	1,00	1,00	1,00
Fr	19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

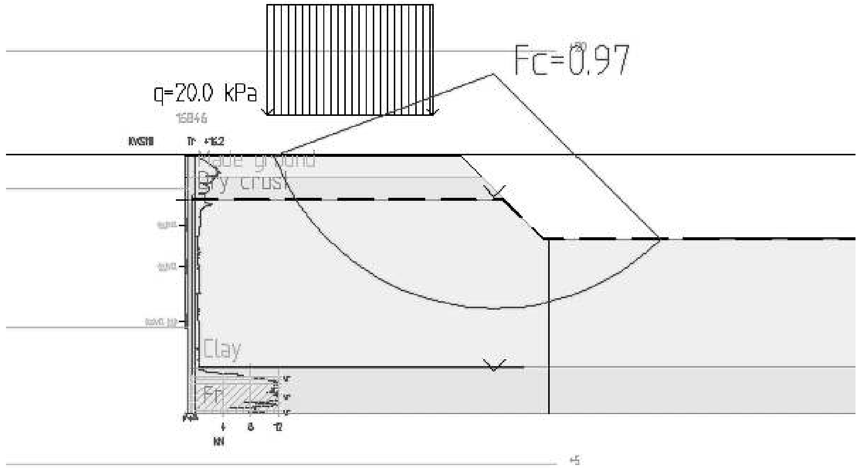


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			13,5	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

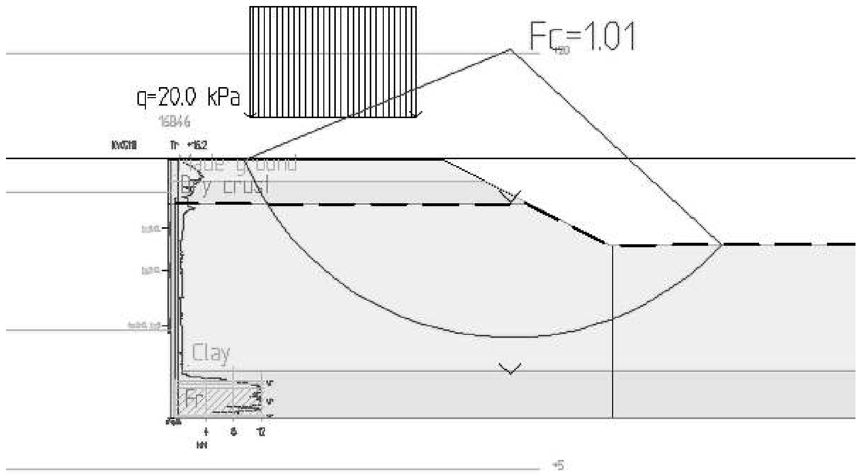


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			13,5	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

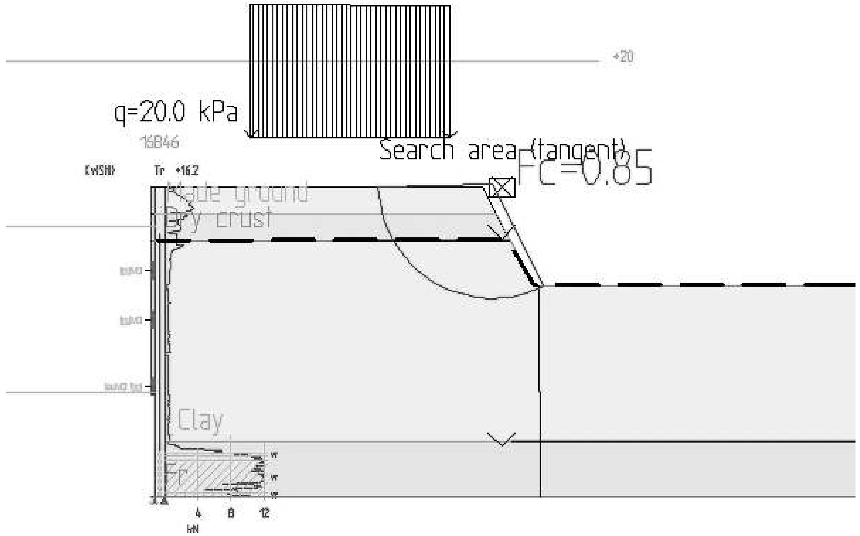


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			13,5	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model



Soil

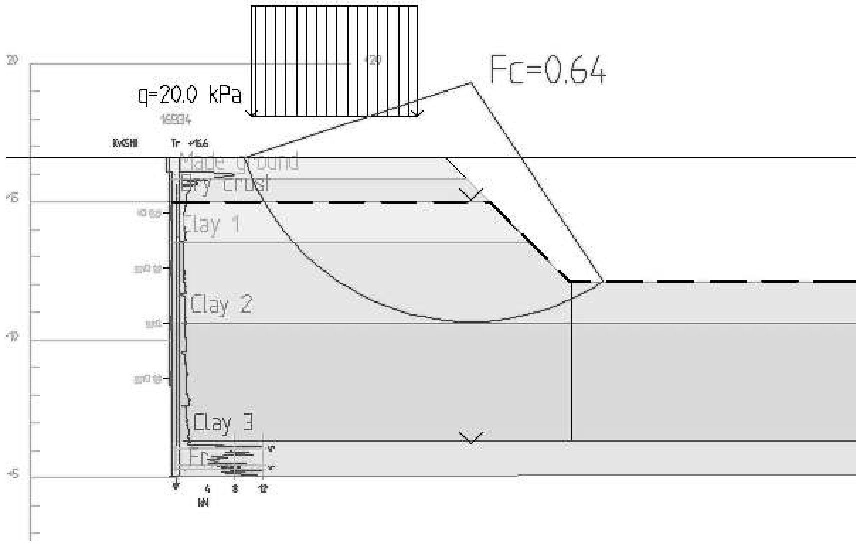
Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,50			8,9	1,00	1,00	1,00
Clay 2		16,80			12,3	1,00	1,00	1,00
Clay 3		17,10			18,6	1,00	1,00	1,00
Fr		19,00	36,0	0,0		1,00	1,00	1,00

Novapoint

16B34 11 4,5M NEW
2016-06-21 11:35
GS Stability, version: 15.0.1.0

Graphic Model



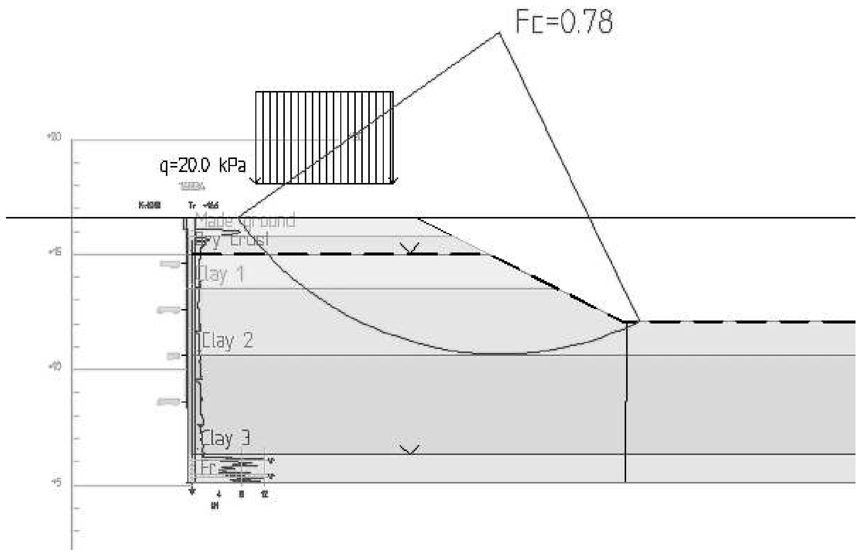
Novapoint

Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		18,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,50			8,9	1,00	1,00	1,00
Clay 2		16,80			12,3	1,00	1,00	1,00
Clay 3		17,10			18,6	1,00	1,00	1,00
Fr		19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

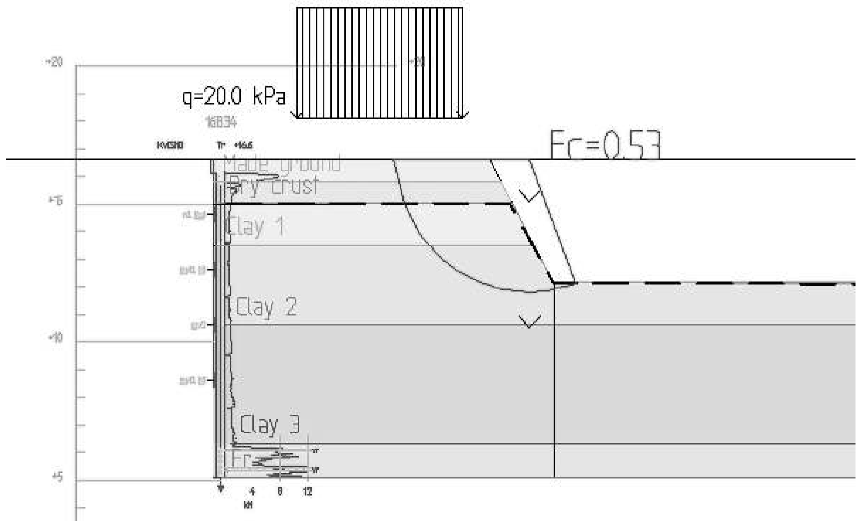


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,50			8,9	1,00	1,00	1,00
Clay 2		16,80			12,3	1,00	1,00	1,00
Clay 3		17,10			18,6	1,00	1,00	1,00
Fr		19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

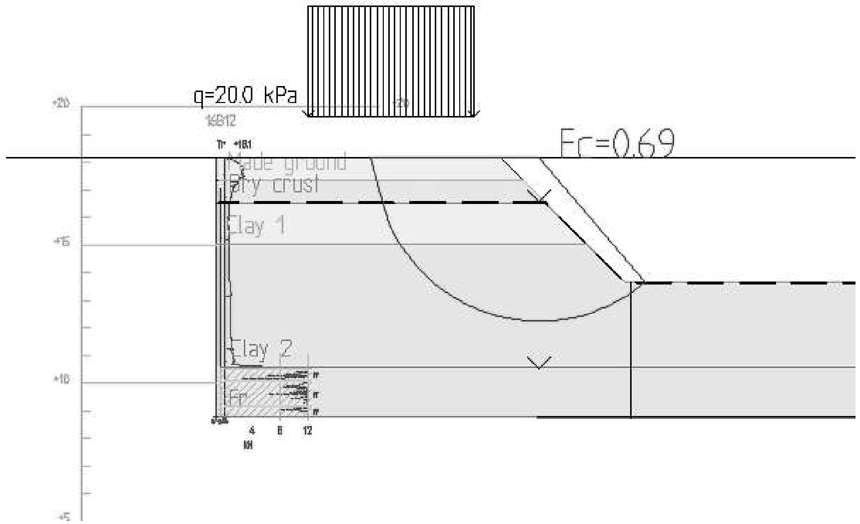


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			10,3	1,00	1,00	1,00
Clay 2	16,90			13,0	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

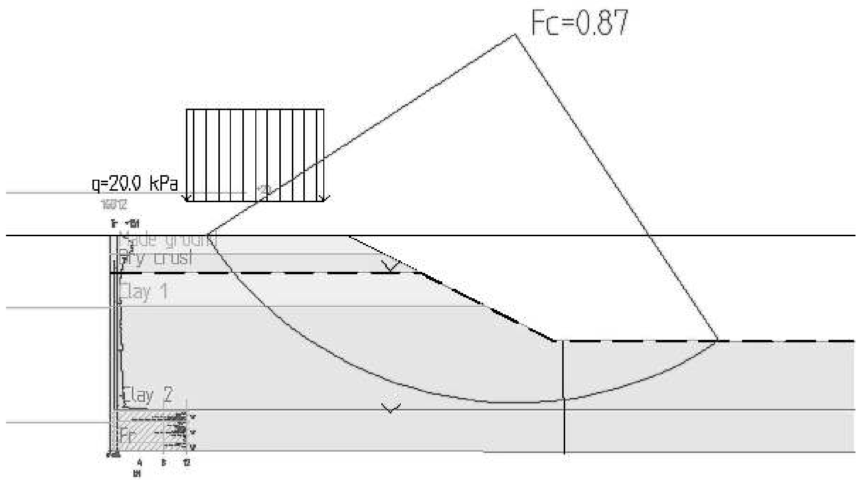


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,30			10,3	1,00	1,00	1,00
Clay 2		16,90			13,0	1,00	1,00	1,00
Fr		19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

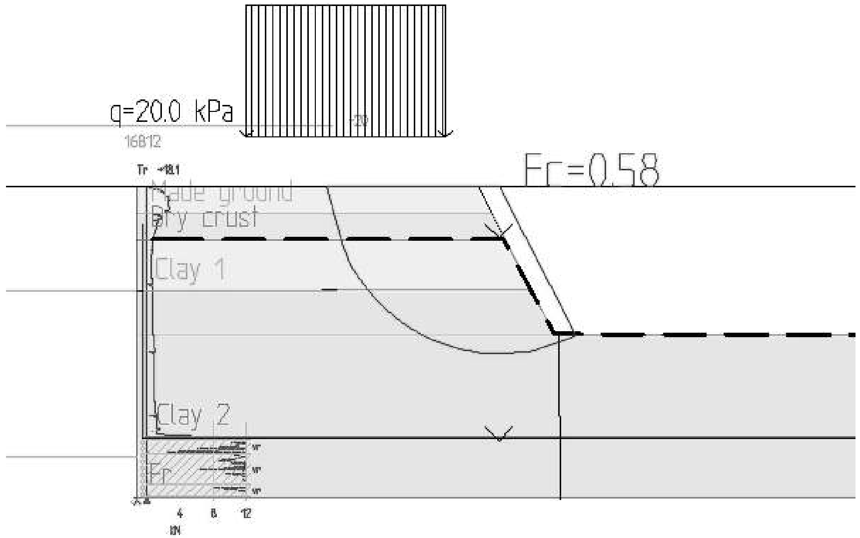


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,30			10,3	1,00	1,00	1,00
Clay 2		16,90			13,0	1,00	1,00	1,00
Fr		19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

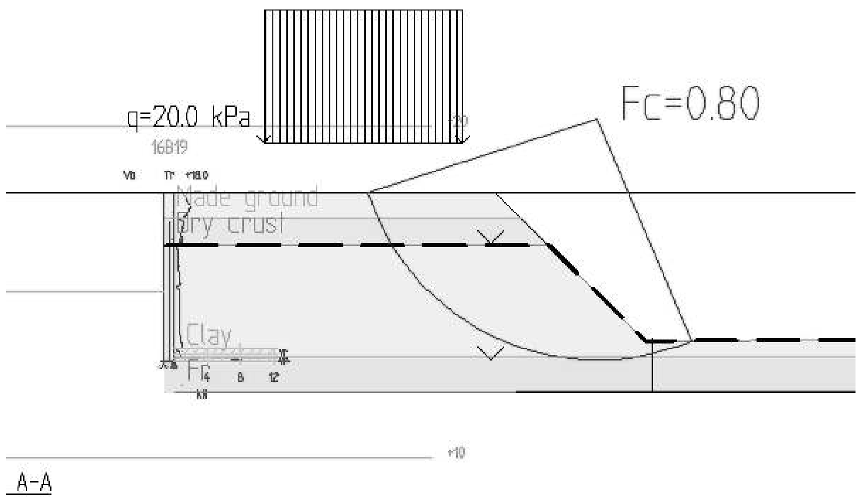


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		18,20			15,0	1,00	1,00	1,00
Fr		19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

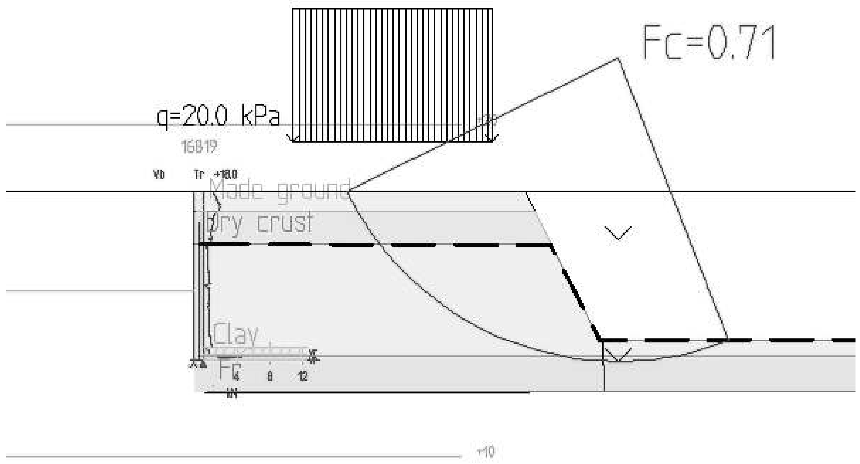


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay	18,20			15,0	1,00	1,00	1,00
Fr	19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model



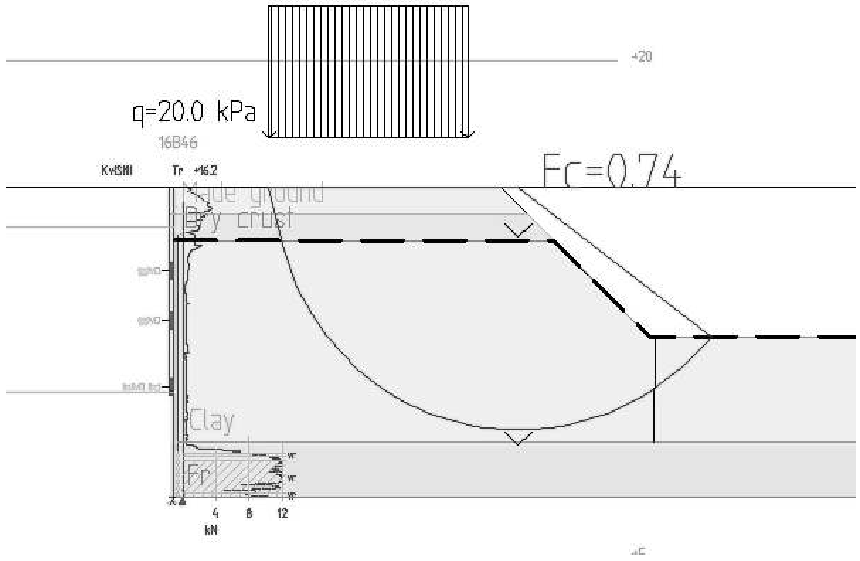
Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			13,5	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

16B46 11 4,5M NEW
2016-06-21 11:32
GS Stability, version: 15.0.1.0

Graphic Model

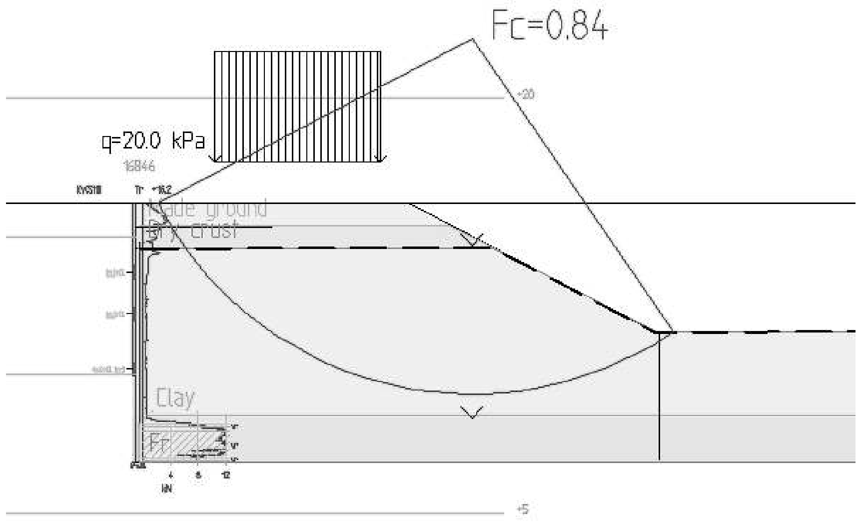


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			13,5	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

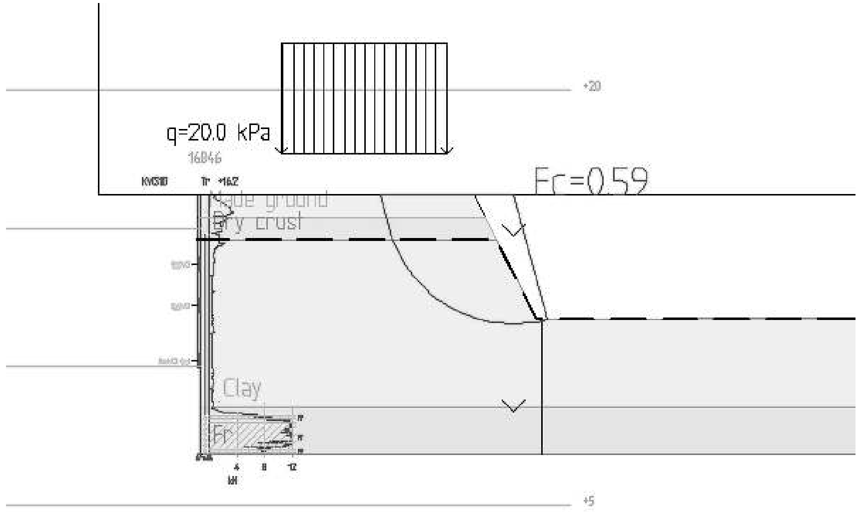


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			13,5	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model



14W44

GWT (m)	1
diameter col. (m)	0.6
c_v	0.0000000073
k col / k clay	500
Creep strength	0.65
Δq_1	0.1
Lenght col. (m)	8.3

h (m)	ρ (t/m^3)	σ'_c (kPa)	M_L (kPa)	σ'_L (kPa)	M´	M_0 (kPa)	M_{COL} (kPa)	τ_{COL} (kPa)
1	1.9	0	0	0	0	0	0	0
2	0.64	38	224	52	14.9	2401	7500	100
2	0.66	39	240	83	18	3375	7500	100
2	0.66	70	230	96	20.4	3315	7500	100
2.3	0.69	75	544	106	20.2	3713	7500	100
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0

c 1 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
0.8	0.442	20		30		40	
			0.0000		0.0000		0.0000
			0.0087		0.0131		0.0195
			0.0118		0.0177		0.0237
			0.0078		0.0115		0.0177
			0.0104		0.0165		0.0225
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.039		0.059		0.083

U (%)	time (days)
10	5.21
20	11.03
30	17.64
40	25.26
50	34.27
60	45.31
70	59.53
80	79.58
90	113.85
99	227.70

14W44

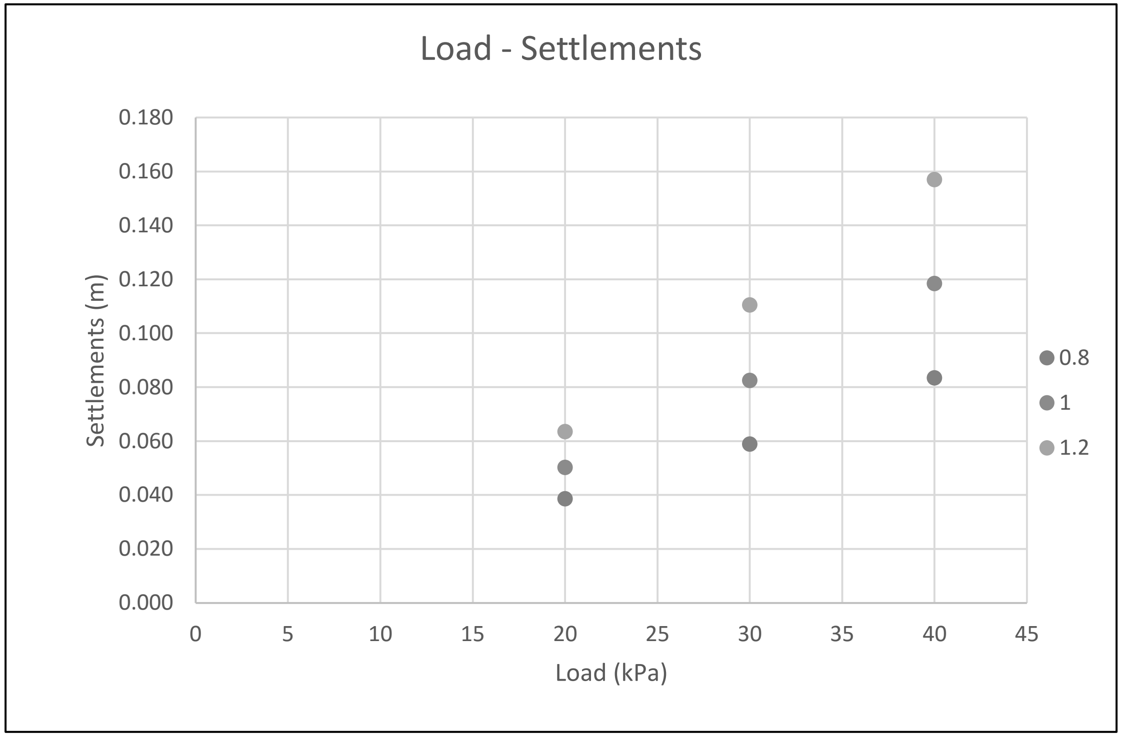
c 2 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1	0.283	20		30		40	
			0.0000		0.0000		0.0000
			0.0105		0.0187		0.0274
			0.0174		0.0256		0.0349
			0.0089		0.0159		0.0244
			0.0134		0.0222		0.0317
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.050		0.082		0.118

U (%)	time (days)
10	12.66
20	26.81
30	42.86
40	61.39
50	83.29
60	110.11
70	144.68
80	193.40
90	276.70
99	553.40

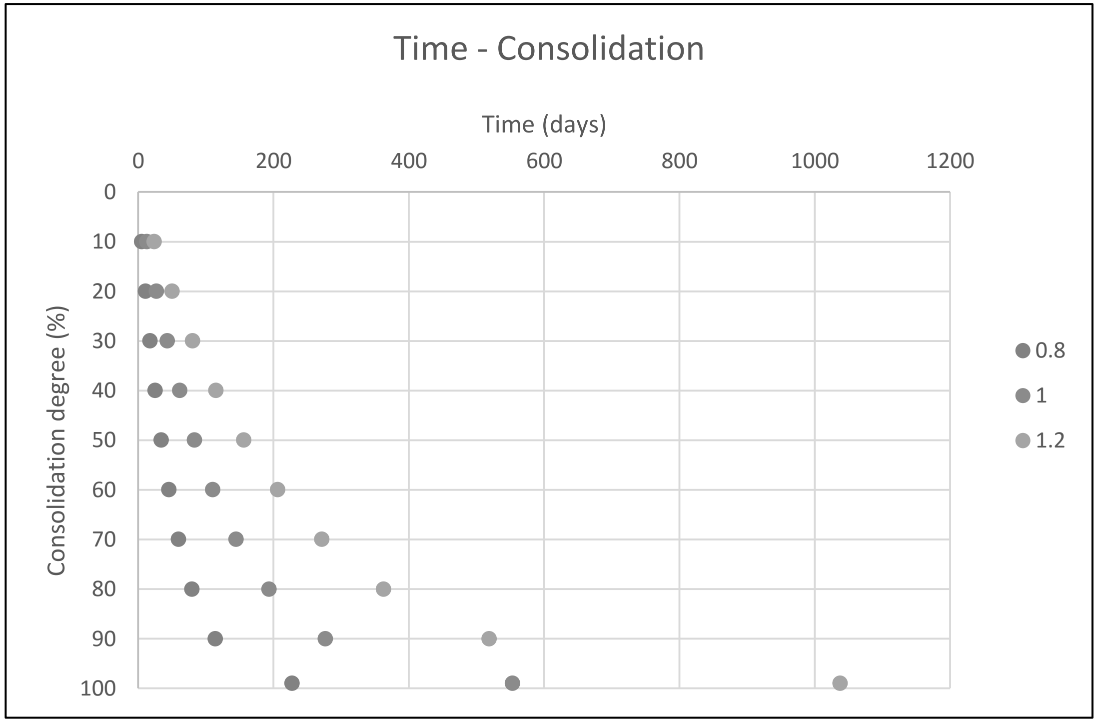
c 3(m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1.2	0.196	20		30		40	
			0.0000		0.0000		0.0000
			0.0135		0.0257		0.0373
			0.0233		0.0358		0.0472
			0.0097		0.0199		0.0318
			0.0169		0.0290		0.0406
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.063		0.110		0.157

U (%)	time (days)
10	23.74
20	50.27
30	80.35
40	115.08
50	156.15
60	206.42
70	271.23
80	362.57
90	518.72
99	1037.44

14W44



Load - Settlements
Settlements (m)
0.180
0.160
0.140
0.120
0.100
0.080
0.060
0.040
0.020
0.000
0
5
10
15
20
25
30
35
40
45
Load (kPa)
0.8
1
1.2



Time - Consolidation
Time (days)
0
200
400
600
800
1000
1200
Consolidation degree (%)
0
10
20
30
40
50
60
70
80
90
100
0.8
1
1.2

14W17

GWT (m)	2
diameter col. (m)	0.6
c_v	0.0000000139
k col / k clay	500
Creep strength	0.65
Δq_1	0.1
Lenght col. (m)	3.8

h (m)	ρ (t/m^3)	σ'_c (kPa)	M_L (kPa)	σ'_L (kPa)	M´	M_0 (kPa)	M_{COL} (kPa)	τ_{COL} (kPa)
2	1.9	285	4041	421	13.2	11386	7500	100
1	0.84	68	223	106	17.7	2803	7500	100
0.8	0.69	62	760	134	16.9	2798	7500	100
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0

c 1 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
0.8	0.442	20		30		40	
			0.0000		0.0000		0.0000
			0.0041		0.0061		0.0082
			0.0033		0.0052		0.0075
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.007		0.011		0.016

U (%)	time (days)
10	1.23
20	2.60
30	4.15
40	5.94
50	8.06
60	10.66
70	14.01
80	18.72
90	26.79
99	53.57

14W17

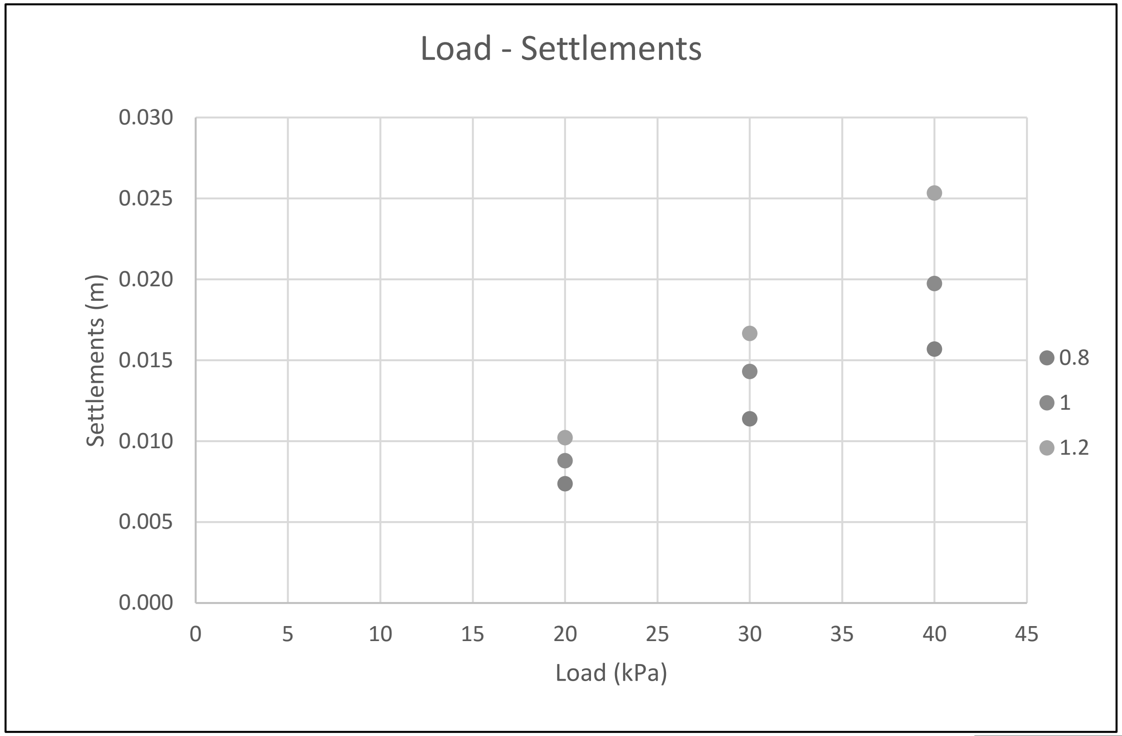
c 2 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1	0.283	20		30		40	
			0.0000		0.0000		0.0000
			0.0048		0.0073		0.0098
			0.0039		0.0070		0.0100
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.009		0.014		0.020

U (%)	time (days)
10	3.62
20	7.66
30	12.25
40	17.54
50	23.80
60	31.46
70	41.34
80	55.26
90	79.06
99	158.13

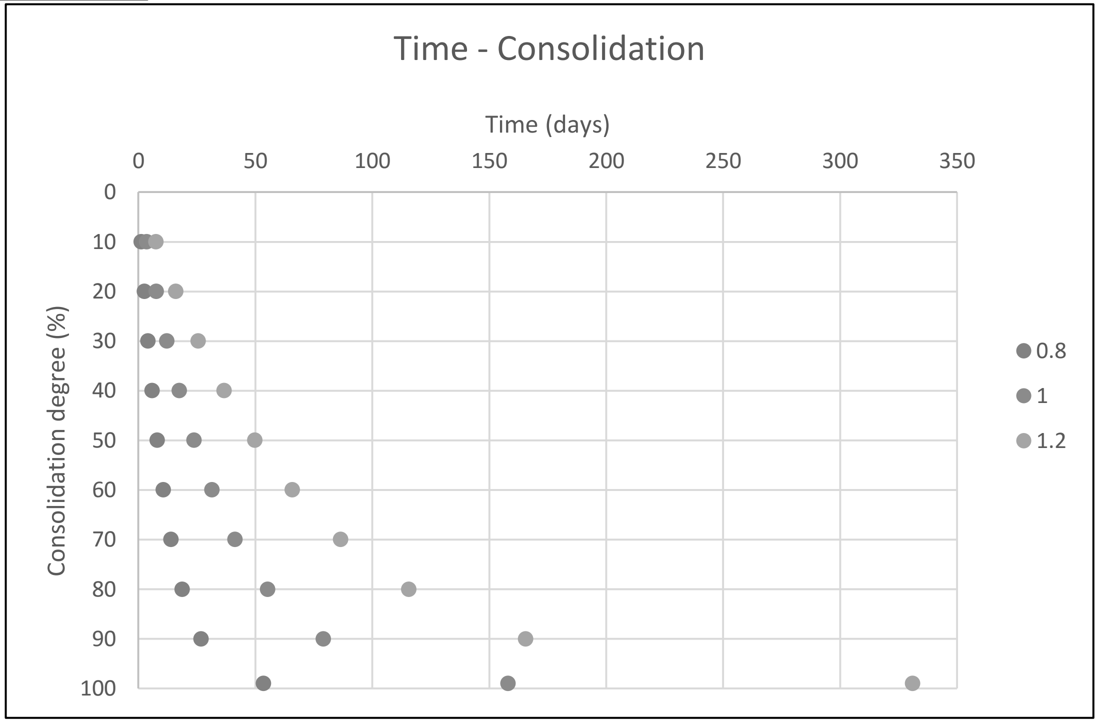
c 3(m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1.2	0.196	20		30		40	
			0.0000		0.0000		0.0000
			0.0054		0.0080		0.0129
			0.0048		0.0086		0.0124
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.010		0.017		0.025

U (%)	time (days)
10	7.57
20	16.04
30	25.64
40	36.72
50	49.83
60	65.87
70	86.55
80	115.70
90	165.53
99	331.06

14W17



Load - Settlements
Settlements (m)
0.030
0.025
0.020
0.015
0.010
0.005
0.000
0
5
10
15
20
25
30
35
40
45
Load (kPa)
0.8
1
1.2



Time - Consolidation
Time (days)
0
50
100
150
200
250
300
350
Consolidation degree (%)
0
10
20
30
40
50
60
70
80
90
100
0.8
1
1.2

30

GWT (m)	1
diameter col. (m)	0.6
c_v	0.0000000156
k col / k clay	500
Creep strength	0.65
Δq_1	0.1
Lenght col. (m)	3.3

h (m)	ρ (t/m^3)	σ'_c (kPa)	M_L (kPa)	σ'_L (kPa)	M´	M_0 (kPa)	M_{COL} (kPa)	τ_{COL} (kPa)
1.2	1.9	0	0	0	0	0	0	0
1.6	0.76	42	339	67	15	2895	7500	100
1.7	0.73	43	418	75	15.6	3062	7500	100
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0

c 1 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
0.8	0.442	20		30		40	
			0.0000		0.0000		0.0000
			0.0065		0.0104		0.0155
			0.0088		0.0139		0.0190
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.015		0.024		0.035

U (%)	time (days)
10	1.00
20	2.13
30	3.40
40	4.87
50	6.61
60	8.73
70	11.48
80	15.34
90	21.95
99	43.90

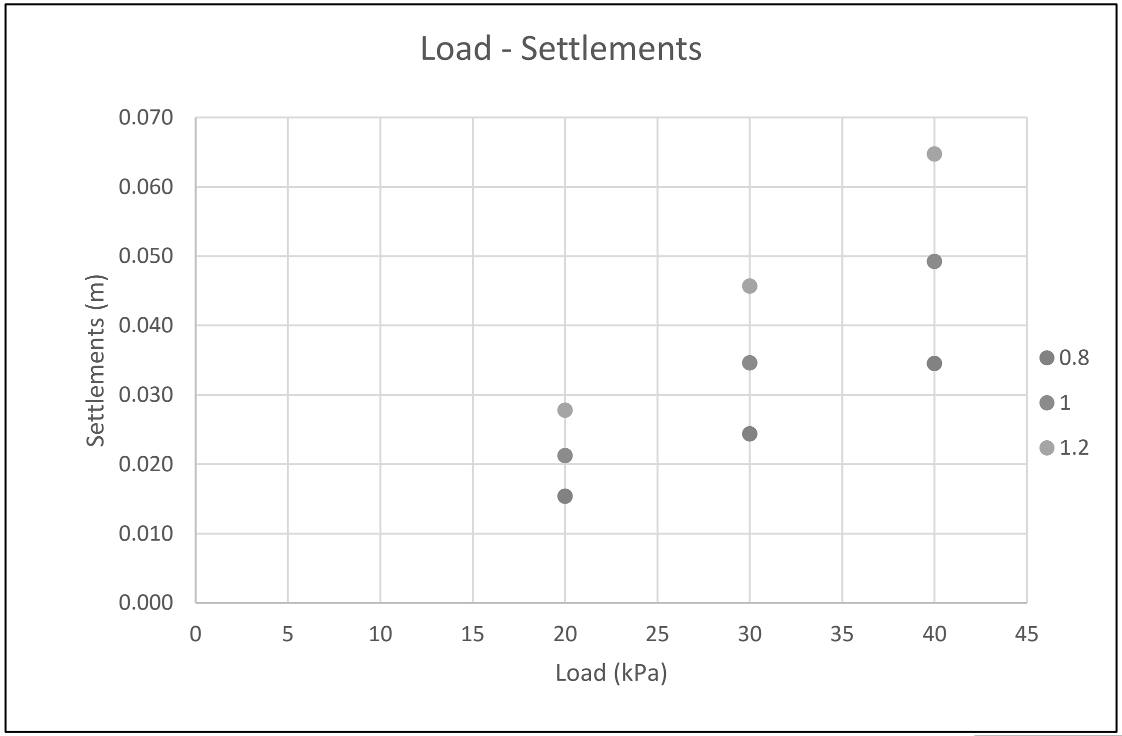
30

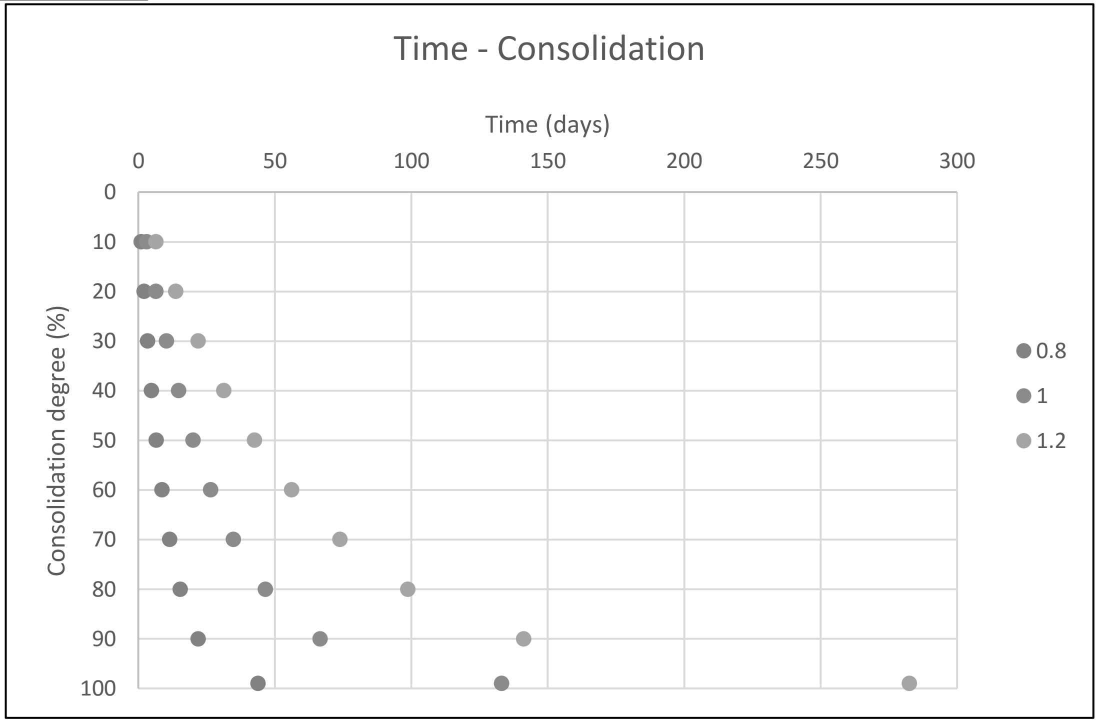
c 2 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1	0.283	20		30		40	
			0.0000		0.0000		0.0000
			0.0078		0.0144		0.0216
			0.0134		0.0202		0.0276
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.021		0.035		0.049

U (%)	time (days)
10	3.05
20	6.45
30	10.32
40	14.77
50	20.05
60	26.50
70	34.82
80	46.55
90	66.60
99	133.20

c 3(m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1.2	0.196	20		30		40	
			0.0000		0.0000		0.0000
			0.0099		0.0187		0.0281
			0.0178		0.0270		0.0366
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.028		0.046		0.065

U (%)	time (days)
10	6.46
20	13.69
30	21.89
40	31.34
50	42.53
60	56.22
70	73.87
80	98.75
90	141.28
99	282.57


Load - Settlements
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
Settlements (m)
0
5
10
15
20
25
30
35
40
45
Load (kPa)
0.8
1
1.2


Time - Consolidation
Time (days)
0
50
100
150
200
250
300
0
10
20
30
40
50
60
70
80
90
100
Consolidation degree (%)
0.8
1
1.2

16B46

GWT (m)	2
diameter col. (m)	0.6
c_v	0.0000000071
k col / k clay	500
Creep strength	0.65
Δq_1	0.1
Lenght col. (m)	6

h (m)	ρ (t/m^3)	σ'_c (kPa)	M_L (kPa)	σ'_L (kPa)	M´	M_0 (kPa)	M_{COL} (kPa)	τ_{COL} (kPa)
2	1.9	0	0	0	0	0	0	0
1.5	0.75	83	802	135	16.4	4973	7500	100
2	0.69	59	446	94	12.4	3783	7500	100
2.5	0.79	93	349	111	14.7	4245	7500	100
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0

c 1 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
0.8	0.442	20		30		40	
			0.0000		0.0000		0.0000
			0.0049		0.0074		0.0098
			0.0104		0.0160		0.0216
			0.0088		0.0143		0.0207
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.024		0.038		0.052

U (%)	time (days)
10	3.59
20	7.60
30	12.15
40	17.41
50	23.62
60	31.23
70	41.03
80	54.85
90	78.47
99	156.94

16B46

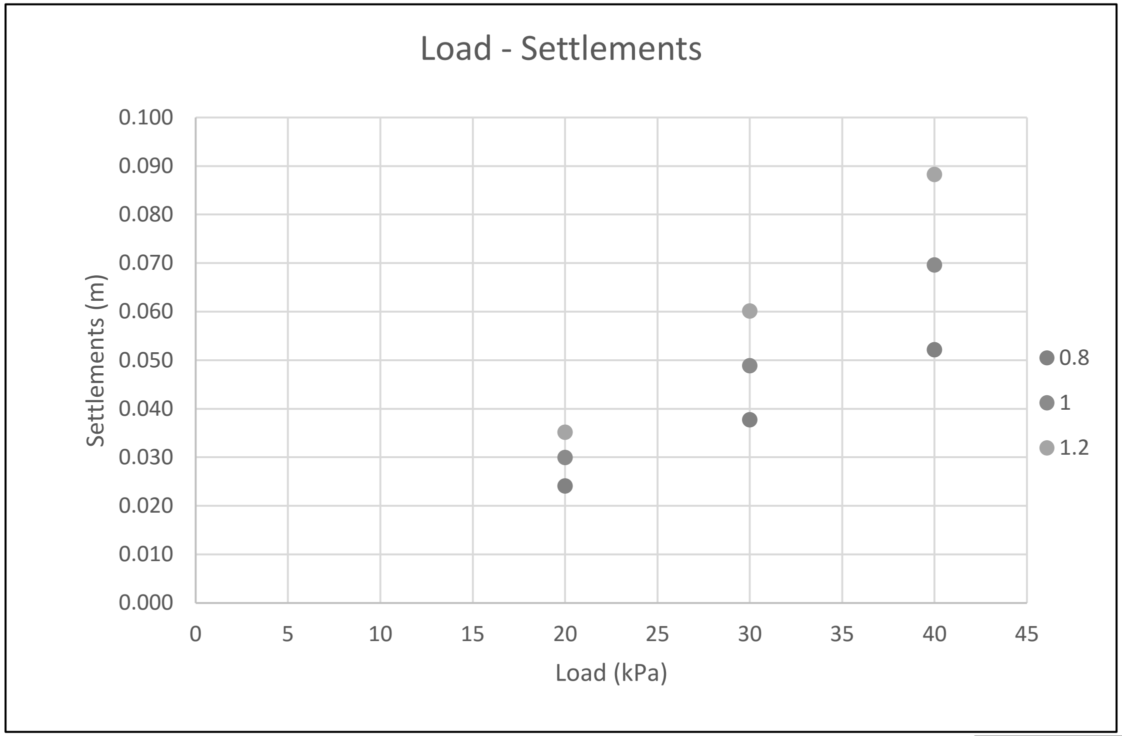
c 2 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1	0.283	20		30		40	
			0.0000		0.0000		0.0000
			0.0053		0.0079		0.0106
			0.0150		0.0231		0.0312
			0.0097		0.0178		0.0278
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.030		0.049		0.070

U (%)	time (days)
10	9.49
20	20.09
30	32.11
40	45.99
50	62.41
60	82.50
70	108.40
80	144.91
90	207.31
99	414.63

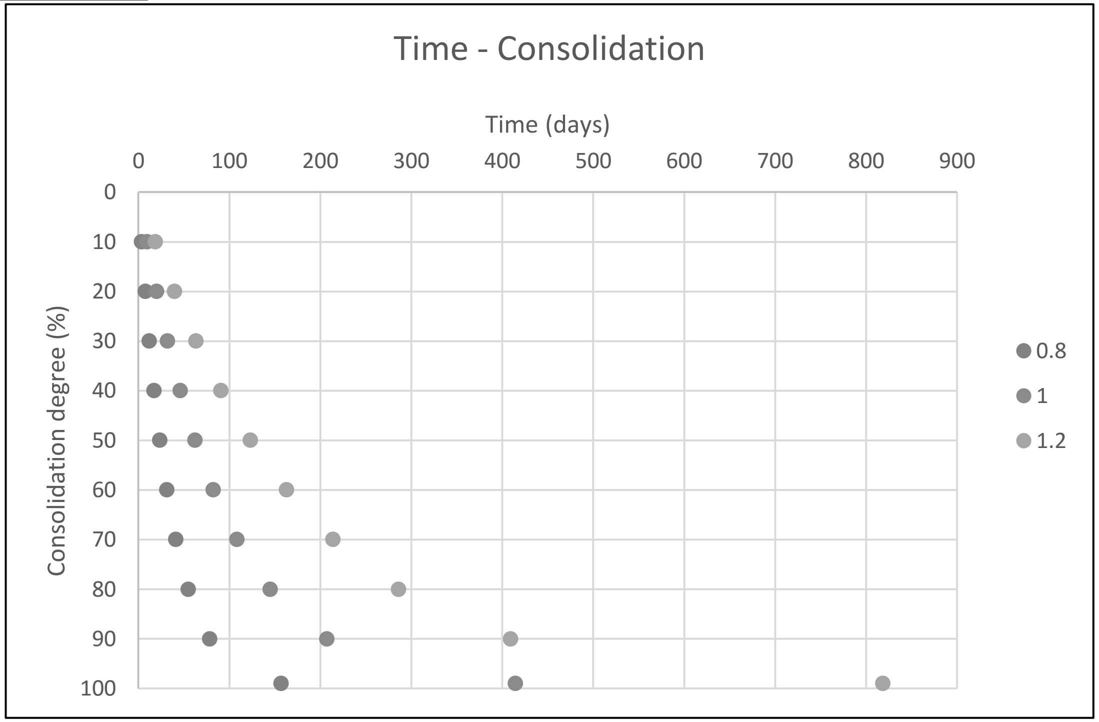
c 3(m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1.2	0.196	20		30		40	
			0.0000		0.0000		0.0000
			0.0055		0.0082		0.0110
			0.0194		0.0306		0.0417
			0.0103		0.0213		0.0356
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.035		0.060		0.088

U (%)	time (days)
10	18.73
20	39.66
30	63.39
40	90.79
50	123.19
60	162.85
70	213.98
80	286.04
90	409.23
99	818.47

16B46



Load - Settlements
Settlements (m)
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
5
10
15
20
25
30
35
40
45
Load (kPa)
0.8
1
1.2



Time - Consolidation
Time (days)
0
100
200
300
400
500
600
700
800
900
Consolidation degree (%)
0
10
20
30
40
50
60
70
80
90
100
0.8
1
1.2

16B40

GWT (m)	1.5
diameter col. (m)	0.6
c_v	0.0000000116
k col / k clay	500
Creep strength	0.65
Δq_1	0.1
Lenght col. (m)	4.3

h (m)	ρ (t/m^3)	σ'_c (kPa)	M_L (kPa)	σ'_L (kPa)	M´	M_0 (kPa)	M_{COL} (kPa)	τ_{COL} (kPa)
1.5	1.9	0	0	0	0	0	0	0
1	0.76	130	3343	314	10.2	5465	7500	100
1.5	0.67	52	313	73	16.2	2563	7500	100
1.8	0.68	58	252	77	16.2	3713	7500	100
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0

c 1 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
0.8	0.442	20		30		40	
			0.0000		0.0000		0.0000
			0.0031		0.0047		0.0063
			0.0063		0.0109		0.0152
			0.0092		0.0143		0.0194
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.019		0.030		0.041

U (%)	time (days)
10	1.60
20	3.40
30	5.43
40	7.77
50	10.55
60	13.94
70	18.32
80	24.49
90	35.04
99	70.08

16B40

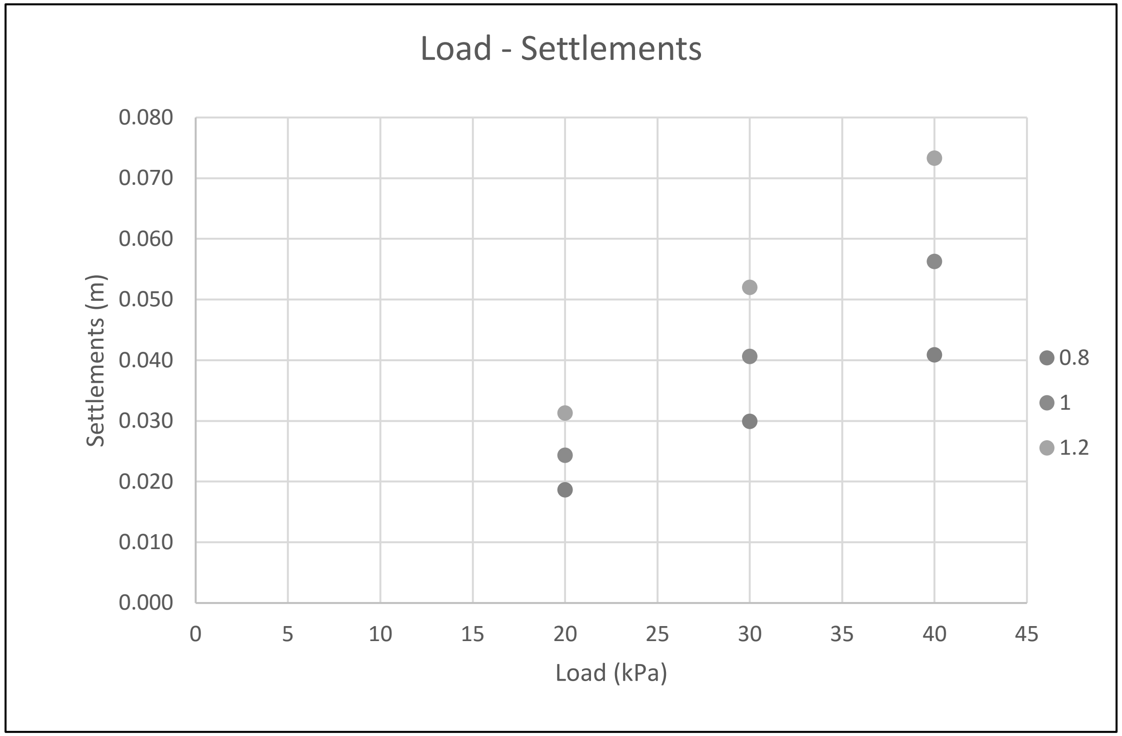
c 2 (m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1	0.283	20		30		40	
			0.0000		0.0000		0.0000
			0.0033		0.0050		0.0066
			0.0084		0.0150		0.0210
			0.0126		0.0206		0.0286
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.024		0.041		0.056

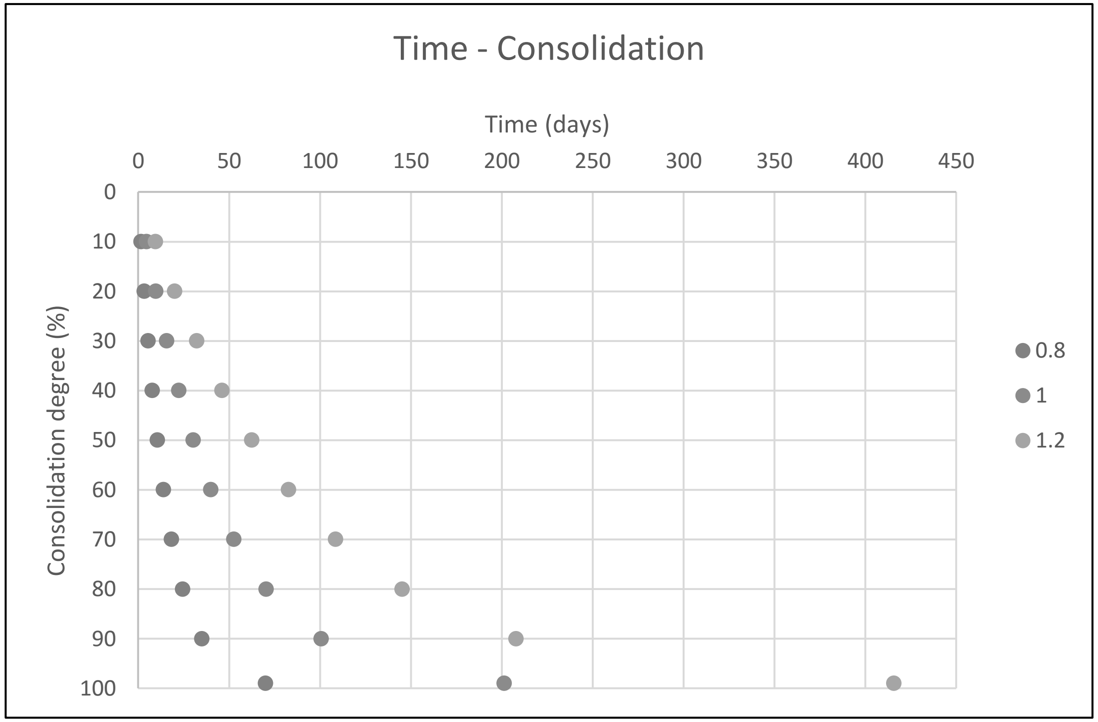
U (%)	time (days)
10	4.61
20	9.75
30	15.59
40	22.33
50	30.30
60	40.05
70	52.62
80	70.35
90	100.64
99	201.29

c 3(m)	a	Load 1 (kPa)	S_M (m)	Load 2 (kPa)	S_M (m)	Load 3 (kPa)	S_M (m)
1.2	0.196	20		30		40	
			0.0000		0.0000		0.0000
			0.0034		0.0051		0.0068
			0.0109		0.0192		0.0282
			0.0169		0.0276		0.0383
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
			0.0000		0.0000		0.0000
		Settlements	0.031		0.052		0.073

U (%)	time (days)
10	9.51
20	20.15
30	32.20
40	46.12
50	62.58
60	82.72
70	108.70
80	145.30
90	207.88
99	415.76

16B40


Load - Settlements
Settlements (m)
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0.000
0
5
10
15
20
25
30
35
40
45
Load (kPa)
0.8
1
1.2


Time - Consolidation
Time (days)
0
50
100
150
200
250
300
350
400
450
Consolidation degree (%)
0
10
20
30
40
50
60
70
80
90
100
0.8
1
1.2

14W44

H	Eff.dens	Sigma'C	ML	Sigma'L	M'	M0	Tau-pel	Mpel
m	t/m3	kPa	kPa	kPa	kPa	kPa	kPa	kPa
1	1.9	30	10000	40	30	12000	100	7500
2	0.64	38	224	52	14.9	2401	100	7500
2	0.66	39	240	83	18	3375	100	7500
2	0.66	70	230	96	20.4	3315	100	7500
2.3	0.69	75	544	106	20.2	3713	100	7500

GWT	1 m
Columns diameter	0.6 m
Columns length	9.3 m
Creep strength	0.65
Cvh	0.0000000073 m^2/s
k columns / k clay	500
Double drainage	

c	0.8m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	11700	100	7500	20	6.88	0.01	0	0
2	25.4	35.4	2401	100	7500	20	14.22	0.01	0	0.01
2	38.4	68.4	338	100	7500	20	18.93	0.01	0	0.01
2	51.6	101.6	3315	100	7500	20	12.81	0.01	0	0.01
2.3	66.14	137.63	2280	100	7500	20	14.45	0.01	0	0.01
Total	Settl.	0.04	m							

c	0.8m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	11084	100	7500	30	10.31	0.01	0	0
2	25.4	35.4	1887	100	7500	30	22.76	0.02	0	0.01
2	38.4	68.4	305	100	7500	30	28.54	0.02	0	0.02
2	51.6	101.6	3146	100	7500	30	19.69	0.02	0	0.01
2.3	66.14	137.63	1606	100	7500	30	23.61	0.02	0	0.02
Total	Settl.	0.062	m							

c	0.8m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	10882	100	7500	40	13.75	0.01	0	0
2	25.4	35.4	1395	100	7500	40	32.4	0.02	0	0.02
2	38.4	68.4	288	100	7500	40	38.14	0.02	0	0.02
2	51.6	101.6	2151	100	7500	40	29.3	0.02	0	0.02
2.3	66.14	137.63	1309	100	7500	40	32.77	0.03	0	0.02
Total	Settl.	0.087	m							

c	1		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	11776	100	7500	20	3.75	0.01	0	0
2	25.4	35.4	2401	100	7500	20	11.09	0.02	0	0.01
2	38.4	68.4	305	100	7500	20	18.13	0.02	0	0.02
2	51.6	101.6	3315	100	7500	20	9.38	0.02	0	0.01
2.3	66.14	137.63	1845	100	7500	20	12.3	0.02	0	0.01
Total	Settl.	0.052	m							

c	1m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	11170	100	7500	30	6.56	0.01	0	0
2	25.4	35.4	1422	100	7500	30	20.27	0.03	0	0.02
2	38.4	68.4	283	100	7500	30	27.36	0.03	0	0.03
2	51.6	101.6	2432	100	7500	30	16.44	0.03	0	0.02
2.3	66.14	137.63	1315	100	7500	30	20.74	0.03	0	0.02
Total	Settl.	0.086	m							

c	1m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	10919	100	7500	40	8.75	0.02	0	0
2	25.4	35.4	1045	100	7500	40	29.57	0.04	0	0.03
2	38.4	68.4	272	100	7500	40	36.64	0.04	0	0.03
2	51.6	101.6	1632	100	7500	40	25.7	0.04	0	0.02
2.3	66.14	137.63	1095	100	7500	40	29.22	0.04	0	0.03
Total	Settl.	0.122	m							

c	1.2		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	11861	100	7500	20	2.5	0.01	0	0
2	25.4	35.4	1981	100	7500	20	9.65	0.03	0	0.01
2	38.4	68.4	288	100	7500	20	17.3	0.03	0	0.02
2	51.6	101.6	3315	100	7500	20	7.19	0.03	0	0.01
2.3	66.14	137.63	1585	100	7500	20	10.74	0.03	0	0.02
Total	Settl.	0.064	m							

c	1.2m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	11193	100	7500	29.31	4.22	0.02	0	0
2	25.4	35.4	1128	100	7500	30	18.54	0.04	0	0.03
2	38.4	68.4	271	100	7500	30	26.13	0.04	0	0.04
2	51.6	101.6	1965	100	7500	30	14.41	0.04	0	0.02
2.3	66.14	137.63	1150	100	7500	30	18.46	0.05	0	0.03
Total	Settl.	0.112	m							

c	1.2m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1	9.5	9.5	10929	100	7500	31.24	5.63	0.02	0	0
2	25.4	35.4	836	100	7500	39.25	27.46	0.05	0	0.04
2	38.4	68.4	263	100	7500	40	34.96	0.05	0	0.05
2	51.6	101.6	1314	100	7500	40	23.32	0.05	0	0.03
2.3	66.14	137.63	971	100	7500	40	26.17	0.06	0	0.04
Total	Settl.	0.161	m							

c	Consoldi. %									
	30	50	60	70	75	80	85	90	95	99
0.8	21	40	53	70	81	94	110	134	174	268
1	49	95	126	166	191	222	261	317	413	634
1.2	90	176	232	305	352	408	481	584	760	1168

14W17

H	Eff.dens	Sigma'C	ML	Sigma'L	M'	M0	Tau-pel	Mpel
m	t/m3	kPa	kPa	kPa	kPa	kPa	kPa	kPa
2	1.9	285	4041	421	13.2	11386	100	7500
1	0.84	68	223	106	17.7	2803	100	7500
0.8	0.69	62	760	134	16.9	2798	100	7500

GWT	2 m
Columns diameter	0.6 m
Columns length	3.8 m
Creep strength	0.65
Cvh	0.0000000139 m^2/s
k columns / k clay	500
Double drainage	

C	0.8 m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	20	6.88	0.01	0	0
1	42.2	47.2	2803	100	7500	20	13.75	0.01	0	0
0.8	49.16	63.16	2798	100	7500	20	13.75	0	0	0
Total	Settl.	0.011	m							

c	0.8 m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	30	10.31	0.02	0	0.01
1	42.2	47.2	2803	100	7500	30	20.39	0.01	0	0.01
0.8	49.16	63.16	2204	100	7500	30	22.03	0.01	0	0.01
Total	Settl.	0.018	m							

c	0.8 m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	40	13.75	0.02	0	0.01
1	42.2	47.2	2803	100	7500	40	27.19	0.01	0	0.01
0.8	49.16	63.16	1752	100	7500	40	30.78	0.01	0	0.01
Total	Settl.	0.024	m							

c	1 m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	20	4.06	0.02	0	0
1	42.2	47.2	2803	100	7500	20	10.31	0.01	0	0
0.8	49.16	63.16	2672	100	7500	20	10.63	0.01	0	0
Total	s„ttning	0.013	m							

c	1 m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	30	6.09	0.03	0	0.01
1	42.2	47.2	2803	100	7500	30	15.47	0.01	0	0.01
0.8	49.16	63.16	1827	100	7500	30	18.52	0.01	0	0.01
Total	Settl.	0.02	m							

c	1m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	40	8.13	0.04	0	0.01
1	42.2	47.2	2541	100	7500	40	21.33	0.02	0	0.01
0.8	49.16	63.16	1518	100	7500	40	26.56	0.02	0	0.01
Total	Settl.	0.028	m							

c	1.2 m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	20	2.81	0.03	0	0
1	42.2	47.2	2803	100	7500	20	7.81	0.01	0	0.01
0.8	49.16	63.16	2290	100	7500	20	8.75	0.01	0	0
Total	Settl.	0.014	m							

c	1.2 m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	30	4.22	0.04	0	0.01
1	42.2	47.2	2803	100	7500	30	11.72	0.02	0	0.01
0.8	49.16	63.16	1633	100	7500	30	15.94	0.02	0	0.01
Total	Settl.	0.022	m							

c	1.2 m	m	Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	11386	100	7500	34.2	5.63	0.05	0	0.01
1	42.2	47.2	2120	100	7500	40	18.67	0.03	0	0.01
0.8	49.16	63.16	1360	100	7500	40	22.97	0.02	0	0.01
Total	Settl.	0.032	m							

	Consolid. %									
c (m)	30	50	60	70	75	80	85	90	95	99
0.8	4	8	11	14	16	19	22	27	35	54
1	12	24	31	41	48	55	65	79	103	158
1.2	26	50	66	86	100	116	136	165	215	331

30

H	Eff.dens	Sigma'C	ML	Sigma'L	M'	M0	Tau-pel	Mpel
m	t/m3	kPa	kPa	kPa	kPa	kPa	kPa	kPa
1.2	1.9	35	10000	50	50	12000	100	7500
1.6	0.76	42	339	67	15	2895	100	7500
1.7	0.73	43	418	75	15.6	3062	100	7500

GWT	1.2m
Columns diameter	0.6m
Columns length	4.5m
Creep strength	0.65
Cvh	0.0000000156 m^2/s
k columns / k clay	500
Double drainage	

c	0.8m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	12000	100	7500	20	6.88	0.01	0	0
1.6	28.88	36.88	2895	100	7500	20	13.44	0.01	0	0.01
1.7	41.17	65.67	711	100	7500	20	17.85	0.01	0	0.01
Total	Settl.	0.018	m							

c	0.8m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	11255	100	7500	30	10.31	0.01	0	0
1.6	28.88	36.88	2067	100	7500	30	22.27	0.01	0	0.01
1.7	41.17	65.67	610	100	7500	30	27.19	0.02	0	0.01
Total	Settl.	0.029	m							

c	0.8m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	10957	100	7500	40	13.75	0.01	0	0.01
1.6	28.88	36.88	1547	100	7500	40	31.72	0.02	0	0.02
1.7	41.17	65.67	563	100	7500	40	36.56	0.02	0	0.02
Total	Settl.	0.039	m							

c	1m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	12000	100	7500	20	3.75	0.01	0	0
1.6	28.88	36.88	2642	100	7500	20	10.47	0.02	0	0.01
1.7	41.17	65.67	621	100	7500	20	16.52	0.02	0	0.01
Total	Settl.	0.024	m							

c	1m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	11338	100	7500	30	6.09	0.02	0	0
1.6	28.88	36.88	1597	100	7500	30	19.45	0.02	0	0.01
1.7	41.17	65.67	552	100	7500	30	25.31	0.02	0	0.02
Total	Settl.	0.038	m							

c	1m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	11001	100	7500	40	8.44	0.02	0	0
1.6	28.88	36.88	1205	100	7500	40	28.44	0.03	0	0.02
1.7	41.17	65.67	517	100	7500	40	34.06	0.03	0	0.03
Total	Settl.	0.053	m							

c	1.2m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	12000	100	7500	20	2.5	0.02	0	0
1.6	28.88	36.88	2232	100	7500	20	9.02	0.02	0	0.01
1.7	41.17	65.67	570	100	7500	20	15.23	0.02	0	0.02
Total	Settl.	0.03	m							

c	1.2m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	11397	100	7500	29.92	4.22	0.02	0	0
1.6	28.88	36.88	1316	100	7500	30	17.46	0.03	0	0.02
1.7	41.17	65.67	518	100	7500	30	23.38	0.03	0	0.03
Total	Settl.	0.049	m							

c	1.2m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.2	11.4	11.4	11025	100	7500	31.83	5.63	0.03	0	0
1.6	28.88	36.88	998	100	7500	39.73	25.9	0.04	0	0.03
1.7	41.17	65.67	492	100	7500	40	31.56	0.05	0	0.04
Total	Settl.	0.069	m							

c (m)	Consolid. %									
	30	50	60	70	75	80	85	90	95	99
0.8	4	8	11	14	16	19	22	27	35	54
1	12	23	31	40	46	54	63	77	100	153
1.2	24	47	63	82	95	110	130	158	205	315

16B46

H	Eff.dens	Sigma'C	ML	Sigma'L	M'	M0	Tau-pel	Mpel
m	t/m3	kPa	kPa	kPa	kPa	kPa	kPa	kPa
2	1.9	70	10000	90	30	12000	100	7500
1.5	0.75	83	802	135	16.4	4973	100	7500
2	0.69	59	446	94	12.4	3783	100	7500
2.5	0.79	93	349	111	14.7	4245	100	7500

GWT	2m
Columns diameter	0.6m
Columns length	8m
Creep strength	0.65
Cvh	0.00000000706 m^2/s
k columns / k clay	500
Double drainage	

c	0.8m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	20	6.56	0.01	0	0
1.5	43.63	51.13	4973	100	7500	20	10.94	0.01	0	0
2	56.15	81.15	930	100	7500	20	17.3	0.01	0	0.01
2.5	72.93	120.43	4245	100	7500	20	11.56	0.02	0	0.01
Total	Settl.	0.028	m							

c	0.8m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	30	9.84	0.02	0	0.01
1.5	43.63	51.13	4973	100	7500	30	16.41	0.01	0	0.01
2	56.15	81.15	760	100	7500	30	26.6	0.02	0	0.02
2.5	72.93	120.43	3665	100	7500	30	18.63	0.02	0	0.01
Total	Settl.	0.043	m							

c	0.8m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	40	13.13	0.02	0	0.01
1.5	43.63	51.13	4973	100	7500	40	21.88	0.02	0	0.01
2	56.15	81.15	678	100	7500	40	35.9	0.02	0	0.02
2.5	72.93	120.43	2526	100	7500	40	28.07	0.03	0	0.02
Total	Settl.	0.061	m							

c	1m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	20	4.06	0.02	0	0
1.5	43.63	51.13	4973	100	7500	20	7.5	0.01	0	0.01
2	56.15	81.15	784	100	7500	20	15.82	0.02	0	0.01
2.5	72.93	120.43	4245	100	7500	20	8.13	0.02	0	0.01
Total	Settl.	0.034	m							

c	1m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	30	6.09	0.03	0	0.01
1.5	43.63	51.13	4973	100	7500	30	11.25	0.02	0	0.01
2	56.15	81.15	663	100	7500	30	24.49	0.03	0	0.02
2.5	72.93	120.43	2933	100	7500	30	15	0.04	0	0.02
Total	Settl.	0.054	m							

c	1m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	40	7.81	0.04	0	0.01
1.5	43.63	51.13	4973	100	7500	40	15	0.03	0	0.01
2	56.15	81.15	607	100	7500	40	33.2	0.04	0	0.03
2.5	72.93	120.43	1973	100	7500	40	23.95	0.05	0	0.03
Total	Settl.	0.078	m							

c	1.2m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	20	2.66	0.03	0	0
1.5	43.63	51.13	4973	100	7500	20	5.31	0.02	0	0.01
2	56.15	81.15	702	100	7500	20	14.45	0.03	0	0.02
2.5	72.93	120.43	4245	100	7500	20	6.09	0.03	0	0.01
Total	Settl.	0.039	m							

c	1.2m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	30	3.98	0.04	0	0.01
1.5	43.63	51.13	4973	100	7500	30	7.97	0.03	0	0.01
2	56.15	81.15	610	100	7500	30	22.5	0.04	0	0.03
2.5	72.93	120.43	2485	100	7500	30	12.77	0.05	0	0.02
Total	Settl.	0.066	m							

c	1.2m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
2	19	19	12000	100	7500	34.2	5.31	0.05	0	0.01
1.5	43.63	51.13	4973	100	7500	40	10.78	0.04	0	0.01
2	56.15	81.15	567	100	7500	40	30.55	0.05	0	0.04
2.5	72.93	120.43	1634	100	7500	40	21.17	0.07	0	0.04
Total	Settl.	0.095	m							

c (m)	Consolid. %									
	30	50	60	70	75	80	85	90	95	99
0.8	17	34	44	58	67	78	92	112	145	224
1	42	83	109	143	165	192	226	274	357	548
1.2	80	156	206	270	311	361	426	517	673	1034

16B40

H	Eff.dens	Sigma'C	ML	Sigma'L	M'	M0	Tau-pel	Mpel
m	t/m3	kPa	kPa	kPa	kPa	kPa	kPa	kPa
1.5	1.9	200	10000	250	30	12000	100	7500
1	0.76	130	3343	314	10.2	5465	100	7500
1.5	0.67	52	313	73	16.2	2563	100	7500
1.8	0.68	58	252	77	16.2	3713	100	7500

GWT	1.5 m
Columns diameter	0.6 m
Columns length	5.8 m
Creep strength	0.65
Cvh	0.0000000116 m^2/s
k columns / k clay	500
Double drainage	

c	0.8m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	20	6.88	0.01	0	0
1	32.3	37.3	5465	100	7500	20	10.63	0.01	0	0
1.5	41.13	58.63	2563	100	7500	20	14.06	0.01	0	0.01
1.8	52.27	86.27	1340	100	7500	20	16.33	0.01	0	0.01
Total	Settl.	0.021	m							

c	0.8m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	30	9.84	0.01	0	0
1	32.3	37.3	5465	100	7500	30	15.47	0.01	0	0
1.5	41.13	58.63	1666	100	7500	30	23.44	0.01	0	0.01

1.8	52.27	86.27	941	100	7500	30	25.93	0.02	0	0.01
Total	Settl.	0.034	m							

c	0.8m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	40	13.13	0.02	0	0.01
1	32.3	37.3	5465	100	7500	40	20.63	0.01	0	0.01
1.5	41.13	58.63	1271	100	7500	40	32.93	0.02	0	0.01
1.8	52.27	86.27	749	100	7500	40	35.51	0.02	0	0.02
Total	Settl.	0.047	m							

c	1m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	20	3.75	0.01	0	0
1	32.3	37.3	5465	100	7500	20	6.88	0.01	0	0
1.5	41.13	58.63	1991	100	7500	20	11.88	0.01	0	0.01
1.8	52.27	86.27	1003	100	7500	20	14.88	0.02	0	0.01
Total	Settl.	0.028	m							

c	1m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	30	6.09	0.02	0	0
1	32.3	37.3	5465	100	7500	30	10.31	0.01	0	0.01
1.5	41.13	58.63	1272	100	7500	30	20.92	0.02	0	0.01
1.8	52.27	86.27	721	100	7500	30	24.11	0.03	0	0.02
Total	Settl.	0.045	m							

c	1m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	40	8.13	0.03	0	0.01
1	32.3	37.3	5465	100	7500	40	14.06	0.02	0	0.01
1.5	41.13	58.63	993	100	7500	40	30	0.03	0	0.02
1.8	52.27	86.27	591	100	7500	40	33.32	0.03	0	0.03
Total	Settl.	0.062	m							

c	1.2m		Q:	20	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	20	2.5	0.02	0	0
1	32.3	37.3	5465	100	7500	20	5	0.01	0	0
1.5	41.13	58.63	1662	100	7500	20	10.55	0.02	0	0.01
1.8	52.27	86.27	828	100	7500	20	13.83	0.02	0	0.02
Total	Settl.	0.033	m							

c	1.2m		Q:	30	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	30	3.98	0.03	0	0
1	32.3	37.3	5465	100	7500	30	7.5	0.02	0	0.01
1.5	41.13	58.63	1044	100	7500	30	19.04	0.03	0	0.02
1.8	52.27	86.27	602	100	7500	30	22.62	0.04	0	0.03
Total	Settl.	0.056	m							

c	1.2m		Q:	40	kPa					
H	Eff.tr	Tot.tr	Mlera	Tpel	Mpel	Q1max/dQ	Q1	S1	S2	SM
m	kPa	kPa	kPa	kPa	kPa	kPa	kPa	m	m	m
1.5	14.25	14.25	12000	100	7500	32.71	5.31	0.03	0	0.01
1	32.3	37.3	5465	100	7500	39.85	10	0.03	0	0.01
1.5	41.13	58.63	823	100	7500	40	27.62	0.04	0	0.03
1.8	52.27	86.27	503	100	7500	40	31.41	0.05	0	0.04
Total	Settl.	0.079	m							

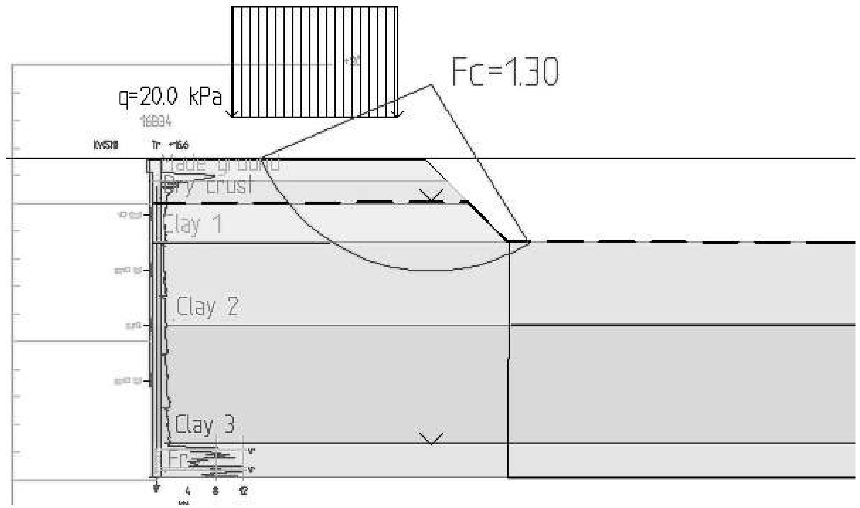
c (m)	Consolid. %									
	30	50	60	70	75	80	85	90	95	99
0.8	7	14	18	24	28	32	38	46	60	92
1	19	37	49	64	74	86	101	123	160	245
1.2	38	73	97	127	146	170	200	243	317	487

Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,50			15,7	1,00	1,00	1,00
Clay 2	16,80			18,9	1,00	1,00	1,00
Clay 3	17,10			24,7	1,00	1,00	1,00
Fr	19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

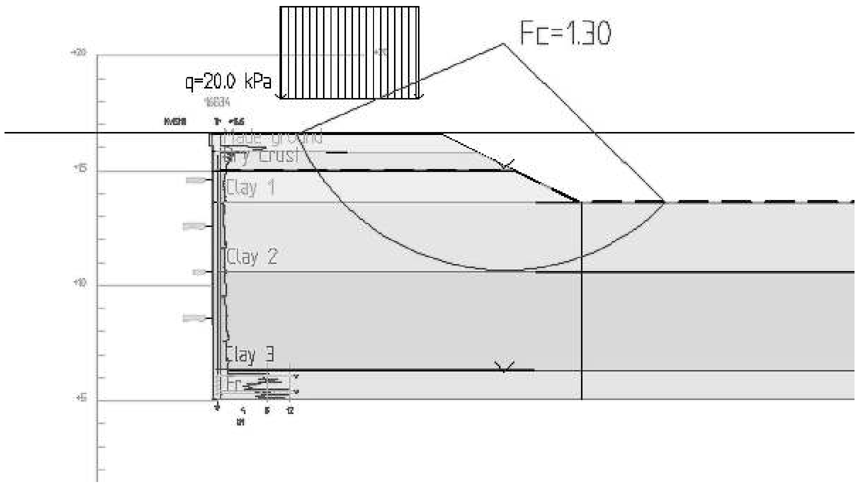


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,50			14,1	1,00	1,00	1,00
Clay 2		16,80			17,3	1,00	1,00	1,00
Clay 3		17,10			23,2	1,00	1,00	1,00
Fr		19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

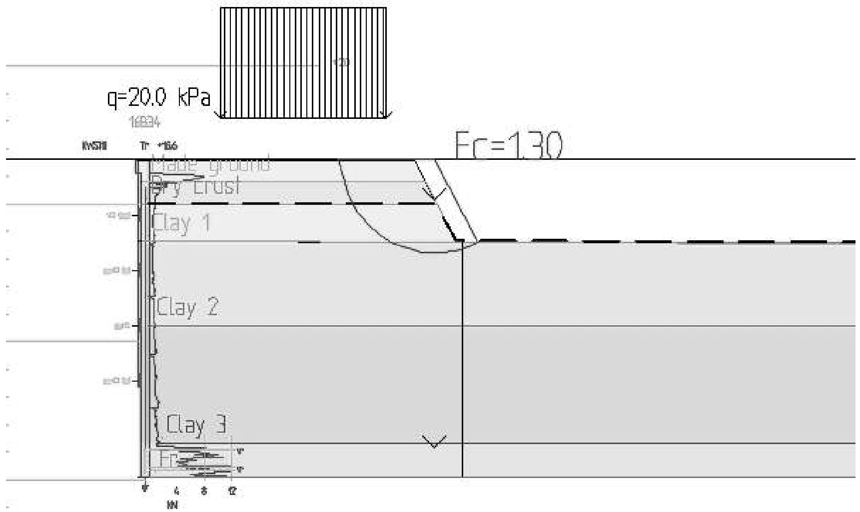


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,50			18,5	1,00	1,00	1,00
Clay 2		16,80			21,5	1,00	1,00	1,00
Clay 3		17,10			27,1	1,00	1,00	1,00
Fr		19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

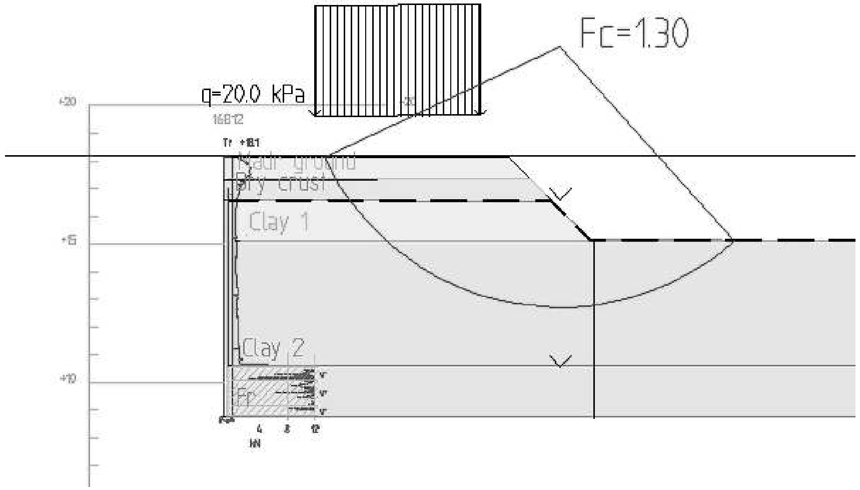


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Madr ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			15,6	1,00	1,00	1,00
Clay 2	16,90			18,1	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

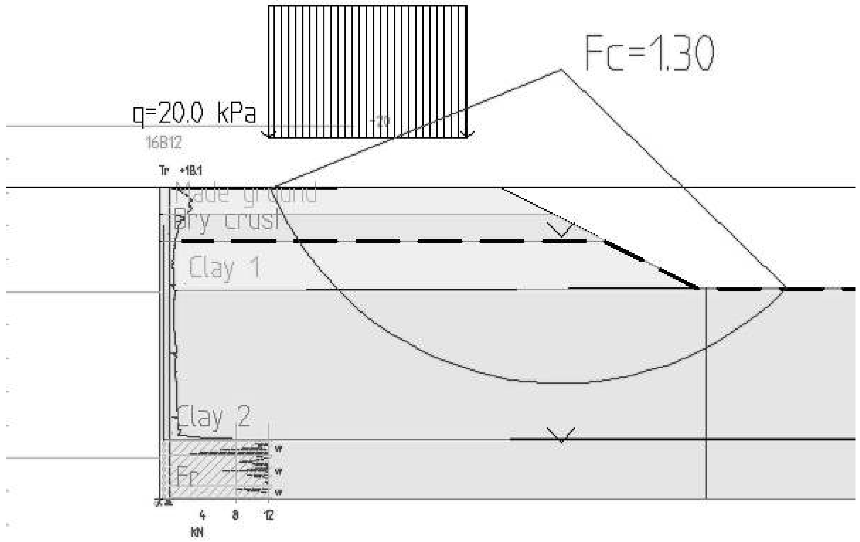


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			15,0	1,00	1,00	1,00
Clay 2	16,90			17,5	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

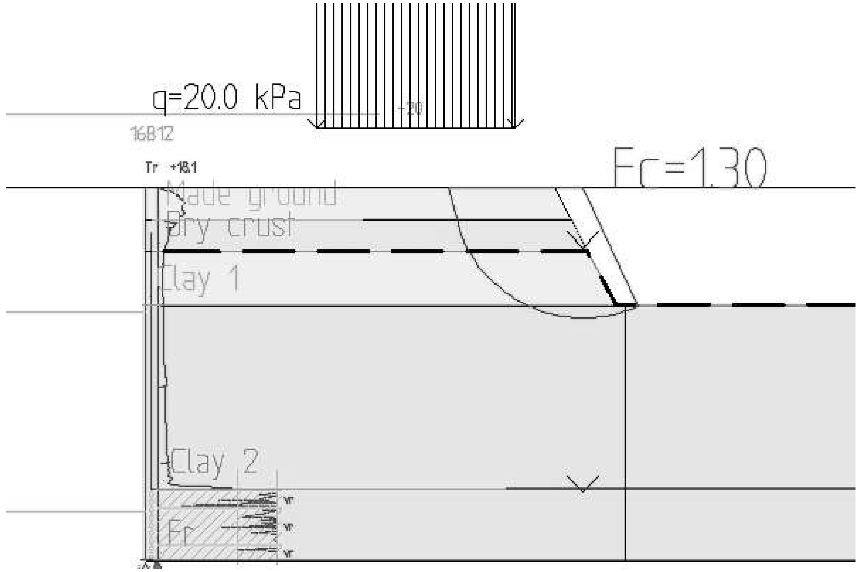


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			19,2	1,00	1,00	1,00
Clay 2	16,90			21,6	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

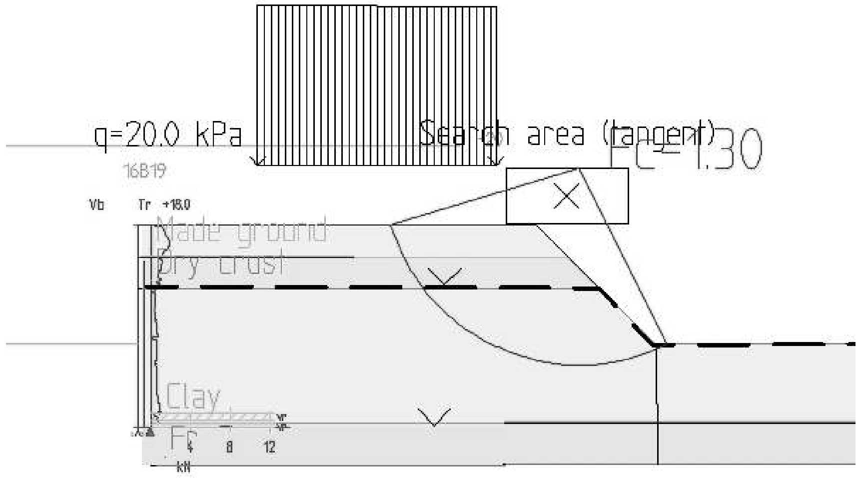


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay	18,20			19,0	1,00	1,00	1,00
Fr	19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

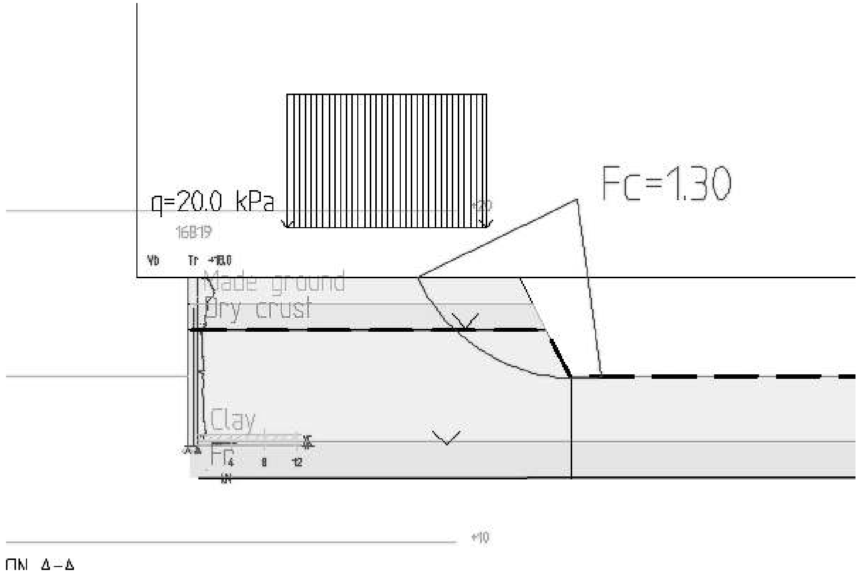


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		18,20			19,2	1,00	1,00	1,00
Fr		19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

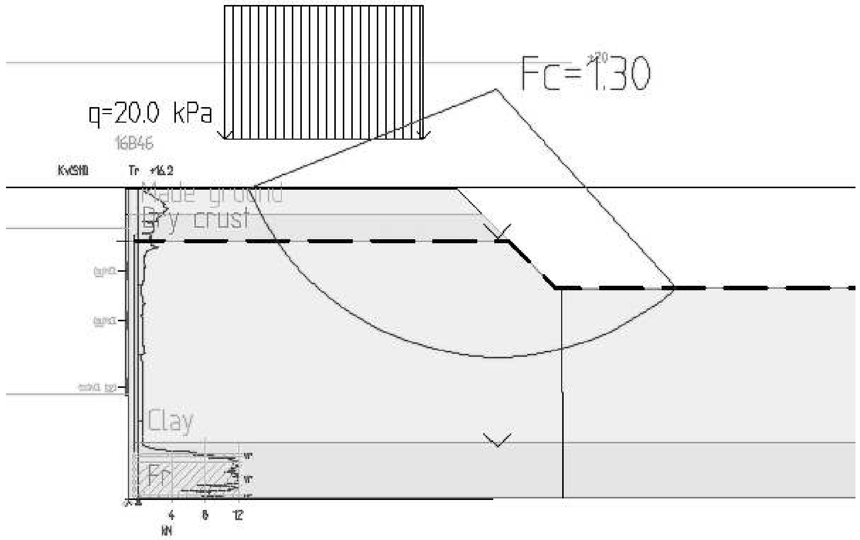


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			18,4	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

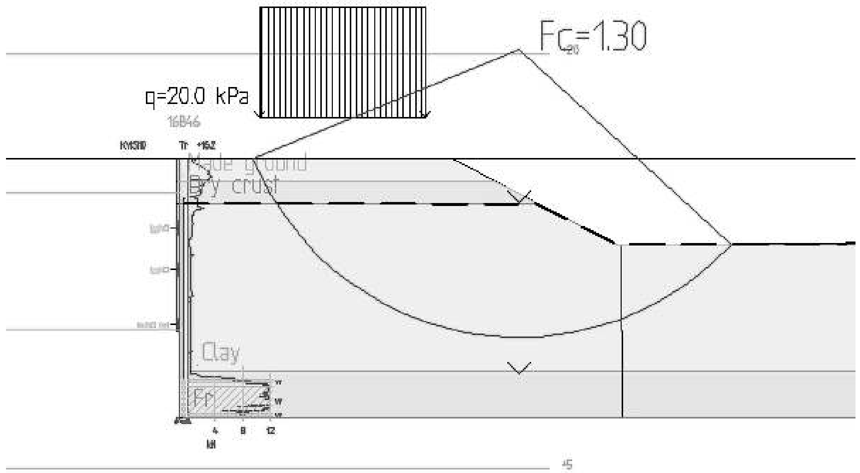


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			17,6	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

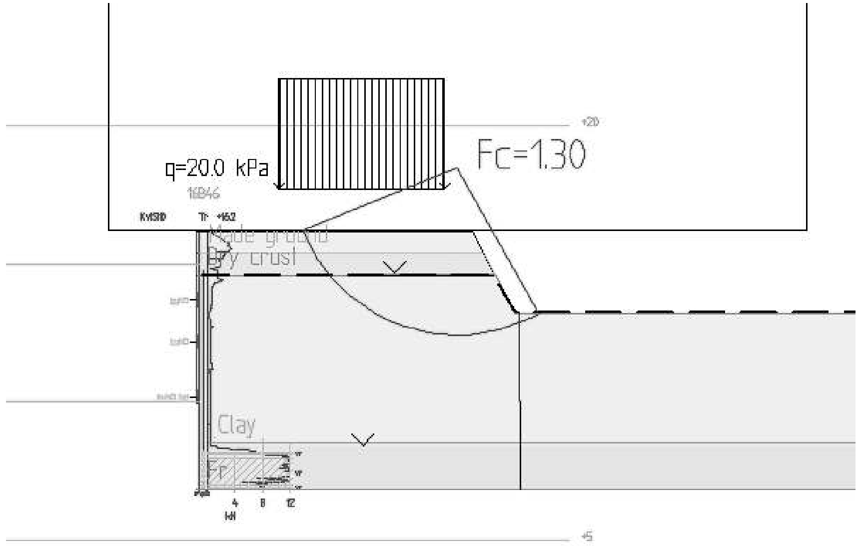


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay	17,50			18,6	1,00	1,00	1,00
Fr	19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

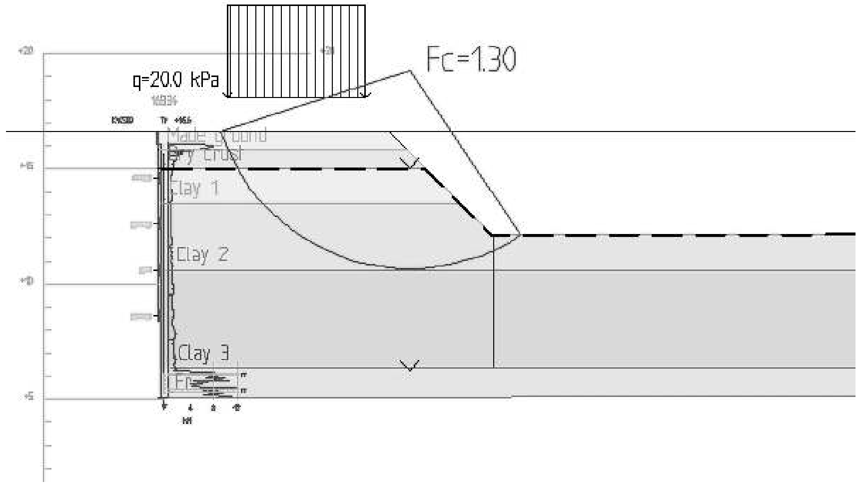


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,50			22,1	1,00	1,00	1,00
Clay 2	16,80			25,0	1,00	1,00	1,00
Clay 3	17,10			30,4	1,00	1,00	1,00
Fr	19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

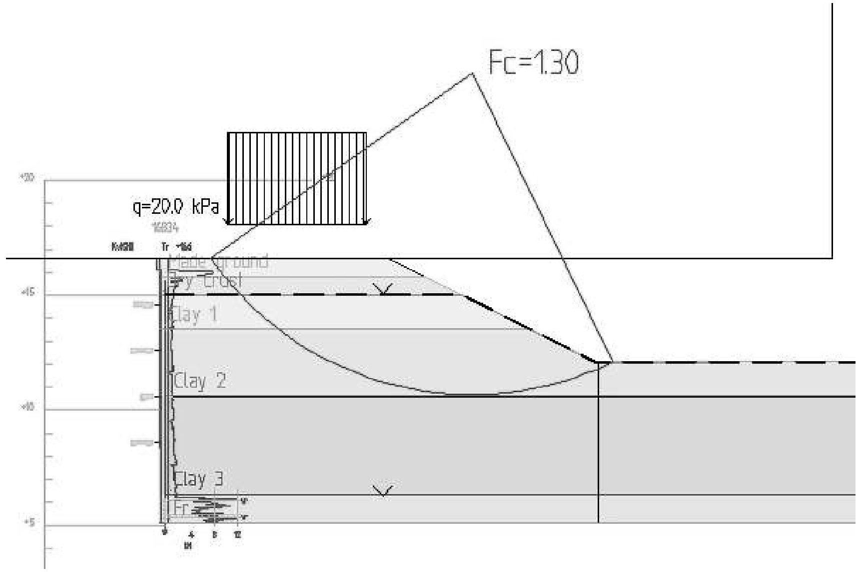


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		18,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,50			17,7	1,00	1,00	1,00
Clay 2		16,80			20,8	1,00	1,00	1,00
Clay 3		17,10			26,5	1,00	1,00	1,00
Fr		19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model

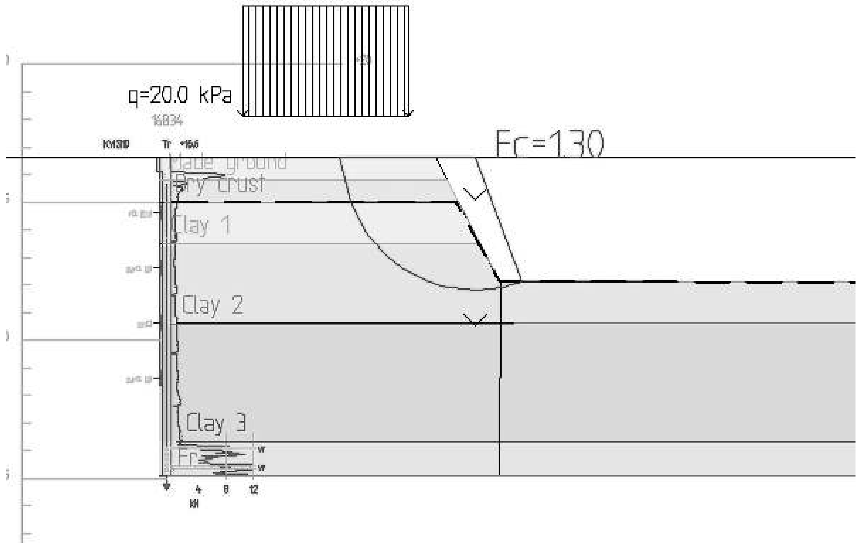


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,50			25,6	1,00	1,00	1,00
Clay 2	16,80			28,3	1,00	1,00	1,00
Clay 3	17,10			33,5	1,00	1,00	1,00
Fr	19,00	36,0	0,0		1,00	1,00	1,00

Graphic Model



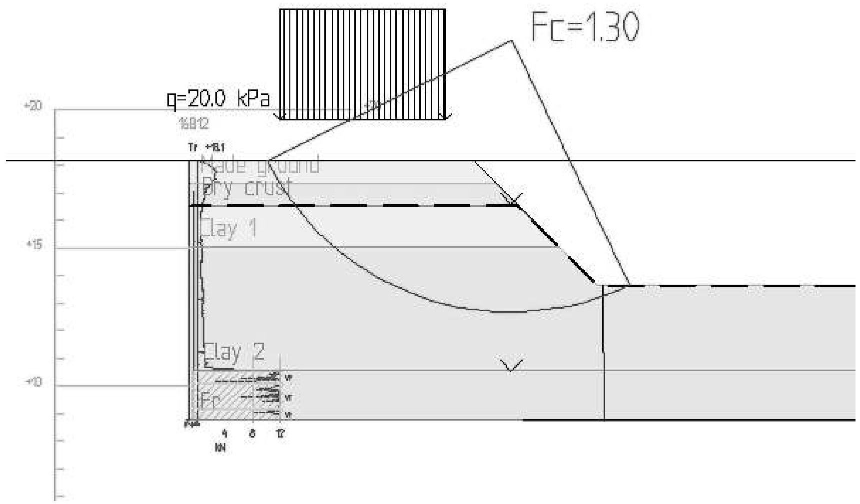
Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			22,3	1,00	1,00	1,00
Clay 2	16,90			24,7	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

VIANOVA GeoSuite
16B12 11 4,5M NEW
2016-06-21 11:42
GS Stability, version: 15.0.1.0

Graphic Model

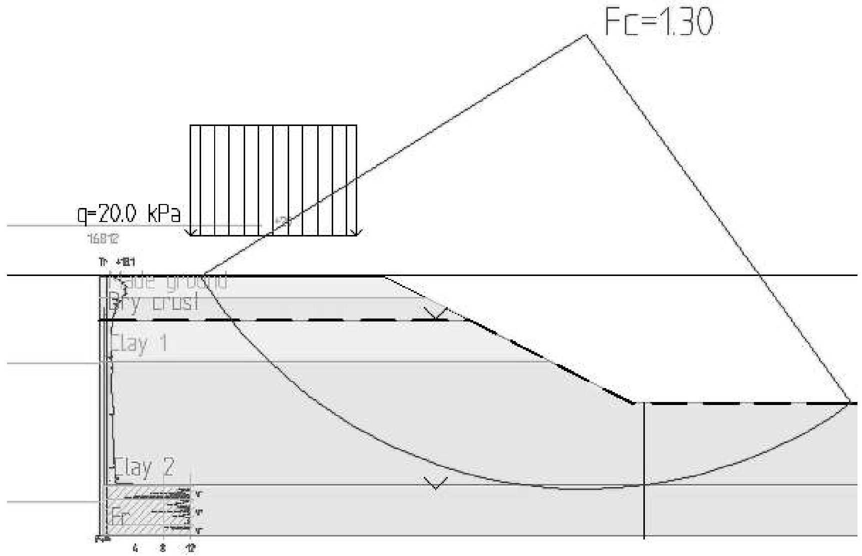


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay 1		17,30			18,2	1,00	1,00	1,00
Clay 2		16,90			20,7	1,00	1,00	1,00
Fr		19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

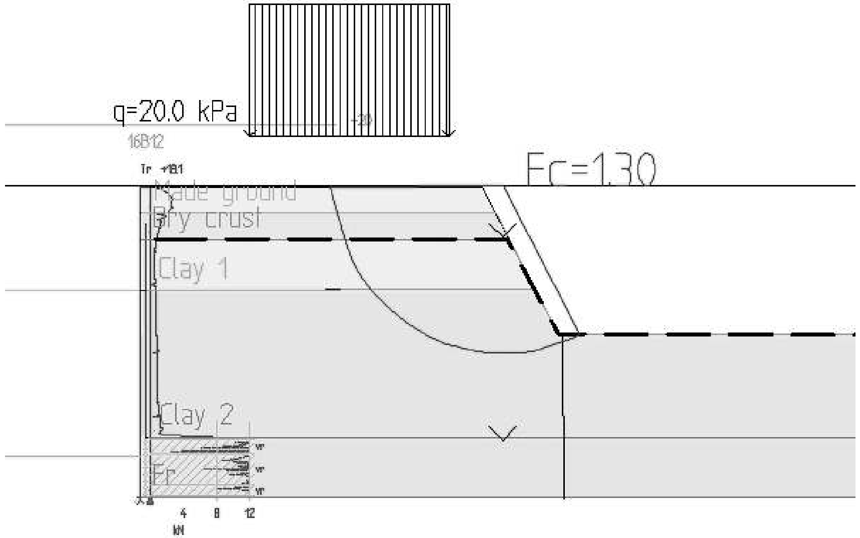


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay 1	17,30			25,8	1,00	1,00	1,00
Clay 2	16,90			28,0	1,00	1,00	1,00
Fr	19,00	30,0	0,0		1,00	1,00	1,00

Graphic Model

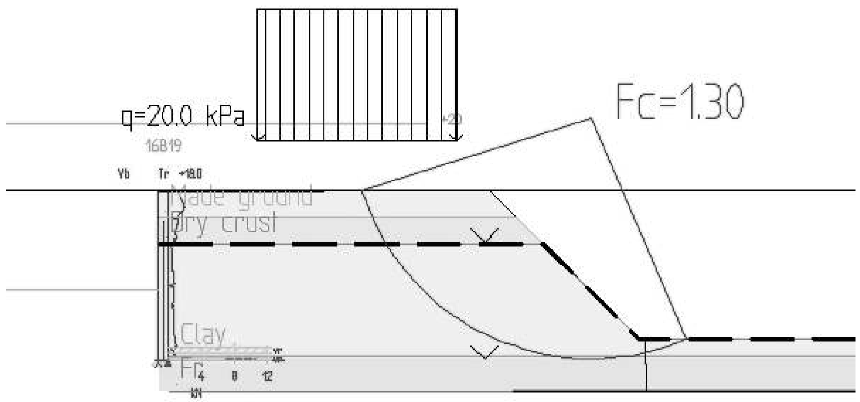


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		18,20			27,3	1,00	1,00	1,00
Fr		19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

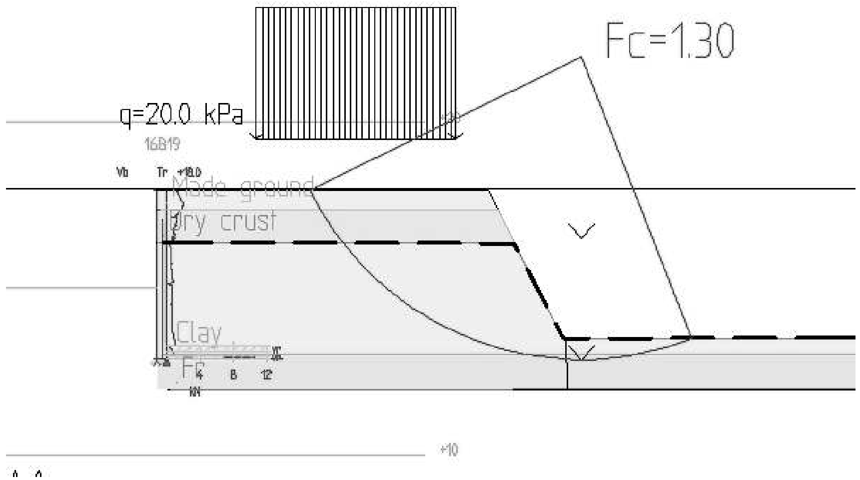


Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay	18,20			31,2	1,00	1,00	1,00
Fr	19,00	33,0	0,0		1,00	1,00	1,00

Graphic Model

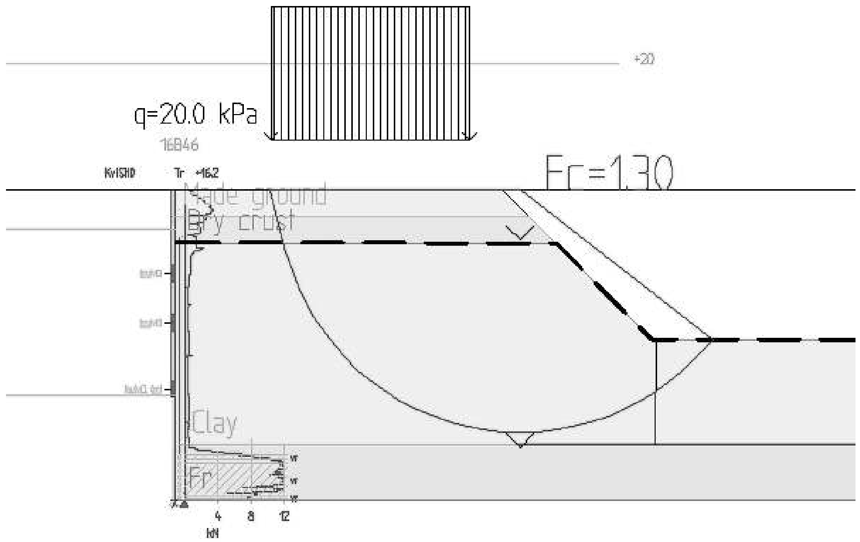


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			23,8	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model

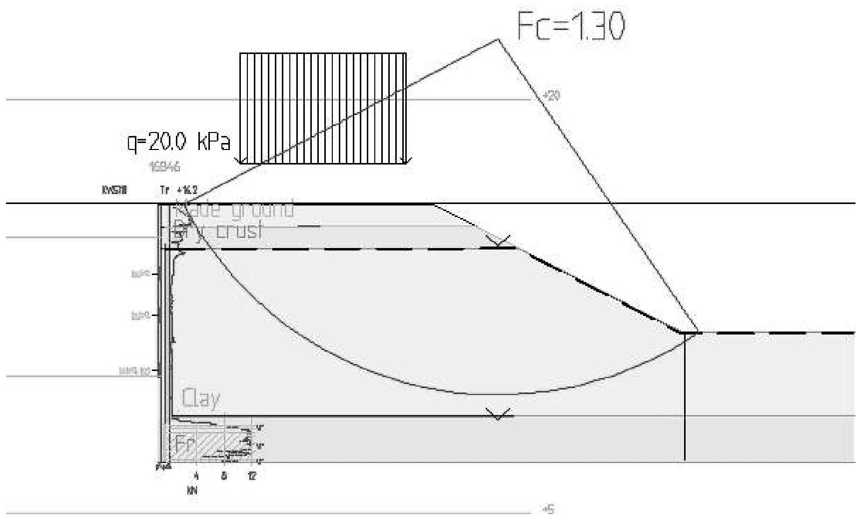


Soil

Materials

Material		ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground		19,00	35,0	5,0		1,00	1,00	1,00
Dry crust		19,00	33,0	5,0		1,00	1,00	1,00
Clay		17,50			21,0	1,00	1,00	1,00
Fr		19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model



Soil

Materials

Material	ρ [kN/m³]	Ø [°]	C' [kPa]	C [kPa]	Aa	Ad	Ap
Made ground	19,00	35,0	5,0		1,00	1,00	1,00
Dry crust	19,00	33,0	5,0		1,00	1,00	1,00
Clay	17,50			27,2	1,00	1,00	1,00
Fr	19,00	34,0	0,0		1,00	1,00	1,00

Graphic Model