CHALMERS

Soil displacement due to installation of lime-cement columns

- An investigation of methods to estimate horizontal displacements

Master of Science Thesis in the Master's Programme Geo and Water Engineering

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Cover:
Installation of lime-cement columns in the project BanaVäg I Väst, photograph taken by Dennis Olsson at the Swedish Transport Administration. The other picture shows a lime-cement column under the ground surface, photograph taken from the Swedish Geotechnical Institute.

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ABSTRACT

There is nowadays no engineering praxis in Sweden to estimate horizontal soil displacement caused by the installation of lime-cement columns. This thesis includes an investigation concerning if methods used for estimation of soil displacement when piling could be adapted for lime-cement columns. The investigated methods were Rehnman's method and the shallow strain path method (SSPM) for both a cylindrical pile and for a planar wall. The FEM-software PLAXIS was utilized for simulating the soil displacement process. In order to evaluate the calculated displacements, field measurement data have been collected. The measured displacements were gained from inclinometers installed near a service road which was recently built in the north of Göteborg in the framework of the contract E41b, as a part of the big infrastructure project BanaVäg I Väst. The inclinometers have measured the soil displacements during the installation of lime-cement columns under the would-be service road. Displacements have been calculated with Rehnman's method and the shallow strain path method in three different sections where inclinometers were installed. The calculated displacements have also been compared with the measured displacements in the same section in order to evaluate the results. The results indicate that the shallow strain path method for a planar wall might be the most suitable method. The method needs however some adjustments. In order to adapt the shallow strain path method for lime-cement columns, an operative value for the wall thickness and an operative value for the column length are utilized. The calculated displacements are in good agreement with the measured displacements when the operative value for the wall thickness is set to 4-6 and the operative value for the column length is set to 0.8. An analysis was also made using the FEM-software PLAXIS to estimate the soil displacements. The analyzed model was made for one of the investigated sections whereby the rows of lime-cement columns are idealized as a wall in the same way as in the shallow strain path method for a planar wall. The PLAXIS model results in a displacement very similar to the displacement calculated with the adapted shallow strain path method when the same operative values for the wall thickness are used in the model.

Key words: lime-cement column, LC-column, soil displacement, shallow strain path method, planar wall, SSPM, PLAXIS, inclinometer, Rehnman's method, BanaVäg I Väst,

Jordundanträngning vid installation av kalkcementpelare
– En undersökning av metoder för att uppskatta horisontella förflyttningar
Examensarbete inom Geo and Water Engineering
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SAMMANFATTNING

Det finns idag ingen teknisk praxis i Sverige för att uppskatta horisontell jordundanträngning som sker vid installation av kalkcementpelare. Detta examensarbete inkluderar en undersökning rörande om metoder som används vid uppskattning av jordundanträngning vid pålning kan anpassas för kalkcementpelare. De metoder som undersökts är Rehnmans metod och ”shallow strain path”-metoden (SSPM) för både en cylindrisk påle och en plan vägg. FEM-programvaran PLAXIS användes för att simulera jordundanträngningsprocessen. För att kunna utvärdera de beräknade förskjutningarna har fältmätningar samlats in. De uppmätta förskjutningarna är hämtade från inklinometrar som installerades innan konstruktionen av en serviceväg som nyligen byggdes norr om Göteborg, i samband med etapp E41b av det stora infrastrukturprojektet BanaVäg i Väst. Inklinometrarna har registrerat jordundanträngningen som skedde under installationen av kalkcementpelare under den blivande servicevägen. Jordundanträngningen har beräknats med Rehnmans metod och SSPM i tre olika sektioner där inklinometrar var installerade. Den beräknade jordundanträngningen har jämförts med den uppmätta från samma sektion för att kunna utvärdera resultaten. Resultaten indikerar att SSPM för en plan vägg är den mest lämpliga metoden, den behöver dock justeras. För att anpassa SSPM för kalkcementpelare används operativa värden för väggtjockleken och pelarlängden. De beräknade förskjutningarna stämmer väl överens med de uppmätta förskjutningarna då den operativa korrektionsfaktorn för väggtjockleken är 4-6 och den operativa korrektionsfaktorn för pelarlängden är 0.8. En numerisk analys gjordes även med FEM-programvaran PLAXIS för att uppskatta och simulera jordförskjutningarna. Modellen i PLAXIS gjordes för en av de undersökta sektionerna där raderna av kalkcementpelarna är idealiserade som en vägg på samma sätt som i SSPM för en plan vägg. PLAXIS-modellen resulterade i en förskjutning likvärd den beräknade förskjutningen med SSPM då samma operativa korrektionsfaktor användes i modellen.

Nyckelord: kalkcementpelare, KC-pelare, markförskjutning, jordundanträngning, inklinometer, Rehnmans method, SSPM, PLAXIS, BanaVäg I Väst, massundanträngning

Contents

Preface

In this master's thesis, horizontal displacements that occur due to installation of Lime-cement columns have been investigated. The thesis aims to investigate methods to estimate horizontal soil displacement due to installation of lime-cement columns. This is made by adapting existing methods for estimation of horizontal displacement when piling.

The thesis has been carried out at the division of GeoEngineering at Chalmers University of Technology during the spring term of 2012. It is the final part of the education in the master's program Geo and Water Engineering and Professor Claes Alén has been the examiner.

This thesis was initiated by our supervisor Kien Du-Thinh, Ramböll Sverige AB, in cooperation with the Swedish Transport Administration. The work has also been carried out at Ramböll, the division of Geotechnics in Göteborg.

First of all we would like to thank our supervisor Kien Du-Thinh who made the work with this thesis possible. We would also like to thank for his great support, guidance and expertise throughout the entire working process.

We would also like to thank Dennis Olsson at the Swedish Transport Administration and Peter Jansson at ÅF AB, who helped us collect all the background information in the investigated area included in the project BanaVäg I Väst. They have also provided us with great support and answered all of our questions, which have been much appreciated.

Göteborg, June 2012

Per Klasson and Tobias Kristensson

Notations

Roman upper case letters

E_{oed}	[kN/m^2]	Oedometer modulus
E_u	[kN/m^2]	Undrained Young's modulus
$E_{u,inc}$	[kN/m^2]	Undrained incremental modulus
G	[kN/m^2]	Shear modulus
K_0	[-]	Increase in lateral stress coefficient
L	[m]	Length of pile, LC-column or LC-column wall
$M_{\max}$	[kNm]	Maximal moment force
$M_{\min}$	[kNm]	Maximal moment force in remoulded state
OCR	[-]	Over consolidation ratio
R	[m]	Radius of pile
S_t	[-]	Sensitivity
V_{binder}	[m^3]	Volume of added binder
V_{column}	[m^3]	Volume of LC-column
V_{piles}	[m^3]	Volume of driven piles
V_{soil}	[m^3]	Volume of soil in LC-column area
V_{clay}	[m^3]	Volume of soil removed with pre-boring

Roman lower case letters

a	[-]	Coverage ratio
b	[m]	Width of piling area
d	[m]	Pile depth from surface
d	[m]	Column diameter
e	[m]	Distance from outer border of piling area
l	[m]	Length of piling area
q_T	[kN/m^2]	Point resistance

r	[m]	Radial distance
s	[m]	CC-distance
w	[%]	Natural water content
w	[m]	Half wall thickness
w_L	[%]	Liquid limit
x	[m]	Heave within piling area
x	[m]	Distance from wall to point of interest
z	[m]	Depth from ground surface

Greek lower case letters

α	[-]	Relative load from building
β	[-]	Relative load from building
γ	[-]	Relative load from building
γ_{unsat}	[kN/m^3]	Unsaturated unit weight
δ	[-]	Relative load from building
δ_h	[m]	Adjusted horizontal displacement
$\delta_{h,surface}$	[m]	Horizontal displacement on ground surface
δ_{rSS}	[m]	Horizontal displacement, SSPM
δ_v	[m]	Heave at ground surface
δ_x	[m]	Adjusted horizontal displacement
δ_{zSS}	[m]	Vertical displacement, SSPM
η	[-]	Heave factor
μ	[-]	Correction factor
μ_L	[-]	Operative correction factor for LC-column length
μ_w	[-]	Operative correction factor for the wall thickness
ν_u	[-]	Undrained Poisson's ratio
ρ	[kg/m^3]	Density

σ_{vo}	[kN/m^2]	Overburden pressure
σ'_c	[kN/m^2]	Pre-consolidation pressure
σ'_0	[kN/m^2]	Effective in-situ stress
τ_{fu}	[kN/m^2]	Undrained shear strength
τ_k	[kN/m^2]	Undrained shear strength, fall cone test
τ_R	[kN/m^2]	Undrained shear strength, fall cone test, remoulded
τ_v	[kN/m^2]	Undrained shear strength, vane test

1 Introduction

Stabilization of soil with lime-cement columns is a method used more often nowadays in large infrastructure projects in Sweden. One of many large infrastructure projects where lime-cement columns are used started in 2004. The project was named BanaVäg I Väst and includes upgrading to a four-lane highway (E45) and a double tracked railway (Norge/vänerbanan) between Göteborg and Trollhättan. The project is planned to be finished by the end of 2012 and it will double the train transport capacity, increase the road capacity and at the same time shorten the travel time.

The highway and the railway will be located in the valley of Göta älv, which is a region known for its difficult geotechnical conditions. The road and the railway run very close to the river Göta älv and close to small communities between Göteborg and Trollhättan. This location causes some difficulties, the short distance to Göta älv may lead to problems regarding stability and the proximity to facilities may raise problems regarding vibrations, mainly from the railway. To increase the stability and to decrease settlements and vibrations from the railway there have been some extensive work done to stabilize the soil with lime-cement columns. Lime-cement columns have been made both underneath and beside the railway, where a service road was constructed.

In this thesis an area included in the project BanaVäg I Väst is investigated. The area is situated south of the town Surte and just north of the bridge Angeredsbron.

1.1 Background

The environmental impacts from lime-cement columns, with special focus on soil displacement, could lead to severe problems. When installing lime-cement columns there are an addition of material in the present soil due to the added mixture of lime and cement, which is distributed into the ground by a mixing tool with high air pressure. There is also a chemical reaction between the lime-cement binder and the water in the soil which increases the volume of the added material. This leads to soil displacements and possible consequences could be that adjacent constructions may be displaced or damaged. If displacements occur in a slope there could also be an increased risk for a landslide to occur.

The soil displacements are hard to predict and could have major consequences if not handled correctly. Unlike for piling, there is no specific praxis used in Sweden today for prediction of soil displacements due to installation of lime-cement columns. Displacements are though still a known problem and it is common to continuously measure soil displacements during the installation phase. Different measuring devices, such as an inclinometer, are installed and in this way it is possible to monitor the displacements during construction.

1.2 Purpose and Aim

Up to now only the lateral soil displacements caused by driving displacement piles into clayey soils have been assessed, more or less successfully in Sweden, using the empirical relations by Rehnman or analytical with the shallow strain path method (SSPM) by Sagaseta and Whittle. In this thesis efforts have been made;

1. To investigate the possibility of applying these above mentioned methods to predict horizontal soil displacements when installing lime-cement columns in soft clay.
2. To select the most promising method and make a temptative change to render it suitable to cope with the topical problem.
3. To validate the suggested adaptation technique by applying it to another documented field situation, whereby the calculated results are compared with the data from field measurements
4. To find an appropriate modelling technique for making a realistic numerical simulation of soil displacements subsequent to the installation of lime-cement columns in soft clay

1.3 Method

A literature study was made in order to learn about geotechnical investigations, lime-cement columns and methods used for calculating displacement when driving piles.

Field- and laboratory data from section 465/300, in the contract E41b included in the project BanaVäg I Väst, were collected and evaluated in order to establish the geotechnical conditions. Further, inclinometer data and lime-cement column properties were collected and compiled. The lime-cement column properties were used as input data in the investigated methods in order to calculate displacements. Since the methods apply for piles, the methods were adapted for lime-cement columns. Further, the calculated displacements were compared to the measured displacements evaluated from the inclinometer data in the investigated section. The method adapted to apply for lime-cement columns is finally evaluated in two other sections, 465/535 and 465/750.

A simplified model of section 465/300 was made in the software PLAXIS, a finite element package, to estimate the soil displacement. The input data were obtained from the evaluated field- and laboratory investigations.

1.4 Scope

The investigation in this thesis only considers horizontal displacements. Further, it is only the displacement due to the lime-cement columns installed under the service road in the contract E41b that is considered. The study is focused on data collected from section 465/300 where a detailed investigation was made, the other sections handled in this thesis, the sections 465/535 and 465/750, are handled in a less detailed way. Three methods used for calculating displacement are included in the investigation; an empirical-, an analytical- and a numerical method.

2 Soil Stabilization with Lime-Cement Columns

Lime-cement column (LC-column) installation has become a popular method in Sweden to improve soft soils in construction projects. This section provides a basic understanding in the procedure of installing LC-columns. It also describes applications, its impacts and the most typical properties for LC-columns.

2.1 Installation procedure

Two different procedures are used to install LC-columns: the dry mixing method and the wet mixing method. The most significant difference between the methods is that in the dry method binder is added as a dry powder ready to react with water in the soil. In the wet method the binder is instead added as a slurry binder already containing water. The soil must have a water content of at least 20 % for the dry mixing method to be applicable. The dry mixing method is the most common method used in Sweden and is further described in this section (Larsson, 2006).

When installing LC-columns the first step is to place the installation machinery in the right position and then place the mixing tool on the right spot with correct angle. The mixing tool is normally placed vertically but can vary with an angle between 0 - 70 degrees in relation to the vertical line. There are different mixing tools used today where "Pinnborr" and "SGF standard tool" are the two most commonly used in Sweden, see Figure 2.1.

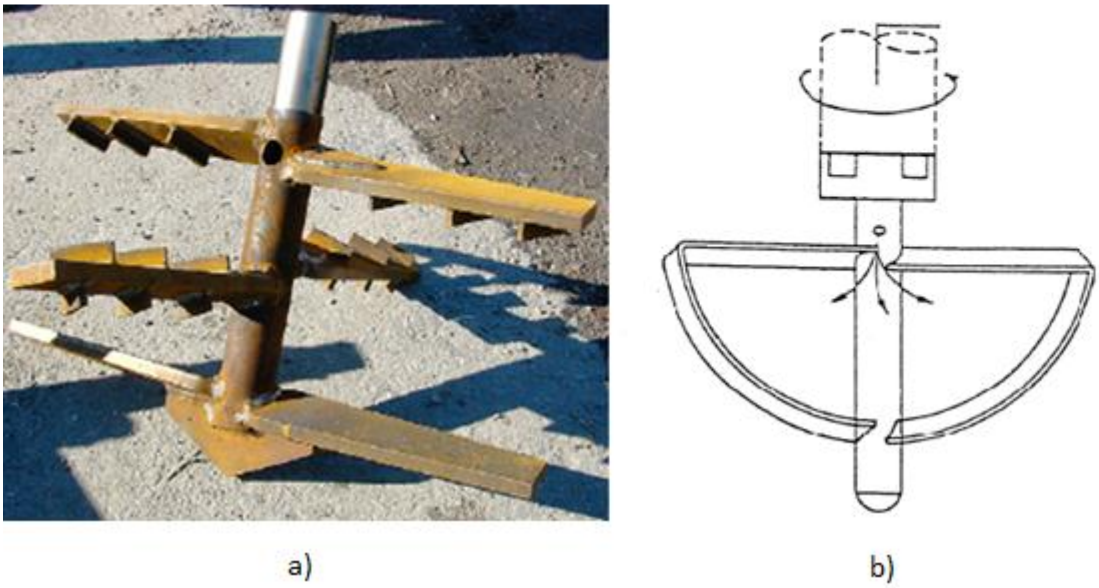

Figure 2.1 Mixing tools commonly used in Sweden. a) Pinnborr (LCM, 2002). *b) SGF standard tool* (SGF, 2000).

Initially the mixing tool starts to rotate as it is driven down into the soil, see Figure 2.2. When the desired depth is reached, the mixing tool starts to rotate in the opposite direction and starts going upwards while the lime-cement binder is injected through the mixing tool. In this way the binder will be mixed with the soil, which creates a soil with higher shear strength and less compressibility. It is recommended to wait until the installed columns have gained 90 % of their designed shear strength before the final loads are applied, normally 3-4 weeks.

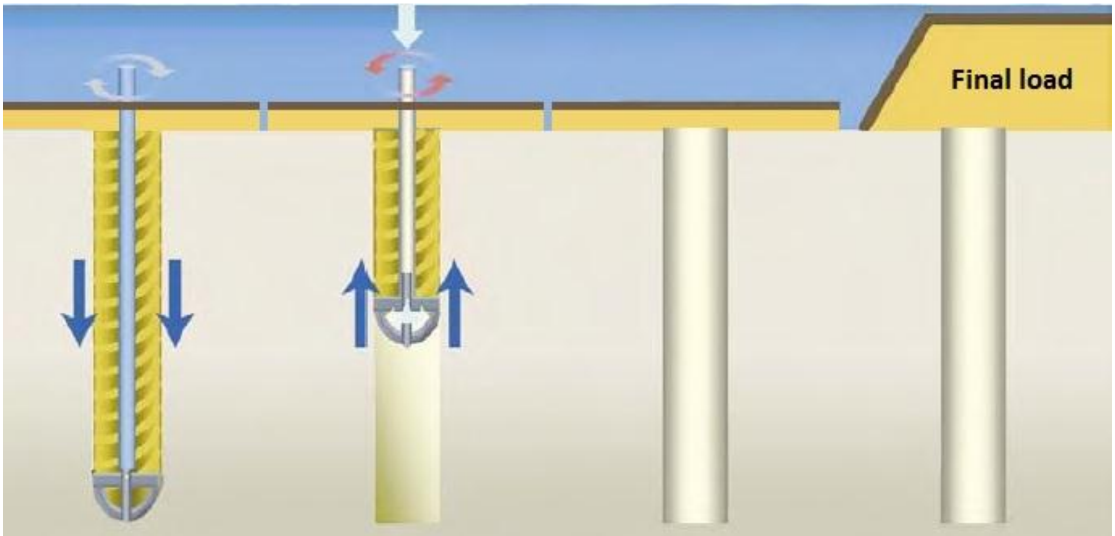

Figure 2.2 Production procedure of LC-columns (Keller ground engineering, 2008, modified).

In order to optimize the mixing it is important that the mixing tool has a correct rotation speed, uplifting speed and injection pressure. The injection pressure for the binder is normally 400-600 kPa down to approximately 15 m. Due to the high pressure, the injection of the binder has to stop 0.5-1.0 m below the ground surface when going upwards in order to avoid the binder to spread above the ground surface. The amount of lime and cement in columns are normally distributed 50/50 when using the dry mixing method but varies depending on the soil properties and the desired strength. The maximum depth used for a LC-column in Sweden is 25 m and the diameter varies between 0.4-1.0 m with 0.6 and 0.8 m as the most commonly used. The minimum target shear strength for the strengthened soil is 100 kPa since this is the highest shear strength allowed in the design. If the soil conditions are favourable and the slope stability has a safety factor higher than 1.2 the design shear strength are allowed to be 150 kPa (SGF, 2000).

2.2 Impact on the surrounding soil

When installing LC-columns, air-pressure is applied and mass is added to the soil. The added mass and the air-pressure have an impact on the surrounding soil. During the installation process the binder is injected with a pressure higher than the initial total horizontal pressure in the soil at the current depth. Due to the intense mixing during the installation, the soil and the binder becomes a viscous mass. The soil pressure increases during and just after the injection. This normally causes vertical heave on the ground surface and horizontal displacement, which varies with the depth. The displacement increases continually during the hardening process.

The installation of LC-columns also has an impact on the pore pressure, both in the stabilized soil and in the unstabilized soil. The pore pressure normally increases and therefore the shear strength and the stability of the soil decrease. When installing LC-columns in an area with low safety it is important to monitor the pore pressure change. Most of the change in pore pressure only lasts for a short time, but it sometimes takes a couple of months for the pore pressure to decrease to the values prior to the installation (Larsson, 2006).

2.3 Control of lime-cement columns

The quality of the installed LC-columns needs to be controlled. The controls differ depending on the soil profile, the topography and the purpose of the installation. This section presents basic and additional controls made when LC-columns are constructed.

2.3.1 Basic control

A basic control of LC-columns needs to be carried out in every project. A basic control includes information about how and when the columns have been made. It includes production date, column label, installation order, machine used and measurements of amount of mixed binder with the depth. Regarding the installation of the columns a specification about mixing tool, injection pressure, uplifting rate and rotation speed must be given. All these above mentioned data are documented during the installation (SGF, 2000).

2.3.2 Additional controls

A basic control is not enough in projects with a large amount of LC-columns or when the geotechnical conditions are difficult. Additional controls are made to ensure that other properties, not included in the basic control, fulfil the requirements.

Shear strength control

The first step in larger LC-column projects is to install test-columns. Test-columns are installed with different recipes and installation parameters in all areas where stratification differs. Strength growth of installed columns varies depending on the recipe and the soil properties. The time required to reach required strength are determined through laboratory testing and is normally between two and six weeks. When test-columns have reached the desired age the columns are tested to evaluate the properties of the columns. The most cost-effective design for each area is chosen and production columns are thereafter ready to be installed. Further controls of production columns are made through the entire installation phase and these controlled columns are named control-columns. Controls are made continuously in order to verify that the strength growth corresponds to the results from the test-columns installed with the same recipe (SGF, 2000).

The method most commonly used in Sweden for control of undrained shear strength is the column penetration test. When performing a column penetration test a winged probe is pushed through the center of the column with a speed of approximately 20 mm/s. The wing width is a bit shorter than the column diameter and it should not reach outside the column during penetration. The force needed to reach 20 mm/s is registered continuously and is used for estimating the shear strength. This method is applicable to a depth of 8 m and for columns with maximum shear strength of 150 kPa. If the method is performed at greater depths or on columns with higher shear strength, pre-drilling is needed. Pre-drilling makes the method useful to a much larger depth and in columns with a shear strength of 300-350 kPa.

The measured probing resistance is reduced with the skin resistance in order to determine the correct resistance. Therefore additional penetration tests are carried out in unstrengthened soil and by comparing results from tests made in columns, estimations of the skin resistance is possible. The reduced force is then divided by the point and wing areas to calculate the penetration pressure. The shear strength in the strengthened soil can then be estimated to 0.1 times the penetration pressure. This could also be applicable on pre-drilled columns but only if they are considered to have a homogeneous cross-section (SGF, 2000).

Settlement-, pore-pressure- and soil displacement control

Apart from the shear strength control it is important to verify that columns and its surroundings act as assumed in the design. Settlements, soil displacements and pore pressure are three important properties to monitor. Settlements are monitored by using soil level gauges or horizontal tubes under an embankment. Monitoring has to continue until most of the settlements have occurred. Soil displacement and pore pressure does not always have to be monitored; only if the columns are to be installed in a natural slope with a safety factor lower than 1.5. Measurements have to start prior to the column installation in order to get a reference value. Soil level gauges are normally used to monitor vertical displacements (settlements and heave) and inclinometers are used to monitor horizontal displacements. Furthermore, piezometers are installed to monitor pore pressure. Monitoring needs to continue until all columns in the area of interest have reached the desired shear strength (SGF, 2000).

2.4 Applications

LC-columns are used in several different types of construction projects (Larsson, 2006). This section presents a few of the most common applications for LC-columns.

2.4.1 Reduce settlements

LC-columns are commonly used to reduce settlements in soft soils. Since the columns have a higher shear strength and stiffness than the original clayey soil they will contribute to reducing the settlements, the columns will also reduce the consolidation time. The installation pattern depends on the slope stability with corresponding safety factor in the area before the improvement. When the safety factor is higher than 1.0 it is acceptable to use singular columns in a rectangular or triangular pattern to reduce settlements, see Figure 2.3. However, if the safety factor is lower than 1.0, panels are required for reducing settlement and increasing the slope stability. The ability to reduce settlements is for instance often utilized when constructing an embankment on soft soil or when constructing a foundation of a small house (Larsson, 2006).

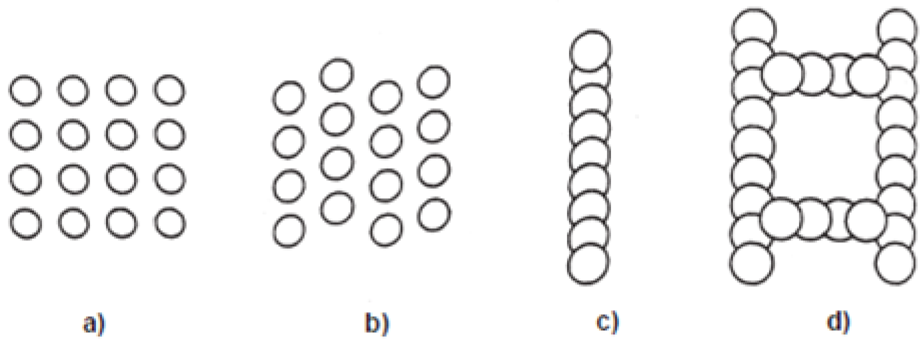

Figure 2.3 Different types of installation patterns a) Rectangular pattern b) Triangular pattern c) Panel d) Lattice (Larsson, 2006).

2.4.2 Increase stability

When installing LC-columns the stability in the soil is increased due to the increased shear strength in the columns. LC-columns are used to increase the stability when the safety factor for an area is considered too low. The columns are installed in panels or lattices to maximize the effect of the columns, see Figure 2.3. There are several situations where LC-columns are used to increase stability. Common situations are when an embankment is constructed on soft soil as well as when stabilization is needed for natural slopes, sheet pile walls, excavations and cuts in clay near roads and railways (Larsson, 2006).

2.4.3 Reduce vibrations

Reducing vibrations with LC-columns are mainly used in connection to railways. When high speed trains runs on tracks, considerable vibrations occur that will spread in soft soils and disturb the residents living close to the railway. LC-columns can be installed directly under the railway tracks to prevent large vibrations to occur. They can also be installed alongside the railway and work as a barrier to protect vibration-sensitive objects from large vibrations. If LC-columns are installed under a railway or alongside as a barrier, the columns are installed in a lattice with overlapping columns, see Figure 2.3. LC-columns for reducing vibrations can also be installed as singular columns in a rectangular pattern in cases where the vibration source is different from a railway (Larsson, 2006).

3 Geotechnical Investigations

This section presents some of the geotechnical investigations made in the area around section 465/300. The different investigations performed are presented in order to evaluate and determine soil properties, soil displacements and hydro-geological properties in the area.

3.1 Cone penetration test

Probing investigations, like cone penetration test (CPT), are carried out for determination of stratigraphy and soil thickness. Especially inhomogeneities in soil layers, such as looser and firmer layers, are noted but also shear strength can be estimated (Commission on slope stability, 1995).

When cone penetration tests are performed, the force against the conical tip is measured directly in ground with an electric sensor. This technique provides a continuous strength profile throughout the soil. Information on point resistance (q_T), frictional resistance and pore pressure are gained and used for evaluation of soil type, stratigraphy and shear strength (SGF & BGS, 2001).

The undrained shear strength (τ_{fu}) can be evaluated from the point resistance together with total overburden pressure (σ_{vo}) and liquid limit (w_L), see equation (3.1) below. A more accurate evaluation therefore demands parallel sampling with determination of liquid limit.

$$\tau_{fu} = \frac{q_T - \sigma_{vo}}{13.4 - 6.65 \cdot w_L} \tag{3.1}$$

Undrained shear strength values evaluated from cone penetration tests should not be used without additional investigations, unless the values are high enough to ensure that no problems will occur (Commission on slope stability, 1995).

3.2 Vane test

In-situ tests are made for determination of soil properties such as shear strength and deformation properties. In contrast to probing and sampling with subsequent laboratory investigations, in situ test gives more direct soil property information. Vane test (Vb) is the most common in situ method and is presented in this section.

Vane tests are routinely used for evaluation of the shear strength in cohesion soils. A vane apparatus is driven into the soil which consists of two perpendicular plates located in the bottom and a measuring instrument placed at the top. The measuring instrument registers the applied rotating force which is applied on chosen depths. The required moment force for failure in the surrounding cylinder, created from the vane, is registered. From the measured maximal moment force ($M_{\max}$), the average undrained shear strength (τ_v) can be calculated and estimated. Vane tests are normally taken at each meter below surface or on preselected levels. Vane tests can also be used for determination of sensitivity. In order to determine the sensitivity, the vane continues to rotate after maximal moment force has been registered, in order to achieve a remoulded soil state. In this way the moment force required for failure after stirring ($M_{\min}$) can be registered and the ratio between the moment forces indicates the sensitivity (S_t), see equation (3.2) (Bergdahl, 1984).

$$S_t = \frac{M_{\max}}{M_{\min}} \tag{3.2}$$

Vane tests are often made in connection with sampling points for better estimation of shear strength and sensitivity in cohesion soils. Results from vane tests can also be used for calibration of probing results. In order to estimate the undrained shear strength (τ_{fu}), the shear strength results from vane tests (τ_v) need to be corrected with a correction factor (μ) because they tend to be too high, see equation (3.3). The correction factor is determined on basis of the liquid limit (w_L) condition on each depth where the shear strength was measured, see equation (3.4). Correction factors above 1.2 should not be used without additional investigations (Commission on slope stability, 1995).

$$\tau_{fu} = \mu \cdot \tau_v \tag{3.3}$$

$$\mu = (\frac{0.43}{w_L})^{0.45} \geq 0.5 \tag{3.4}$$

3.3 Deformation measurements with an inclinometer

In cases where on-going or probable future soil movements, due to construction activities, could occur, soil-level gauges and inclinometers can be installed (Commission on slope stability, 1995). Vertical and horizontal movements can then be measured and monitored. Only inclinometers have been used in the investigated sections and information regarding inclinometers is presented in this section.

An inclinometer measures horizontal movements in the soil. It is commonly used for monitoring soil displacements in situations where probability for landslide is high and during construction. An inclinometer consists mainly of inclinometer sensors and a flexible pipe that is driven down into the soil. The inclinometer sensors are located inside the pipe at chosen depths, often on every meter. This pipe has a low bending stiffness and will deform if the soil moves.

The inclinometer has to be driven down to a depth where the end of the pipe no longer is subjected to horizontal movements, preferably down to firm bottom. This fixed point is used as a reference point. When the soil deform at a certain depth, the pipe will follow the deformation (Commission on slope stability, 1994). After the deformation, the inclinometer sensors will deviate from their original vertical positions and the deformation is interpreted as a change of inclination. The sensors are connected to equipment that registers the data with a desirable time interval. This equipment does also monitor the data and if the deformations exceed the pre-set limit an alarm will be sent. The reference point, the inclination of the sensors and the distance between the inclination sensors makes it possible to calculate the total deformation through the soil profile (Svärd, 2010).

3.4 Hydro-geological investigations

Hydro-geological investigations are often carried out in combination with field investigations. Groundwater relations are in a geotechnical aspect important in connection with stability and settlements as well as for excavations, drains and durability of piles.

Both ground water tubes and piezometers can be installed in order to estimate ground water level and pore-water pressure in the soil. This is an important part in geotechnical investigations because of great significance when calculating effective stresses. The ground water level in permeable soils is measured with open tubes and in less permeable soils pore pressure measurements with piezometers are used, either with an open or a closed system (Sällfors, 2001).

3.5 Laboratory investigations

This section presents methods for sampling and also different tests and properties which are investigated in a laboratory.

3.5.1 Sampling

Sampling is necessary for estimation and verification of soil composition and sampling carried out is either disturbed or undisturbed, depending on designated method (Commission on slope stability, 1995). This section presents different methods used for sampling.

Sampling is carried out in order to achieve complement for other investigations like probing in order to determine stratigraphy. Sampling is also necessary for determination of required correction factors when evaluating field investigations. The need for sampling varies depending on geological conditions and purpose of the investigation (Sällfors, 2001).

Undisturbed samples can be taken in clay, silty clay and to some content in clayey silt. The most common method for undisturbed samples is a piston sampler, where sampling can be made each meter and three tubes with soil are obtained. The tubes are normally 17 cm long with a diameter of 5 cm. Samples in the middle tube and the upper part of bottom tube can be seen as undisturbed. The material in the top tube is disturbed but can be used in other laboratory investigations appropriate for disturbed samples.

Disturbed samples are taken if undisturbed samples are difficult to take or if the investigation only requires disturbed samples. There are several methods to obtain disturbed samples but the most common method used for cohesion and silt soils is a helical auger. It consists of a speared steel rod with a helical flange, which by rotation is driven into the soil, either manually or by machine. Samples can be taken from surface down to 5–10 m depth depending on soil type (Bergdahl, 1984)

3.5.2 Routine tests

Soil properties which normally are tested routinely on samples are soil type, density (ρ), natural water content (w) and liquid limit (w_L) which can be determined from either disturbed or undisturbed samples. Furthermore is also undrained shear strength (τ_{fu}) and sensitivity (S_t) determined by tests made on undisturbed samples. Undrained shear strength, liquid limit and sensitivity in cohesion soils is normally determined in a laboratory with a fall-cone apparatus (Commission on slope stability, 1995). The liquid limit specifies the water ratio when transition between ductile and liquid consistency in the sample occur. Furthermore, the sensitivity defines the ratio in soil between the undrained shear strength in undisturbed state (τ_k) and the undrained shear strength in remoulded state (τ_R) (Sällfors, 2001).

3.5.3 Constant rate of strain test

Constant rate of strain (CRS) tests are performed with an oedometer where the sample is contained in a ring, placed against a bottom plate and during the test a load is applied, see Figure 3.1. At top and bottom of the sample there are filter stones placed for drainage, for CRS tests the bottom drainage filter is prevented in order to achieve a one side drainage situation. CRS tests are routinely performed on undisturbed clay samples while tests on sand are uncommon due to difficulties in collecting undisturbed sand samples.

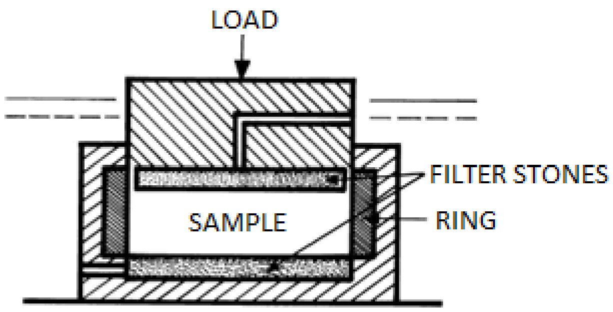

Figure 3.1 Principal sketch of a CRS test (Larsson, 2008, modified).

In a CRS test the sample is deformed with a constant rate and the sample is as mentioned one sided drained. A piezometer is placed on the undrained side and registrations of applied force, deformation and pore pressure are made during the test. Effective vertical stress and compression can therefore be evaluated. Pre-consolidation pressure is also evaluated from the plotted results in a diagram and the compression modulus variation with the effective stress is presented. From the results concerning pore pressure and deformation rate, permeability can be calculated and with this known the coefficient of consolidation can be determined (Sällfors, 2001).

CRS tests can also be used for empirical evaluation of shear strength when the pre-consolidation pressure has been estimated. This can be carried out by comparing CRS results with assemblies of test results from Scandinavian clays which shows how the undrained shear strength (τ_{fu}) normally varies with pre-consolidation pressure (σ'_c) and liquid limit (W_L) (Larsson, 2008).

3.5.4 Direct shear tests

Direct shear tests are performed for evaluation of shear strength, drained or undrained, on undisturbed samples that have been consolidated for in-situ stresses. Results from direct shear tests should be used as complement on levels where other investigations presents a large distribution of shear strength. In soils where uncertainties of relevant correction factors and disturbance on in-situ tests are facts, correct managed direct shear tests normally is considered the most reliable (Commission on slope stability, 1995).

The shear test is carried out in a shear-apparatus where a force (N) first is applied on the sample, resulting in a normal stress which the sample consolidates for. A shear force (T) is then slowly applied at the sample top surface while the sample height is locked, see Figure 3.2. The drainage ways are either closed or opened. Failure will occur in horizontal plane and during the test the applied shear force and associated shear deformations are registered. Tensions in failure plane are therefore the applied normal stress and the shear force and at least two tests are performed where the failure values are presented in a diagram. The strength parameters, shear strength and friction angle, can be evaluated for both drained and undrained conditions (Sällfors, 2001).

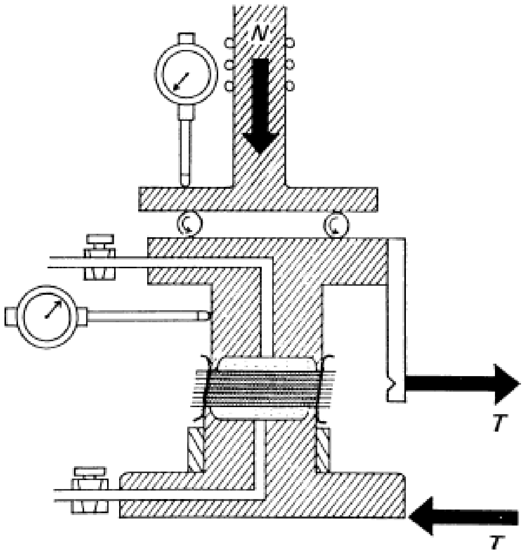

Figure 3.2 Principal sketch of a shear-apparatus (Larsson, 2008).

3.5.5 Triaxial test

When performing a triaxial test the cylindrical undisturbed sample is enclosed in a rubber membrane and placed in a cell filled with a liquid, see Figure 3.3. The sample is normally applied with a vertical load, P, and associated load change, deformation and pore-pressure are measured. Horizontal stresses are applied through liquid pressure (σ_3) and the relation between vertical- and horizontal stresses can be regulated by either changing the liquid pressure or by applying either tension or compression in the load mechanism. Before the testing starts, consolidation of the sample is made by loading the sample with predetermined stresses which corresponds to in-situ stresses and properties as in-situ where the sample was taken.

Triaxial tests are either performed as active, where the vertical load is increased, or passive, where the vertical load is decreased, while the liquid pressure is held constant. Draining conditions are regulated with a tap, closed or opened, and undrained or drained tests can therefore be performed. Triaxial tests are carried out in order to evaluate strength- and deformation properties in the soil (Larsson, 2008).

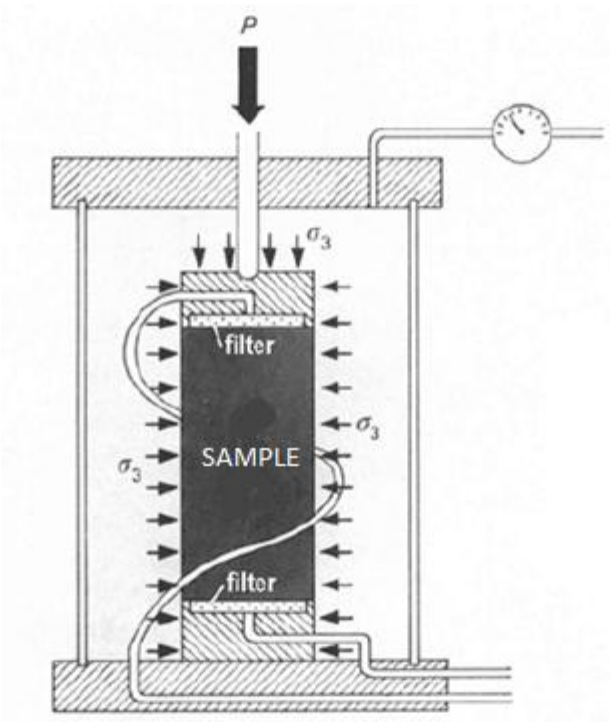

Figure 3.3 Principal sketch of a triaxial test (Larsson, 2008, modified).

4 Calculation methods for soil displacement

In order to predict soil displacements due to installation of LC-columns, different methods have been investigated. There is no specific praxis or method used in Sweden today for prediction of soil displacements due to installation of LC-columns. Therefore, known methods normally used for estimating displacements due to piling will be presented. These methods are later in this thesis applied for LC-columns. One empirical method, one analytical method and one numerical method will be investigated in this thesis and are presented in this chapter.

4.1 Rehnman's method

Sven-Erik Rehnman presented in 1993 an empirical model for calculations of surface heave and horizontal soil movement due to piling, based on work made by partly Lars Hellman and partly Dugan and Freed.

When installing mass displacing piles in clay the displaced soil is simplified equal to the volume of the installed piles. If pre-boring has been made, where soil has been removed in order to reduce ground heave and horizontal deformations, the total displaced soil volume is reduced with the removed soil volume.

The model is based on a central assumption that ground heave in connection to piling only occurs within an area defined by lines with a 45 degree angle from pile tip and up, see Figure 4.1. Ground heave (x) within the piling area at a horizontal ground surface can be calculated by using relations given in equation (4.1). By assigning relative ground loads for the buildings within the areas A-D, ground heave and its distribution can be estimated in these areas (Hintze, S et al., 1997).

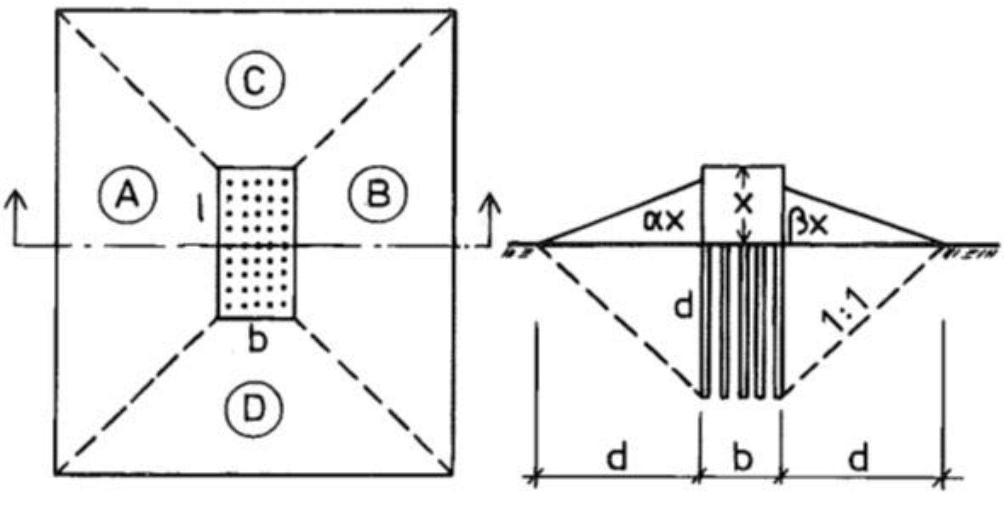

Figure 4.1 Ground heave in connection with piling, inside and around the piling area (Hintze, S et al., 1997).

$$x = \frac{\eta\left(V_{piles} - \Delta V_{pr}\right)}{d\left[(\alpha+\beta)\left(\frac{l}{2}+\frac{d}{3}\right)+(\gamma+\delta)\left(\frac{b}{2}+\frac{d}{3}\right)+\frac{bl}{d}\right]} \tag{4.1}$$

$x =$	Heave within piling area	$\gamma x =$	Heave of building C
$\eta =$	Heave factor	$\delta x =$	Heave of building D
$V_{piles} =$	Volume of driven piles	$\alpha, \beta, \gamma, \delta =$	Relative load from buildings
$V_{clay} =$	Volume of soil removed with pre-boring	$d =$	Pile depth from surface
$\alpha x =$	Heave of building A	$b =$	Width of piling area
$\beta x =$	Heave of building B	$l =$	Length of piling area

The heave factor (η) takes into account that clay to some content is compressible, therefore not all of the displaced soil volume results in ground heave. The heave factor varies between $0.5 \le \eta \le 1.0$ and normally $\eta = 0.75$ is applied.

The relative loads assigned for area A-D varies between $0 \le \alpha, \beta, \gamma, \delta \le 1$ where 1 corresponds to a light building and 0 corresponds to a heavy building.

The method described above can as mentioned be used for estimation of ground heave but also for horizontal displacement in area A-D outside of the piling area, see Figure 4.2. This can be carried out due to an assumption that the ground surface heave is equal to the horizontal displacement at the ground surface. Ground surface heave (δ_v) and horizontal displacement on ground surface ($\delta_{h,surface}$) can be estimated on a certain distance (e) from the piling area with equation (4.2) below. Horizontal displacement can also be estimated on different depths and is just as the heave limited by lines with a 45 degree angle from pile tip and up. Further, the horizontal displacement distribution with the depth (δ_h) can be estimated on a specific depth (z) from surface, see equation (4.3).

$$\delta_v(e) = \delta_{h,surface}(e) = \alpha \cdot x \cdot \left(1 - \frac{e}{d}\right), \qquad 0 \le e \le d \tag{4.2}$$

$$\delta_h(z) = \delta_{h,surface}(e) \cdot \left(1 - \frac{z}{d-e}\right), \qquad 0 \le z \le (d-e) \tag{4.3}$$

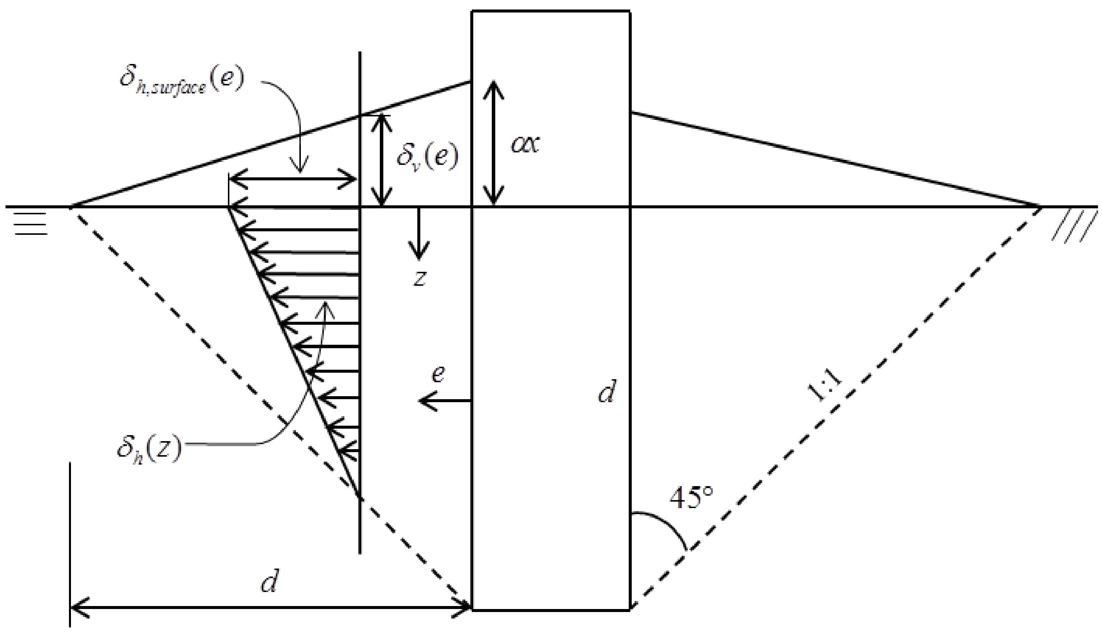

Figure 4.2 Ground surface heave and horizontal displacement distribution with the depth outside of the piling area (Hintze, S et al., 1997, modified)

Rehnman's method is applied for LC-columns in this thesis and the input data and the results are presented in Chapter 8.

4.2 The shallow strain path method

The shallow strain path method (SSPM) is an analytical method which can be used to estimate vertical and horizontal deformations when driving piles in clay. The method was presented by Sagaseta, Whittle and Santagata (1997).

In the SSPM the pile is seen as a line with point sources sending out an irrotational flow of an ideal fluid, see Figure 4.3. The fluid creates a spherical flow- and displacement field. To fulfil the boundary of a normal stress-free ground surface some assumptions are made. Image sinks are placed over the ground surface at the same distance to the ground surface as the sources. The sources and the sinks have the same properties except that the sources are creating a positive displacement field while the sinks create an equally large but negative displacement field. This will remove the normal stresses at the ground surface but double the shear stresses. To cancel out the shear stresses, corrective shear tractions are added at the ground surface.

With the assumptions described above, the SSPM will fulfil the boundary of a stress-free ground surface. As the corrective shear tractions cancel out the shear stresses they also creates a displacement field which should be added to the already existing ones. The resulting displacement field is independent of the soil modulus. The assumption regarding the corrective shear tractions requires that the soil has a linear elastic behaviour (Sagaseta & Whittle, 2001).

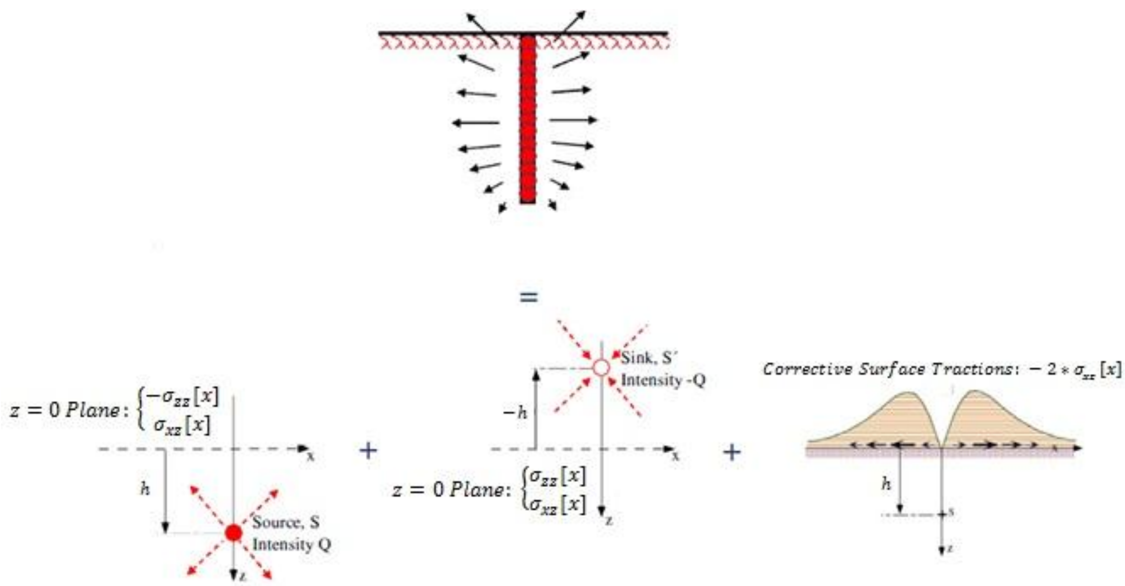

Figure 4.3 *Steps included in the theory of the SSPM when estimating soil displacement due to piling, at the ground surface* (Edstam, 2011) .

When considering deformations very close to a driven pile, large strain theory is required. At some distance from the pile, small strain theory is a good approximation for estimating the deformations. From small strain theory it is possible to get closed-form expressions for three different geometries; a simple wall, a simple tube and a simple pile. These geometries are presented in Figure 4.4 and the associated solutions are presented below.

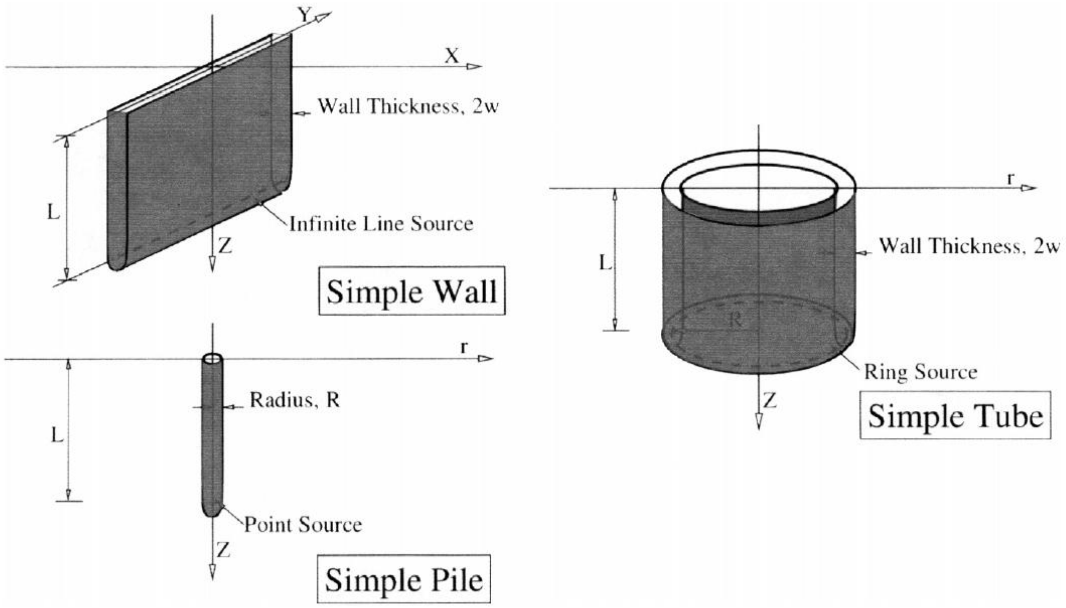

Figure 4.4 *The geometries and notations used in the SSPM analysis* (Sagaseta & Whittle, 2001).

Cylindrical Pile

For a cylindrical pile, the vertical and horizontal displacement $(\delta_{zSS}, \delta_{rSS})$ at the ground surface at a chosen radial distance (r) depends on the pile length (L) and the pile radius (R). Equation (4.4) is used to calculate the horizontal displacement while equation (4.5) is used to calculate the vertical displacement.

$$\delta_{rSS}(r,0) = \frac{R^2}{2} \cdot \frac{L}{r \cdot \sqrt{r^2 + L^2}} \tag{4.4}$$

$$\delta_{zSS}(r,0) = -\frac{R^2}{2} \cdot \left(\frac{1}{r} - \frac{1}{\sqrt{r^2 + L^2}} \right) \tag{4.5}$$

Tube

For a tube, the horizontal displacement at ground surface is equal to zero while the vertical displacement (δ_{zSS}) is calculated with equation (4.6). The displacement depends on the tube length (L), the tube radius (R) and the tube wall thickness $(2w)$.

$$\delta_{zSS}(0,0) = -2wR \cdot \left(\frac{1}{R} - \frac{1}{\sqrt{R^2 + L^2}} \right) \tag{4.6}$$

Planar wall

In order to calculate displacements with the case of a planar wall, the wall has to be assumed to have an infinite length. Unlike the calculations for a cylindrical pile, it is possible to calculate both horizontal and vertical displacement at different depths $(\delta_{xSS}, \delta_{zSS})$. The displacement at a certain distance (x) and at a certain depth (z) depends on the wall depth (L) and the wall thickness $(2w)$. The horizontal displacement is calculated with equation (4.7) and the vertical displacement with equation (4.8) (Sagaseta & Whittle, 2001).

$$\delta_{xSS} = \frac{w}{\pi} \cdot \left[\begin{array}{l} \tan^{-1}\left(\frac{z+L}{x}\right) - \tan^{-1}\left(\frac{z-L}{x}\right) \\ + 2xz \cdot \left(\frac{1}{x^2 + (z+L)^2} - \frac{1}{x^2 + z^2} \right) \end{array} \right] \tag{4.7}$$

$$\delta_{zSS} = -\frac{w}{\pi} \cdot \left[\begin{array}{l} \ln\left(\frac{\sqrt{[x^2 + (z-L)^2] \cdot [x^2 + (z+L)^2]}}{x^2 + z^2} \right) \\ - 2z \cdot \left(\frac{z+L}{x^2 + (z+L)^2} - \frac{z}{x^2 + z^2} \right) \end{array} \right] \tag{4.8}$$

The SSPM method for a planar wall and cylindrical piles are applied for LC-columns in this thesis and the input data and the results are presented in Chapter 8.

4.3 Finite element analysis, PLAXIS

The numerical simulation of displacements caused by the installation of LC-columns can be conducted with the finite element program PLAXIS. In this thesis PLAXIS 2D-version 11 was used.

In PLAXIS different advanced constitutive models are incorporated for simulating the linear or nonlinear behaviour of soil and rock. Also time dependent processes as for example consolidation, creep and flow can be analyzed and special procedures to deal with non-hydrostatic or hydrostatic pore pressures in the soil are available. Furthermore, PLAXIS enables to model the interaction between different structures and the soil. Information about the model used in this thesis and the results from the calculations made with PLAXIS are presented in Chapter 9.

5 Site Study

The studied area in this thesis is located around section 465/300 included in the contract E41b in the project BanaVäg I Väst. The information concerning section 465/300 together with the estimated soil properties presented in this chapter is based on information given by the Swedish Transport Administration.

The studied area is situated south of the town Surte and just north of the bridge Angeredsbron, see Figure 5.1. The contract included the construction of a new double-track railway with a service road and also the expanding of the road E45 into a four-lane highway. This chapter presents a general description of the area and its geotechnical conditions.

Figure 5.1 The studied area around section 465/300, marked with a red circle (Svärd, 2010, modified).

5.1 General description

The area is fairly flat, with a small inclination towards the river Göta älv on the western side. Prior to the project there was a single-track railway (Norge/Vänerbanan) parallel to the river, about 35 m to the east. The road E45 was located at the eastern side of the single-track railway, followed by a mountain rising with an inclination of 2:9 to the east; see Figure 5.1 and Figure 5.2.

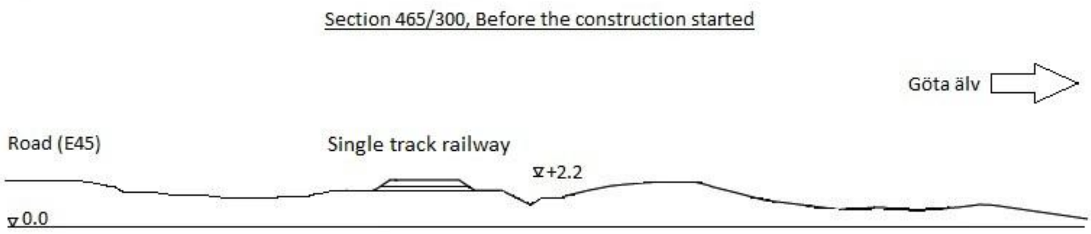

Figure 5.2 Section 465/300 before the construction started.

When the construction started in the studied area, the first step was to construct a service road strengthened with LC-columns. The service road was constructed to ease the construction phase and the LC-column installation for the new double-track railway. The service road is located about 20-25 m east of Göta älv and centrum-centrum (cc) distance between the service road and the double-track railway is approximately 12.5 m, see Figure 5.3. The old single-track railway was replaced by the new railway. The expanded road, E45, is still located to the east of the railway.

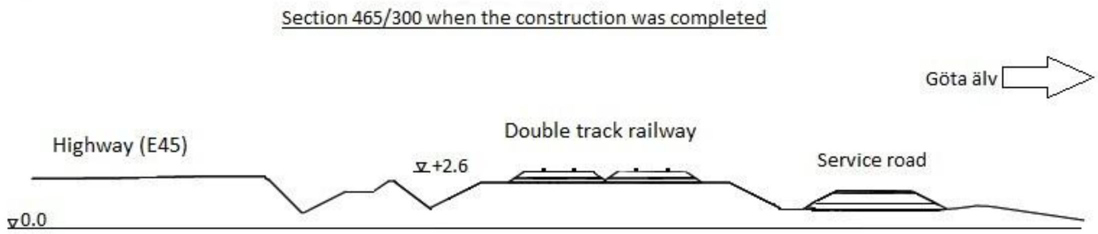

Figure 5.3 Section 465/300 when the construction was completed.

5.2 Geotechnical conditions

From the geotechnical investigations made in the area, stratigraphy and soil properties have been evaluated and are presented in this section. Many tests have been carried out both in field and in laboratory and the performed tests are listed below. The location of the boreholes used for the evaluation of soil properties and stratigraphy are seen in Appendix 10.

Field investigations: CPT tests, Soil/rock probing tests, Vane tests**,** Pyramid penetration tests.

Laboratory investigations: Routine tests, Direct shear tests, Triaxial tests, CRS tests.

5.2.1 Soil stratigraphy

The soil stratigraphy in the area has been determined from tests made in section 465/300 and nearby. A picture of the approximated soil stratigraphy in the section can be seen in Figure 5.4 and the centerline where the service road was constructed is marked. This section describes the soil stratigraphy at the location where the service road was built.

The surface level at the location of the service road is approximately at level +1. From piezometers installed it can be seen that the ground water level lies about 0.5 m below the ground surface, with a small inclination towards the river. The top soil layer, 0.5 - 1 m, consists of dry crust clay and the following layer consists of a thick clay layer down to level -52 m. The eastern top soil layer in the section consists of fill under the existing road and railway. The bedrock layer starts at the bottom level of the clay layer which is indicated from tests made in the section. It can also be seen that the bedrock have a large westerly inclination in section 465/300.

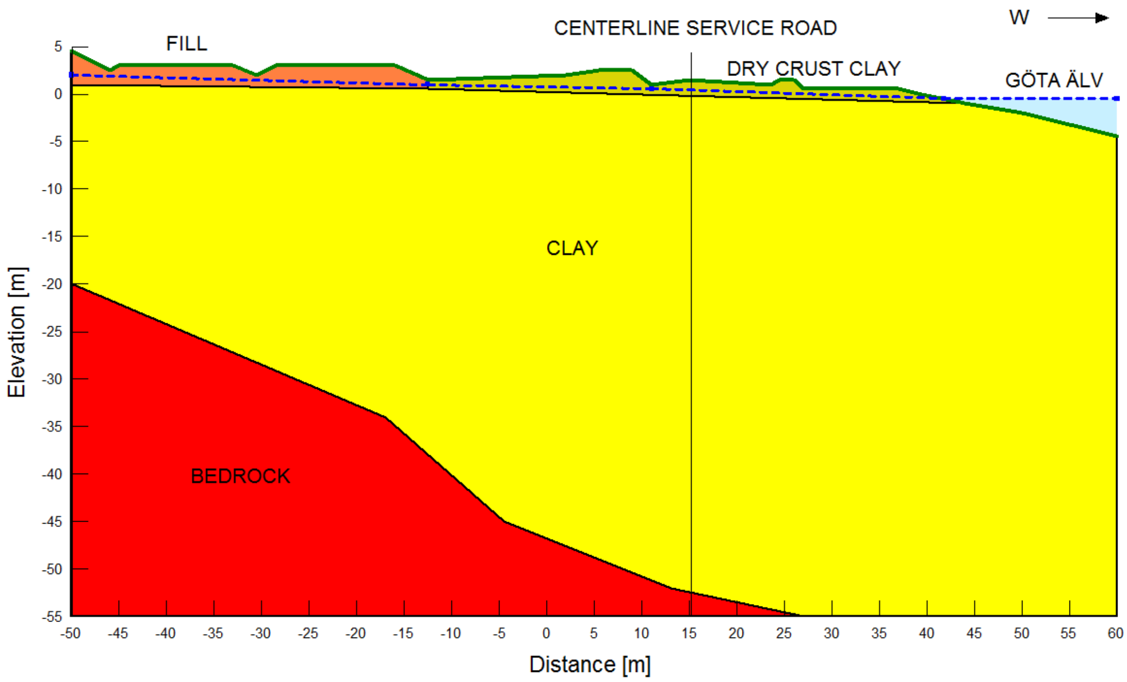

Figure 5.4 Estimated soil stratigraphy in section 465/300, the centerline of the constructed service road is marked.

5.2.2 Soil properties

The soil properties in the area around the service road have been evaluated and estimated from the different investigations made. The clay in this area is extremely soft and due to the short distance to Göta älv there is severe geotechnical conditions.

The soil deposit in the site consists of approximately 50 m of soft clay under the service road. The characteristic undrained shear strength is evaluated from tests made and the shear strength is estimated and presented in Figure 5.5 below. The measured undrained shear strength from the vane tests have been corrected according to equation (3.3) and (3.4). The undrained shear strength determined with a fall-cone apparatus have been corrected in the same manner.

It can be seen that the corrected undrained shear strength is 8 kPa at level -1 m, 2 m below ground surface, and increases to 16 kPa at level -9 m. Beneath this level the strength increases and advancing about 1.8 kPa per metre until a shear strength of 25 kPa at level -14 m is reached. From level -14 m to -23 m the shear strength increases with around 0.2 kPa per metre and from -23 m to -37 m the characteristic shear strength increases with about 1.4 kPa per metre. From the level -37 m down to the bedrock there has not been any tests made regarding undrained shear strength. The shear strength in the clay with depths below level -37 m can therefore not be estimated but it could be assumed to increase with the depth.

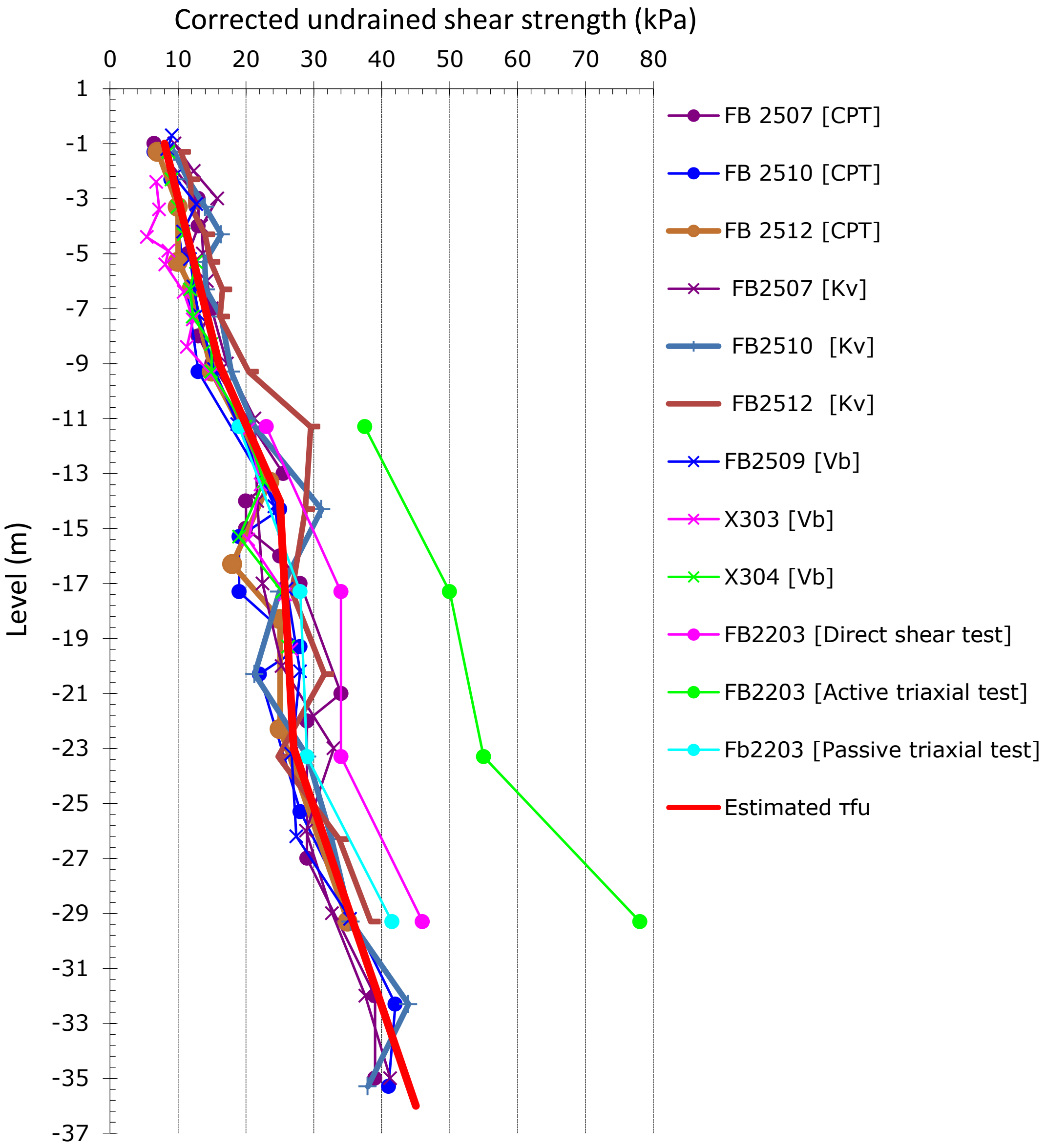

Figure 5.5 Corrected undrained shear strengths from tests made in the area and the estimated undrained shear strength is symbolised by a red line.

The soil density has been determined from tests made in laboratory and the density of the clay varies between 1.50 – 1.65 ton/m^3, see Appendix 1.

The liquid limit variation in the clay layer is presented in Appendix 1 and it can be seen that the liquid limit varies approximately between 80 – 90 % the top 10 meters, from +1 m down to -9 m. The liquid limit is 70 – 80 % between -9 down to -19 and from -19 to -37 the liquid limit is approximately 60 -70 %.

The pre-consolidation pressure (σ'_c) have been estimated from CRS tests made in three different boreholes in the area nearby section 465/300. The pre-consolidation pressure and the effective in-situ stress (σ'_0), calculated from the estimated density, are presented in Figure 5.6 below. It can be seen that the pre consolidation pressure is higher than the effective in-situ stress which means that the clay is slightly over consolidated. In order to estimate the over consolidation rate, OCR, a line where the effective in-situ stress is multiplied with 1.25 is plotted. As seen in the figure, the plotted line is in good agreement with the plotted pre-consolidation pressure values. This gives an indication that the clay is over consolidated and the OCR value is equal to approximately 1.25.

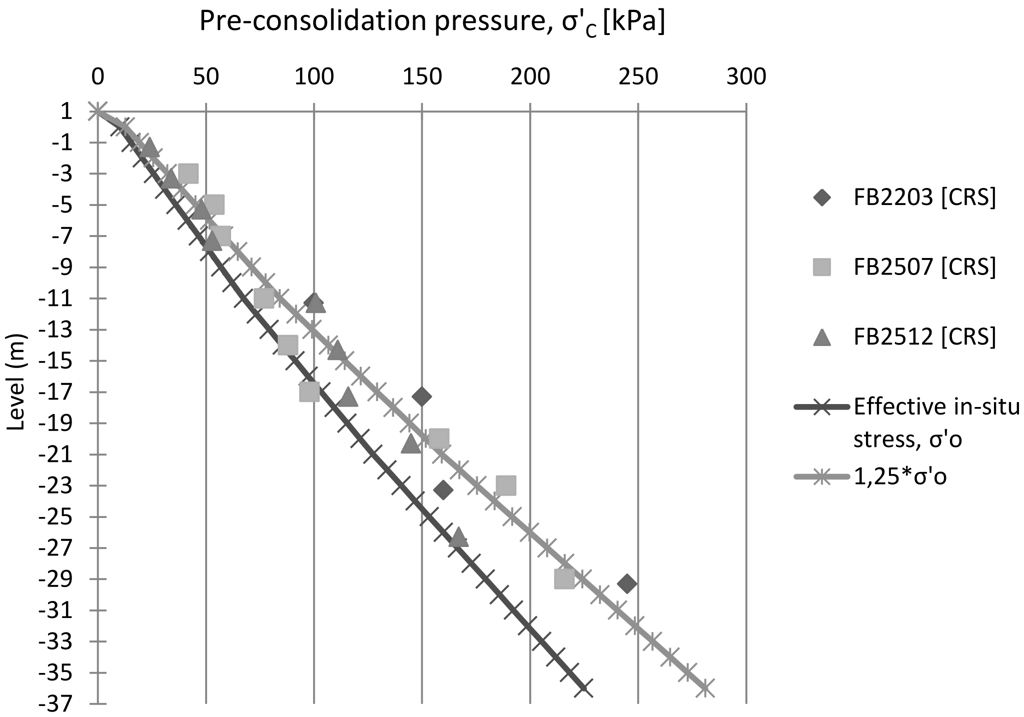

Figure 5.6 *Plotted pre-consolidation pressures (σ'_c) from CRS-tests, effective in situ stresses and a line 1.25*σ'_0.*

6 Stabilized Service Road with Lime-Cement Columns

The service road have been constructed and stabilized with LC-columns and this chapter presents information concerning the installation of the LC-columns. Information presented is based on information given by the Swedish Transport Administration.

The geotechnical conditions around the studied service road are complicated. The clay is extremely soft and it was a close distance to Göta älv and the single-track railway during the LC-column installation. The safety requirements with a safety factor of 1.5 are not reached for the single-track railway. Due to the low safety, the installation procedure needs to be very carefully executed in order not to risk a landslide. The service road is designed to have a bearing capacity high enough to make it capable of transporting heavy vehicles during the construction of the new double-track railway. It is designed for a maximum pressure of 25 kN/m^2 and the stability requirements is according to safety class 2.

6.1 Installation pattern

The LC-columns installed for the service road in the studied area were installed on 30th April and 3th May 2010, for a more detailed presentation of installation time see Appendix 2. The installation pattern has a width of four columns and the LC-columns were installed from east to west in a northern direction, see Figure 6.1. The column rows are installed with a distance of ±0.75 m and ±2.25 m from the centerline of the service road. All columns have a cc distance of 1.5 m.

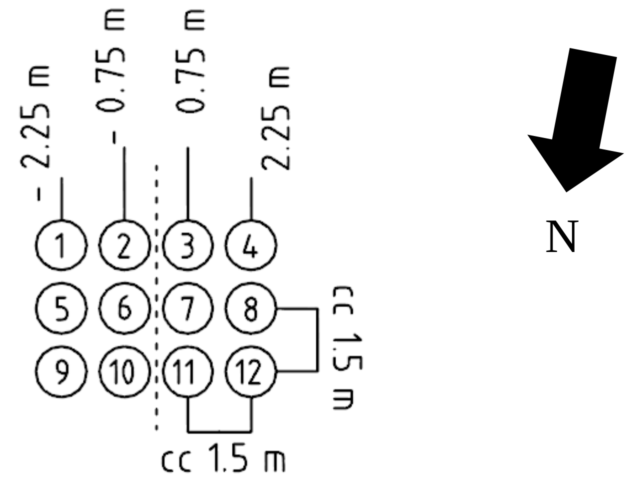

Figure 6.1 Installation order and installation pattern for LC-columns under the service road.

6.2 Installation properties

The LC-columns in this area were installed with a mixing tool of the type “Pinnborr” with a diameter of 0.6 m and with three mixing levels. The Installation level for the columns was +1.3 m and in the top 0.5 m no binder was injected, therefore the top of the columns are at level +0.8 m, see Figure 6.2. However, the bottom level varies because the columns length in the area varies between 6.2 - 6.5 m.

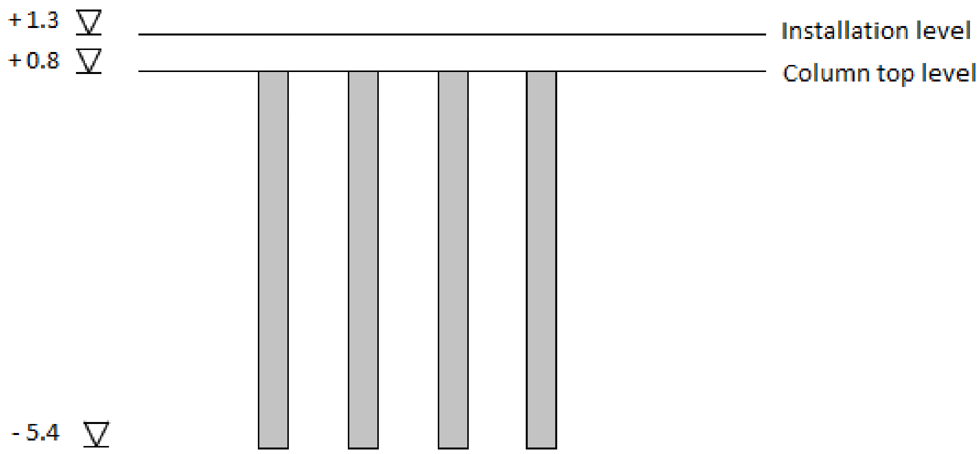

Figure 6.2 The installation level together with the column top- and bottom level.

The binder recipe used consisted of equal parts of lime and cement (50/50) and the amount of injected binder was approximately 30 kg/m (106 kg/m^3) through the entire column with a constant injection pressure of 550 kPa. The installation was made with a rotation speed of 140-160 rotations/min and with an uplifting rate of 20 mm per rotation. All columns in the area are installed vertically.

For more detailed information about the installation properties see Appendix 2, where the properties for each installed column are presented.

7 Measured Displacements

This chapter presents the soil displacement which has been measured in connection to the installation of LC-columns under the service road, in the area nearby section 465/300. The soil displacements have been measured with an automatic inclinometer located 6.35 m west of the center of the service road, in section 465/300, see Figure 7.1 below. The information concerning the inclinometer and the measurements presented in this chapter is based on information given by the Swedish Transport Administration.

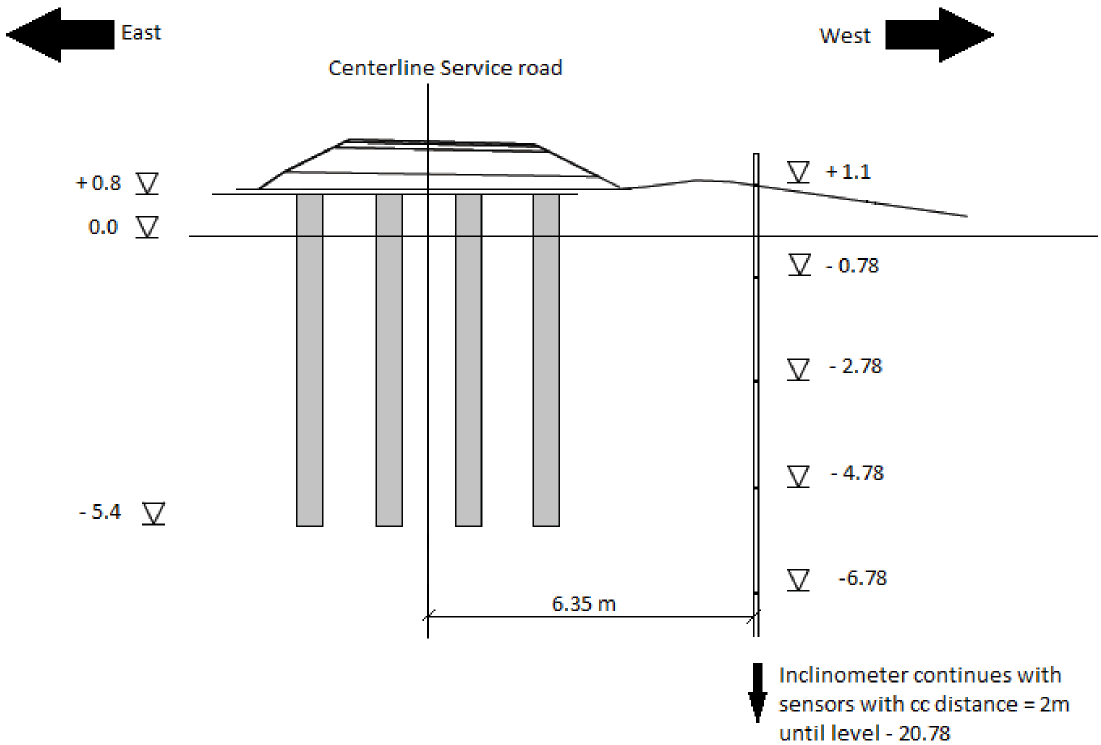

Figure 7.1 Location of the automatic inclinometer and the levels of the sensors.

Displacements have automatically and continually been measured every hour since the installation of the inclinometer. The installation was made long before the service road construction began and measurements were made until after the new double-track railway was constructed. The levels where the inclinometer sensors have been installed in the inclinometer pipe can be seen in Figure 7.1 above. The first sensor is installed at the ground surface, +1.1 m and the second sensor was installed at level -0.78 m and then there are ten sensors installed with a cc distance of 2 m until bottom level, -20.78 m.

Positive displacement values are seen as soil displacement in a westerly direction. The service road lies with an angle of 7 degrees from a north direction and therefore the measured displacements must be corrected with regard to the angle, see Figure 7.2. This was made in order to be able to compare measured displacements with calculated displacements, which will be calculated perpendicular to the service road, presented later in this thesis.

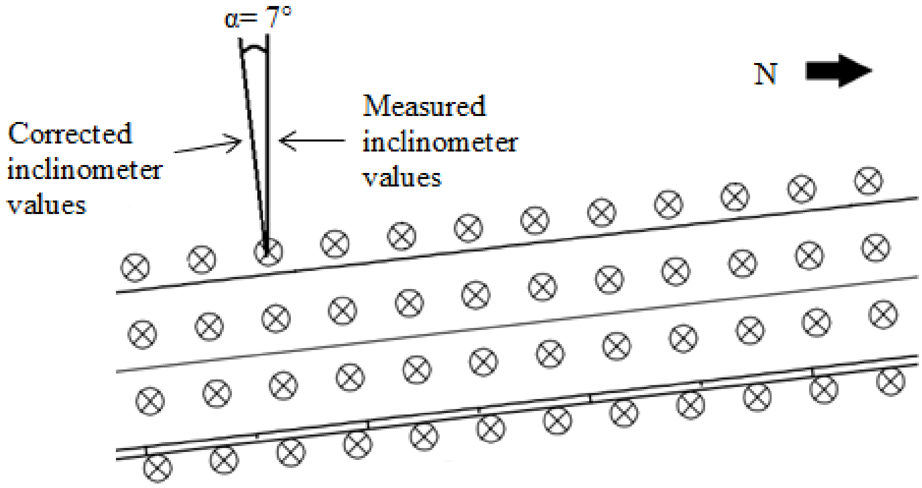

Figure 7.2 *The direction of the service road in relation to the westerly measured inclinometer values.*

A large amount of measurements have been generated from the automatic inclinometer which resulted in a comprehensive management and evaluation of data. An evaluation has therefore been carried out in order to decide when the inclinometer detected soil displacements in section 465/300 due to the installation of LC-columns. By comparing the time when the LC-columns where installed with the measurements from the inclinometer, a time span when the inclinometer detected soil displacements has been found. The evaluation shows that the inclinometer was affected and distinctive westerly soil displacements can be seen between 13:00 and 20:00 on April 30, 2010. The inclinometer values at 13:00 is therefore seen as a zero value, reference value, and westerly displacements occurring from this time is given as positive values, see Figure 7.3.

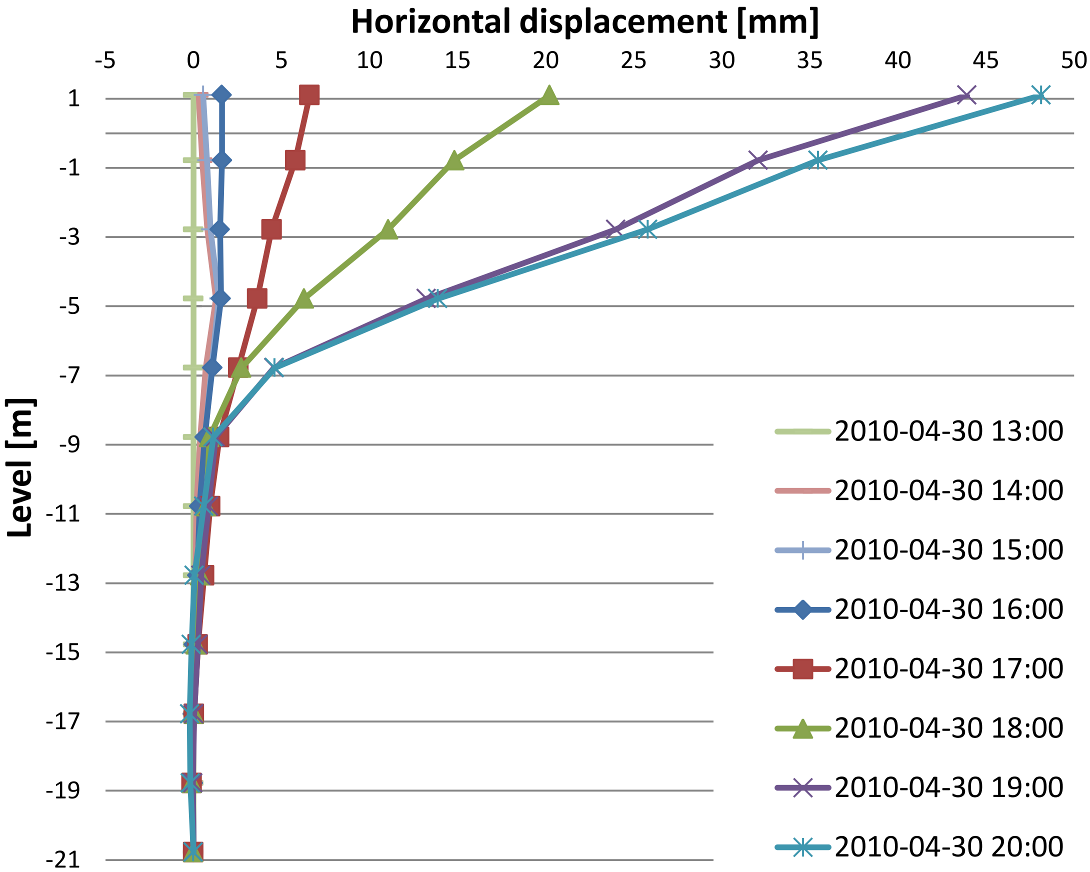

Figure 7.3 Horizontal displacements in section 465/300 during the installation of LC-columns under the would-be service road.

According to the time span and the detected soil displacements shown in the figure above, most of the soil displacement occurred during three hours between 16:00 - 19:00 while the total displacement occurred between 13:00 - 20:00. Therefore initial analyses made later in this report will be focused on four different phases. Phase A-C represents parts of the total installation phase where most of the total displacement occurred. Phase D represents the total installation phase where the maximal and total displacement, 48.1 mm, was measured in section 465/300.

Table 7.1 Properties and measured maximal displacement for the phases A-D, which will be used in the analysis, performed later in this thesis.

Phase	Time	Number of installed lime cement columns	Maximal displacement (mm)
A	16:00 – 17:00	10	5,0
B	16:00 – 18:00	26	18,6
C	16:00 – 19:00	44	42,3
D	13:00 – 20:00	85	48,1

8 Calculated Displacements with Rehnman's method and the Shallow Strain Path Method

This chapter presents calculations made with the SSPM and Rehnman's method for estimation of soil displacement when installing LC-columns in section 465/300. The SSPM for a planar wall and for a cylindrical pile are used in this chapter due to their ability to calculate horizontal displacement. The SSPM for a tube is not investigated in this thesis.

Since the methods are made for estimating displacements when piling or sheet piling, different approaches are investigated with a view towards making the methods suitable for LC-columns. The calculated displacements will be compared with the measured displacement in section 465/300. The result from the final approach will also be tested on additional sections.

The different calculations applied in this chapter are mainly made as for piles. Adjustment must however be made to account for the added mass volume. This is necessary because the added mass volume (the binder volume in the LC-column) does not correspond to the total volume of a column as in the case of piling.

8.1 Different approaches

This section presents the different approaches performed in order to adapt the methods for LC-columns. Five different approaches have been made and all approaches are related, each approach is a development from the previous one.

8.1.1 Approach 1

In the first approach calculations are made for Phase A-D using Rehnman's method and the SSPM for a cylindrical pile. The calculations with the SSPM for a planar wall are only made for Phase D due to the small amount of columns in the other phases. Phase D includes all columns that has an impact on the inclinometer and is therefore the only phase that could be seen as a planar wall with infinite length.

In the calculations made with Rehnman's method standard values are used regarding the heave factor and the factors for the relative load. For each phase the length of the piling area is set to the mean length of the LC-column area and the width of the LC-column area is constant in all phases. The calculated displacement needs to be adjusted since the column volume does not correspond to the actual added volume, which is the added binder volume. The calculated displacement is therefore multiplied with the ratio of the added binder volume in relation to the volume of the column, see equation (8.1). In order to obtain a calculated displacement which can be compared with the measured inclinometer displacement, the calculations are made at a point corresponding to the inclinometer location. The input data and calculations for Rehnman's method are presented in Appendix 3.

$$Ratio_{Binder/Column} = \frac{V_{Binder}}{V_{Column}} \qquad (8.1)$$

When calculating displacement with the SSPM for a cylindrical pile, the column length, the radius and the distance to the inclinometer are needed. In order to obtain a calculated total displacement from all columns at the inclinometer location, superposition is applied to sum the displacement from each column. The added binder volume does not correspond to the column volume and therefore the total displacement is adjusted by multiplication with the same ratio as in Rehnman's method, see equation (8.1). The ratio describes the relation between the added binder volume and the column volume, for one column. The adjusted displacement is calculated with equation (8.2). The input data and calculations with the SSPM for a cylindrical pile are presented in Appendix 4.

$$\delta_{rSS}(r,0) = \frac{R^2}{2} \cdot \frac{L}{r \cdot \sqrt{r^2 + L^2}} \cdot Ratio_{Binder/Column} \qquad (8.2)$$

In the calculations with the SSPM where the strip zone delineated by the rows of the LC-columns is considered as a planar wall, the width of the wall is seen as the width of the LC-column area. The calculated displacement is adjusted by multiplication with a ratio as in the calculations described above. The ratio in the planar wall case describes the relation of the total added binder volume for all columns in relation to the soil volume, defined by the LC-column area and column depth, see equation (8.3). The adjusted displacement (δ_x) is calculated with equation (8.4). For input data and calculations for the SSPM for a planar wall, see Appendix 5.

$$Ratio_{Binder/Soil} = \frac{\sum V_{Binder}}{V_{Soil}} \qquad (8.3)$$

$$\delta_x = \frac{w}{\pi} \cdot \left[\begin{array}{l} \tan^{-1}\left(\frac{z+L}{x}\right) - \tan^{-1}\left(\frac{z-L}{x}\right) \\ + 2xz \cdot \left(\frac{1}{x^2 + (z+L)^2} - \frac{1}{x^2 + z^2} \right) \end{array} \right] \cdot Ratio_{Binder/Soil} \qquad (8.4)$$

8.1.1.1 Results

This section presents the results of the calculations made with the SSPM and Rehnman's method using the adjustments for LC-columns, presented in Section 8.1.1 above.

The results for **Phase A**, including 10 columns, are presented in Figure 8.1 below, together with the measured displacement occurring in Phase A. The calculated displacement with Rehnman's method is 5.17 mm at the top level of the columns, with a linear decrease. The displacement is zero where the depth from the column top level is equal to the distance to the inclinometer. In the cylindrical pile case with the SSPM the displacement is 2.90 mm at the column top level. The maximal measured displacement occurs at the top level of the inclinometer and is 5.0 mm and decreases with the depth, see Figure 8.1 below.

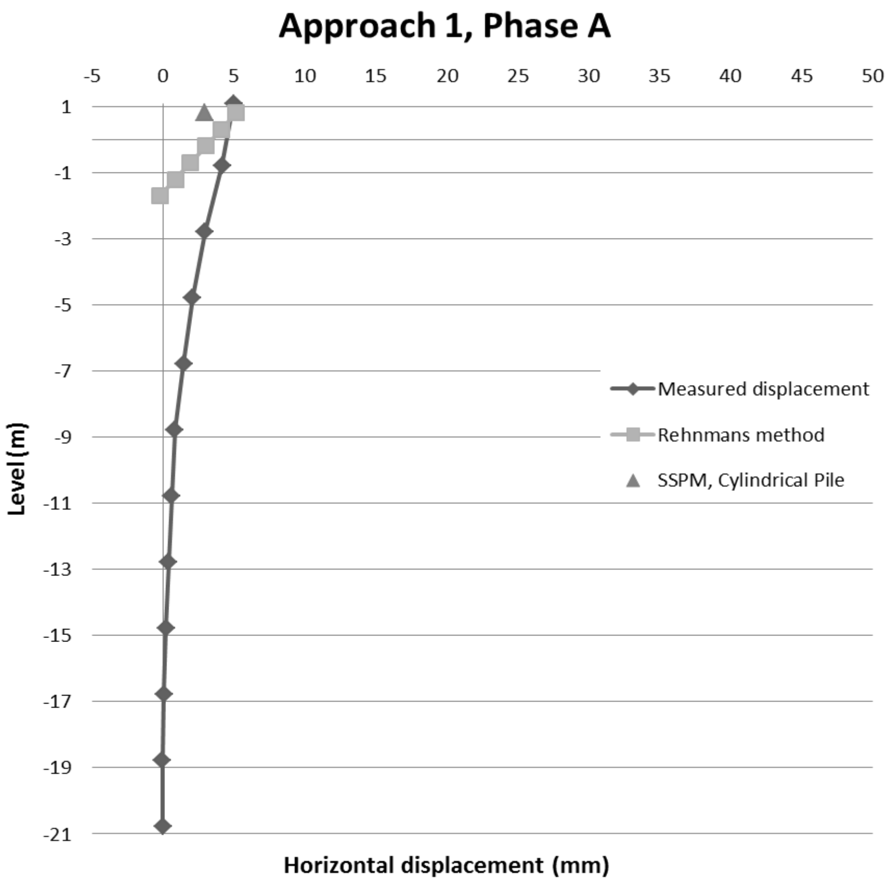

Figure 8.1 Horizontal displacements for Phase A, Approach 1.

In **Phase B**, including 26 columns, both the methods results in similar displacement at the column top level, see Figure 8.2 below. The calculation with Rehnman's method results in a maximal displacement of 7.79 mm, with a linear decrease. The displacement is zero where the depth from the column top level is equal to the distance to the inclinometer. The calculation with the SSPM for a cylindrical pile resulted in a displacement of 7.39 mm. The maximal measured displacement for Phase B is 18.6 mm and decreases with the depth as seen in Figure 8.2 below.

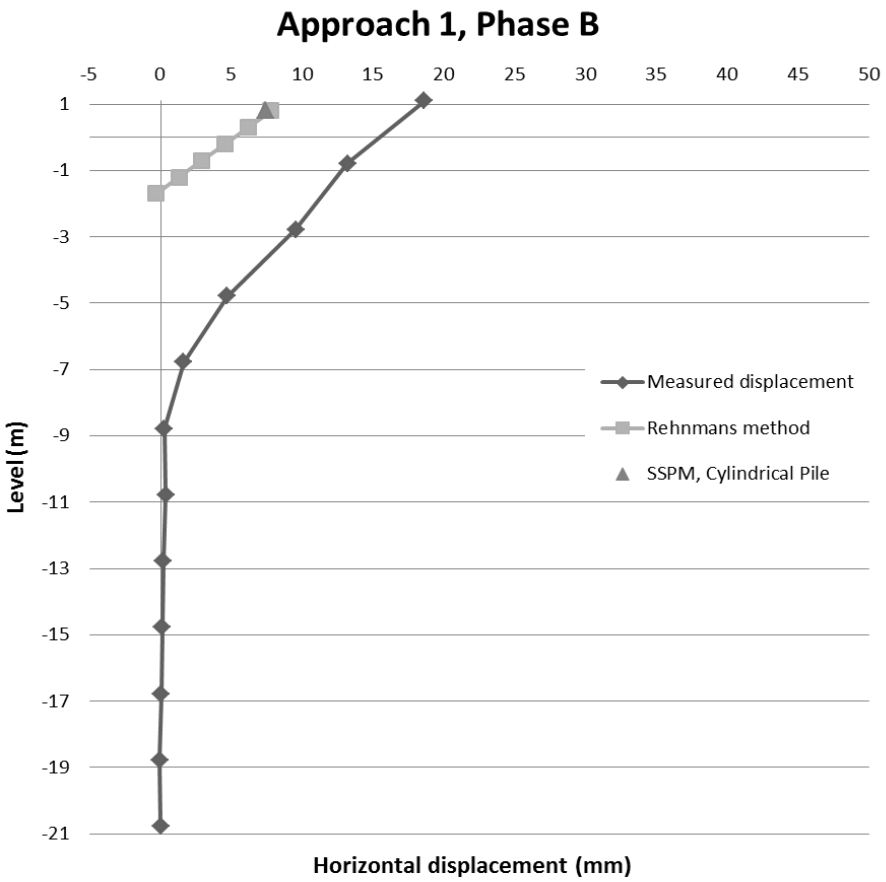

Figure 8.2 Horizontal displacements for Phase B, Approach 1.

In **Phase C**, including 44 columns, the calculated maximal displacement with Rehnman's method was 9.39 mm and decreases linearly, as mentioned for Phase A and B. The calculation with the SSPM for a cylindrical pile resulted in a displacement of 9.67 mm. The maximal measured displacement occurs at top level of the inclinometer and is 42.3 mm and decreases with the depth as seen in Figure 8.3 below.

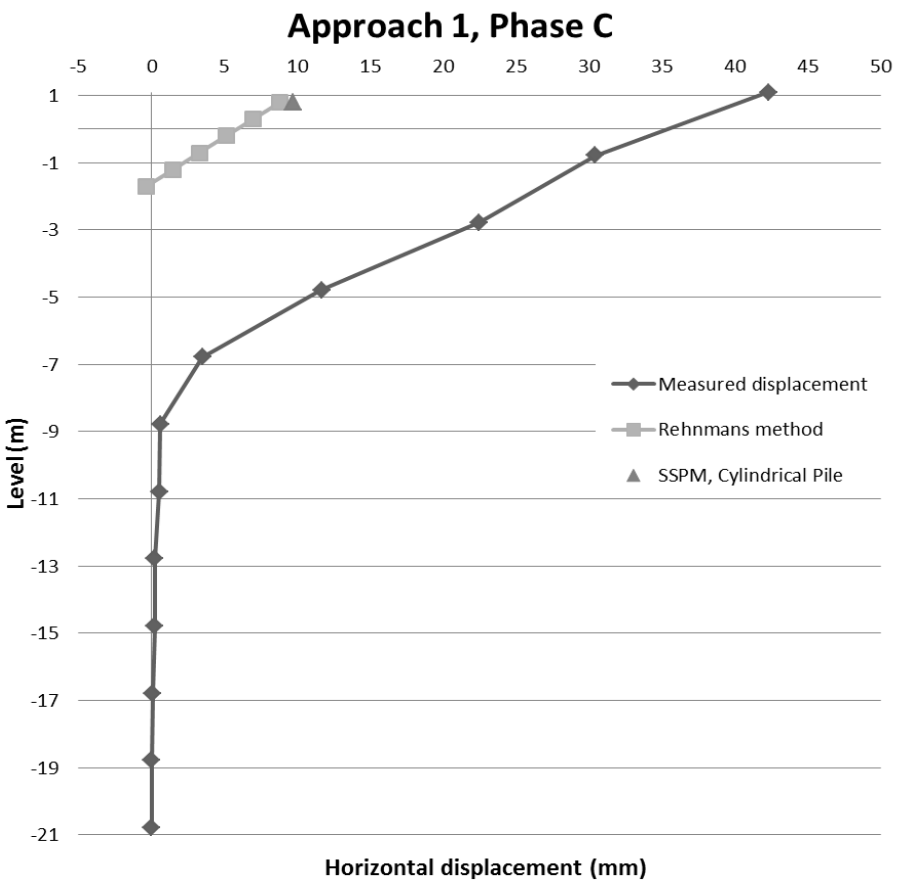

Figure 8.3 Horizontal displacements for Phase C, Approach 1.

In **Phase D**, including 85 columns, the calculation with the SSPM for a cylindrical pile resulted in a displacement of 13.64 mm at the column top level, see Figure 8.4. The calculated displacement with Rehnman's method resulted in a displacement of 10.62 mm at the column top level and decreases linearly, as mentioned for Phase A and B.

The calculations made for Phase D includes a calculation with the SSPM for a planar wall. The result of this calculation is a maximal displacement of 11.57 mm, occurring at the column top level. The displacement decreases with depth as seen in Figure 8.4.

The maximal measured displacement occurs at top level of the inclinometer and is 48.1 mm and decreases with the depth as seen in the figure below.

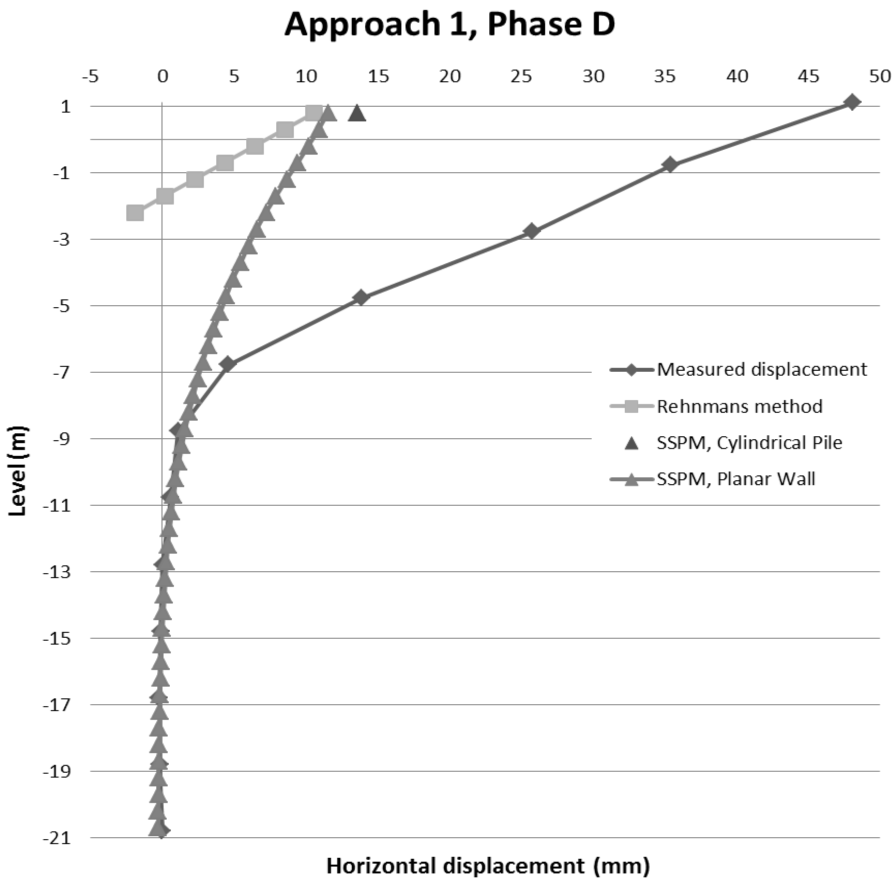

Figure 8.4 Horizontal displacements in Phase D, Approach 1.

The general impression of the results in Approach 1 is that all three methods underestimate the displacement compared to the measured displacements in section 465/300. The difference between the calculated displacements and the measured displacements increases with larger amount of LC-columns included in the different phases.

Rehnman's method agrees well with the measured displacement in Phase A but in the other phases it underestimates the displacement. According to Rehnman's method there is no displacement occurring under the column bottom level, which does not correspond to the measured displacement. The measured displacements show considerable movements under the column bottom level.

The results from the SSPM for cylindrical piles also underestimate the displacement. The results from the calculations made in the cylindrical pile case are similar to the results from Rehnman's method at the column top level.

The SSPM for a planar wall results in a displacement smaller than the measured displacement. However, the calculated displacement at depths below column bottom level corresponds rather well with the measured displacement.

8.1.2 Approach 2

From the results in Approach 1, the SSPM method for a planar wall indicated some tendency to reflect the measured displacement. Therefore, the SSPM for a planar wall applied in Phase D will be further adapted in Approach 2. Neither Rehnman's method nor the SSPM for a cylindrical pile will be further investigated. The SSPM for a planar wall, which only can be applied in Phase D, will be prioritized because the displacement can be estimated at different depths, while also the result at top column level is similar as for the other methods.

In Approach 1, the calculated displacement for a planar wall was adjusted with the ratio between the total added binder volume and the total soil volume in the column area. In Approach 2, the ratio applied in Approach 1 is replaced by the coverage ratio (a) together with the ratio between the binder volume and the column volume. The coverage ratio is introduced to describe the LC-column coverage of the total column installation area, see equation (8.5) and Figure 8.5. The ratio between the added binder volume and the column volume, which were used in Approach 1 in the SSPM for a cylindrical pile, can be seen in equation (8.1). This new approach is introduced to ease the calculations with the SSPM for a planar wall when there is possible variation of cc-distance between LC-columns in different sections. This approach also removes the requirement of knowing the LC-column area length that will have an impact on the inclinometer. For a presentation of the input data, see Appendix 6.

$$a = \frac{\pi \cdot d^2}{4 \cdot s_1 \cdot s_2} \tag{8.5}$$

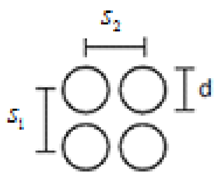

Figure 8.5 Geometry and notations for calculation of the coverage ratio of LC-columns in the column area.

The adjusted displacement (δ_x) for Phase D in Approach 2 will accordingly to the description above be calculated with equation (8.6) below, which is a modification of the SSPM for a planar wall, see equation (4.7) in Chapter 4.

$$\delta_x = \frac{w}{\pi} \cdot \left[\begin{array}{l} \tan^{-1}\left(\dfrac{z+L}{x}\right) - \tan^{-1}\left(\dfrac{z-L}{x}\right) \\ +2xz \cdot \left(\dfrac{1}{x^2+(z+L)^2} - \dfrac{1}{x^2+z^2} \right) \end{array} \right] \cdot Ratio_{Binder/Column} \cdot a \tag{8.6}$$

8.1.2.1 Results

The result for the SSPM for a planar wall in Approach 2 is a displacement slightly smaller than the calculated displacement in Approach 1. The displacement at the column top level is 9.44 mm and the displacement decreases with depth as seen in Figure 8.6 below.

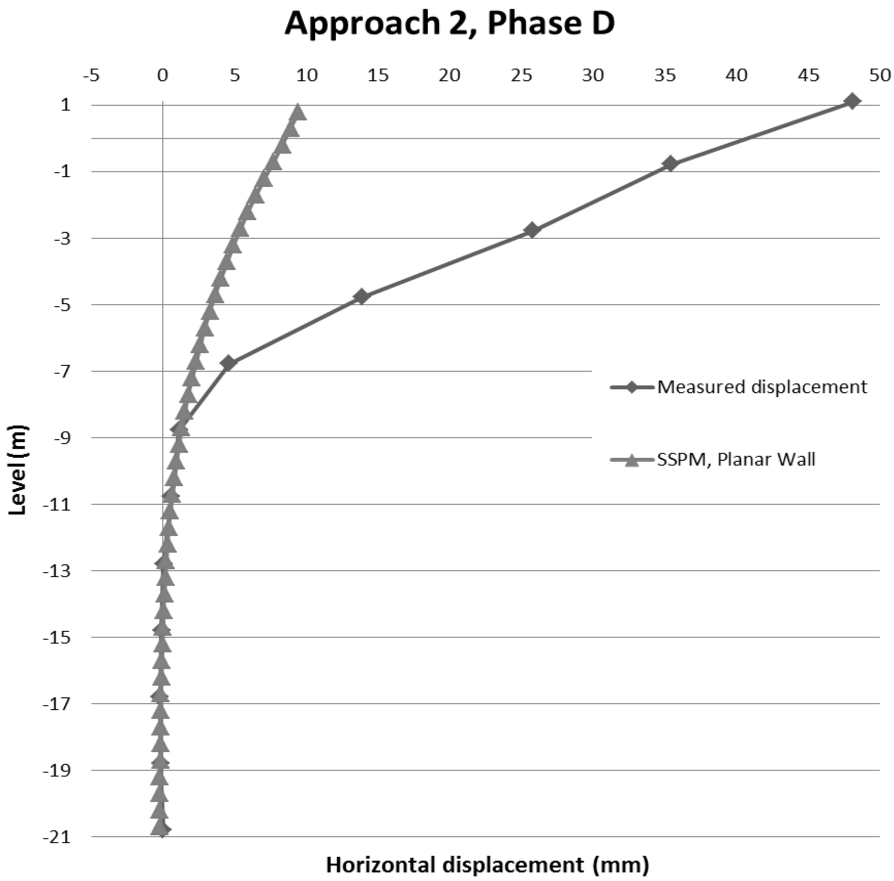

Figure 8.6 Horizontal displacements in Phase D, Approach 2.

The result of the calculated displacement in Approach 2 is very similar to the result calculated with the SSPM for a planar wall in Approach 1. This was expected since the ratio used in Approach 1 for adjusting the calculated displacement is only expressed in a different manner in Approach 2.

8.1.3 Approach 3

In Approach 2, the calculations were adjusted by taking into account the injected amount of binder in relation to the column volume and the coverage ratio of the LC-columns. In Approach 3 with the SSPM for a planar wall, an operative value for the wall thickness ($\mu_w \cdot w$) will be introduced in order to adjust and increase the calculated displacement given in Approach 2, which highly underestimates the displacement in relation to the measured displacement. This is made since the SSPM for a planar wall does not take into account the specific parameters only existing when installing LC-columns. The operative correction factor for the wall thickness (μ_w) is investigated in order to get a result which complies better with the measured displacement.

The parameters existing during installation of LC-columns which are not present when driving piles are not included in the original SSPM for a planar wall. It is not clear how much these parameters, for instance the injection pressure and chemical reactions when the binder reacts with the soil/water, affect the displacement. The impact of these parameters is in the adapted method merged into the operative value for the wall thickness. By investigating different operative correction factors and evaluate the displacement results, it is possible to establish which operative value that results in calculated displacements that complies best with the measured displacement. The adjusted displacement (δ_x) for Phase D in Approach 3 will be calculated with different operative correction factors for the wall thickness (μ_w) see equation (8.7). The input data used in Approach 3 is presented in Appendix 6.

$$\delta_x = \frac{\mu_w \cdot w}{\pi} \cdot \left[\begin{array}{l} \tan^{-1}\left(\frac{z+L}{x}\right) - \tan^{-1}\left(\frac{z-L}{x}\right) \\ + 2xz \cdot \left(\frac{1}{x^2 + (z+L)^2} - \frac{1}{x^2 + z^2} \right) \end{array} \right] \cdot Ratio_{Binder/Column} \cdot a \qquad (8.7)$$

8.1.3.1 Results

An investigation has been made to achieve operative values which results in displacements that comply well with the measured displacement in Phase D. The result indicates that by applying an operative correction factor of 4, 5 or 6 the calculated displacement is in good agreement with the measured displacement, see Figure 8.7 below. An operative correction factor of 5 resulted in the calculated displacements which best comply with the measured displacements.

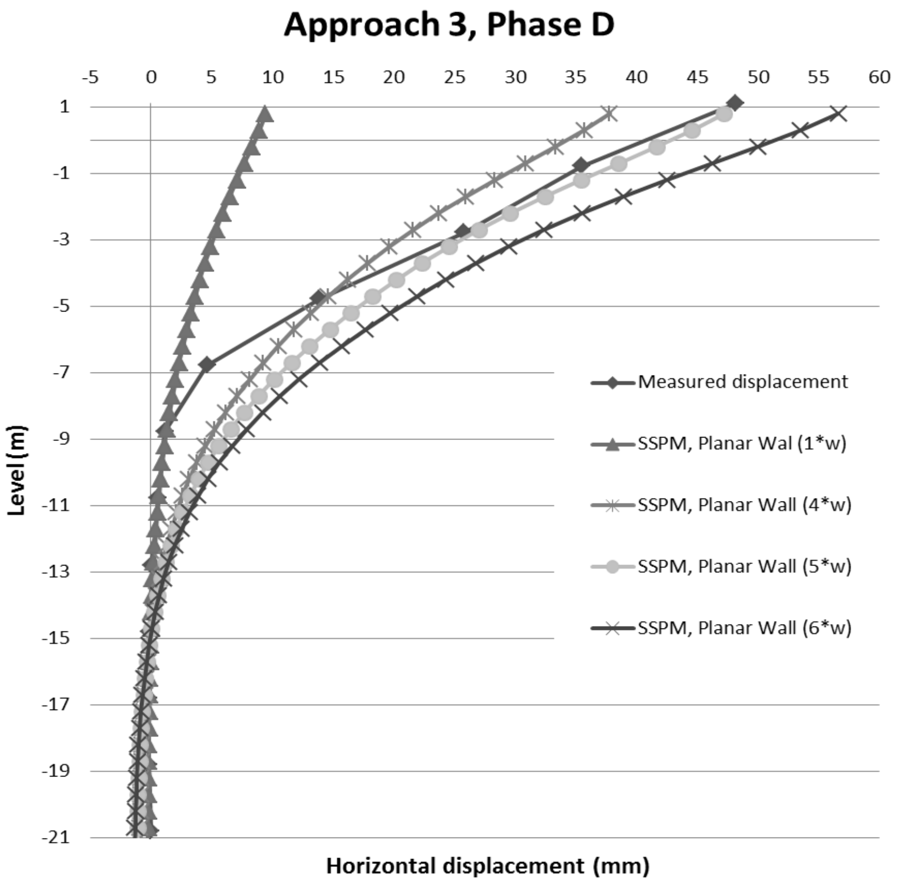

Figure 8.7 Horizontal displacements in Phase D, Approach 3.

No investigated operative value results in displacements that totally agree with the measured displacements. However, the applied operative values results in displacements that corresponds well with the measured displacements in the upper part of the columns. At larger depth, all the investigated operative values results in displacements that are overestimated compared to the measured displacements.

8.1.4 Approach 4

The results from Approach 3 showed that the adapted method overestimates the displacements on depths larger than the column length. Approach 3 will be further adapted in Approach 4 to comply better with the measured displacements on larger depths. In Approach 4 an operative column length will be introduced and an operative correction factor will be investigated to acquire a decreased effective length of the columns. The shorter effective length will decrease the calculated displacements on larger depths. The operative correction factor for the column length (μ_L) is investigated by insertion into equation (8.8). By evaluating the results, the operative column length that complies best with the measured displacements in Phase D will be obtained. The other parameters in the equation are as in Approach 3, with the operative correction factor for the wall thickness (μ_w) set to 5, see Appendix 6.

$$\delta_x = \frac{\mu_w \cdot w}{\pi} \cdot \left[\begin{array}{l} \tan^{-1}\left(\dfrac{z+\mu_L \cdot L}{x}\right) - \tan^{-1}\left(\dfrac{z-\mu_L \cdot L}{x}\right) \\ + 2xz \cdot \left(\dfrac{1}{x^2+(z+\mu_L \cdot L)^2} - \dfrac{1}{x^2+z^2}\right) \end{array} \right] \cdot Ratio_{Binder/Column} \cdot a \qquad (8.8)$$

8.1.4.1 Results

The results for the investigation of the operative correction factor for the column length indicates that values between 0.7-0.9 results in a displacements that comply best with the measured displacements, see Figure 8.8.

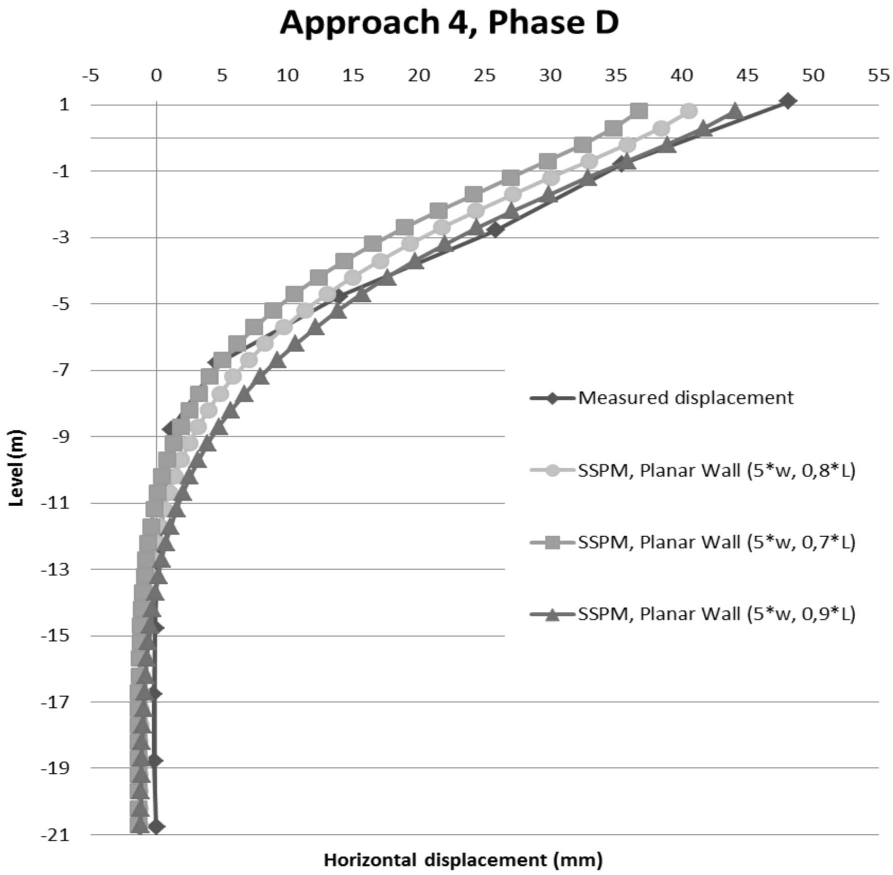

Figure 8.8 Horizontal displacements in Phase D, Approach 4.

The results shows that the operative value for the column length have a notable impact on the calculated displacement. The operative column lengths presented in Figure 8.8 comply very well with the measured displacements on depths below the column bottom level. The operative correction factor of 0.8 seems to comply best with the measured displacements. The displacements on smaller depths are also decreased by the shorter effective column length.

8.1.5 Approach 5 – Final approach

No new parameters will be introduced in Approach 5, the approach will instead be focused on combining the operative correction factors (μ_w, μ_L) to result in displacements that complies well with the measured displacements. The operative correction factor for the column length is set to a constant value of 0.8, based on the results from Figure 8.8 in Approach 4. By investigating different operative correction factors for the wall thickness, an operative value will be obtained that result in displacements that complies best with the measured displacements in Phase D. The investigation will be similar to Approach 3 but with an operative correction factor for the column length set to 0.8 included in the equation, see equation (8.9). For a complete presentation of the input data used in Approach 5, see Appendix 6.

$$\delta_x = \frac{\mu_w \cdot w}{\pi} \cdot \left[\begin{array}{l} \tan^{-1}\left(\dfrac{z + 0.8 \cdot L}{x}\right) - \tan^{-1}\left(\dfrac{z - 0.8 \cdot L}{x}\right) \\ + 2xz \cdot \left(\dfrac{1}{x^2 + (z + 0.8 \cdot L)^2} - \dfrac{1}{x^2 + z^2}\right) \end{array} \right] \cdot Ratio_{Binder/Column} \cdot a \qquad (8.9)$$

8.1.5.1 Results

The results from the investigation shows that the operative correction factor values for the wall thickness which resulted in a displacement that complied well in Approach 3, needs to be slightly increased to comply well in Approach 5. With the operative correction factor for the column length set to 0.8, the calculated displacements comply best with the measured displacements when the operative correction factor for the wall thickness is between 5-6, see Figure 8.9.

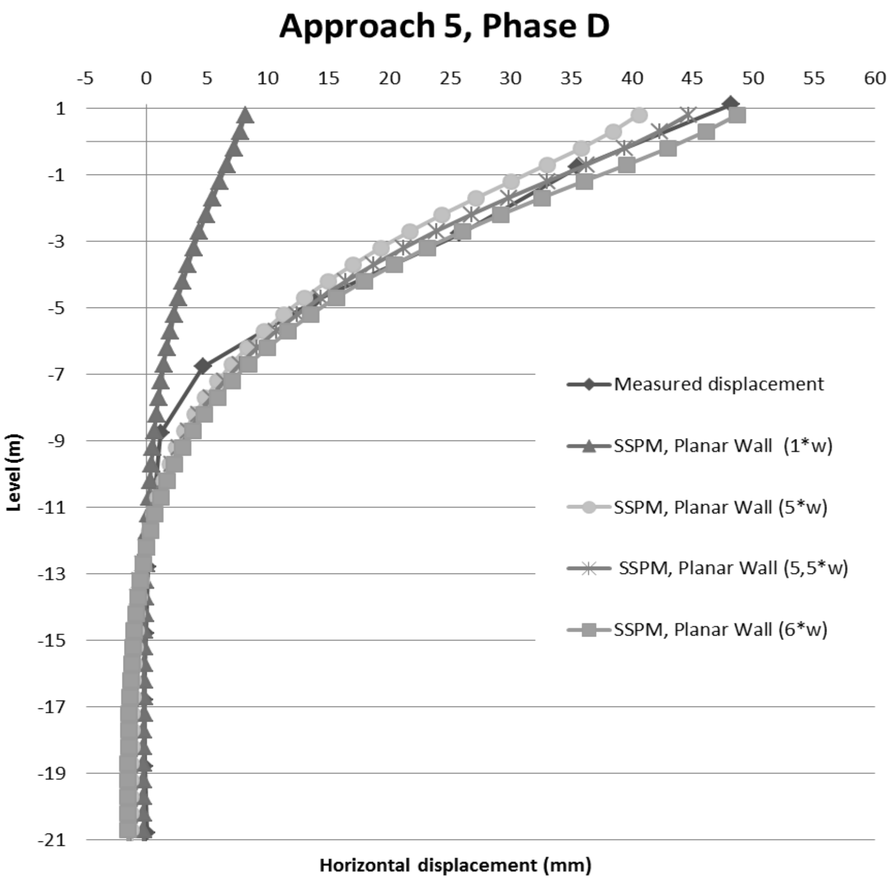

Figure 8.9 Horizontal displacement for Approach 5.

The result from Approach 5 indicates that the calculated displacements comply very well with the measured displacements when the appropriate operative values are applied. It agrees well both in the top soil and at depths below the column bottom level.

8.2 Test in other sections

The adjustments made to the original SSPM for a planar wall in Section 8.1 are made to adapt the original piling method for LC-columns. An evaluation is carried out to investigate if the adapted method in the final approach in Section 8.1.5 might correspond to the displacement occurring when installing LC-columns. The adapted method is therefore tested at other sections. In this section, the adapted SSPM for a planar wall is tested at two other sections included in the project BanaVäg I Väst.

The investigated sections are section 465/535 and section 465/750 where displacement was measured with inclinometers during the installation of LC-columns under the service road.

8.2.1 Section 465/535

Section 465/535 is located approximately 235 m north of section 465/300. Approximately 15 m north of the inclinometer the LC-columns are replaced by wooden piles. The section has a varying rectangular column pattern where the cc distance and the amount of columns in each row vary for the service road. In the north part of the LC-column area the cc distance is 1 m for all columns, while in the south part of the area the cc distances are 1 m transversal and 1.4 m longitudinal. The amount of columns in each row varies in the LC-column area; the north part has 8 columns in each row while the south part has 7 columns in each row. Due to these variations, average values for the coverage ratio and the wall thickness are used in the calculations, see Appendix 7.

The LC-column length in section 465/535 is 7.5 m and the column diameter is 0.6 m. The columns are installed with the mixing tool PB600 with an injection pressure of 550 kPa. The mean amount of binder in each column is 30 kg/m and it consists of equal parts of lime and cement.

When calculating the displacement in section 465/535 the equation (8.9) is applied and the operative correction factor for the wall thickness varies between 4 and 6. The calculated displacement will be compared to the measured displacement from the inclinometer located 5 m west of the center of the service road.

8.2.1.1 Results

The resulting displacements of the calculations are seen in Figure 8.10, together with the measured displacements in section 465/535. The displacements calculated with an operative correction factor for the wall thickness of 4 complies fairly well from the bottom level up to the level -4 m. The displacements calculated with the higher operative correction factor for the wall thickness results in larger displacements than the measured displacement at all levels.

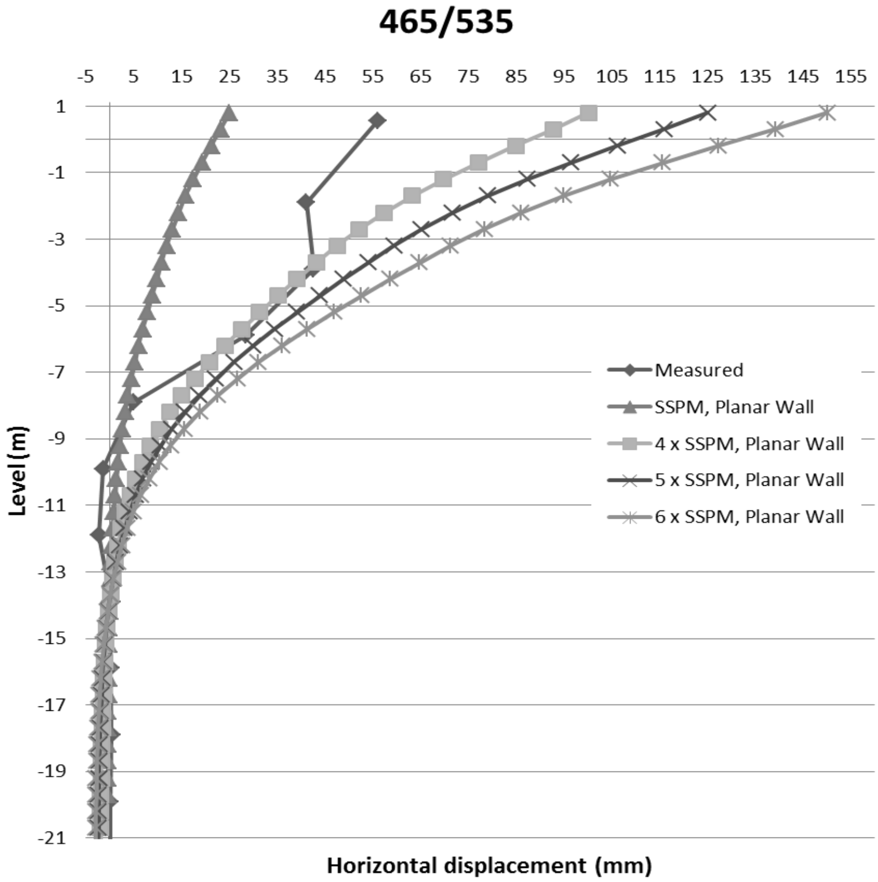

Figure 8.10 Measured and calculated horizontal displacements in section 465/535.

8.2.2 Section 465/750

Section 465/750 is located approximately 450 m north of section 465/300. The LC-columns in section 465/750 has a constant rectangular column pattern, the cc distance is 1 m transversal and 1.4 m longitudinal. Every column row consists of 7 LC-columns which results in a total wall width of 6.6 m.

The column length in the LC-column area is 5.8 m and the installation is made with the mixing tool PB600, which results in a column diameter of 0.6 m. The binder added consisted of equal parts of lime and cement and the amount added was 30 kg/m. Further, the columns were installed with an injection pressure of 550 kPa.

When calculating the displacement in section 465/750 the equation (8.9) is applied and the operative correction factor for the wall thickness varies between 4 and 6. The calculated displacements are compared with the measured displacements from the inclinometer located 6.65 m west of the center of the service road. A presentation of the input data for the calculations can be seen in Appendix 7.

8.2.2.1 Results

The calculated displacements are presented in Figure 8.11 below, together with the measured displacements in section 465/750. The calculated displacements comply very well with the measured displacements. The measured displacements lies between the calculated displacements lines, where the operative correction factor for the wall thickness is set to 4 and 5. The calculation made with an operative correction factor of 6 results in overestimated displacements.

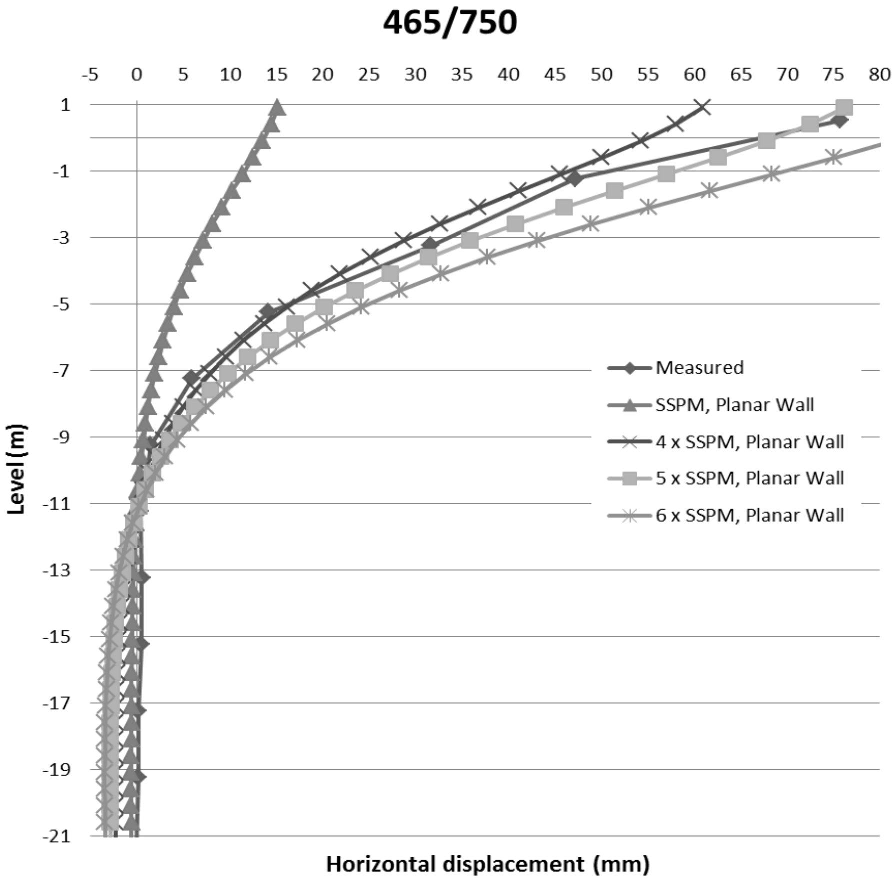

Figure 8.11 Measured and calculated horizontal displacements in section 465/750.

8.3 Discussion

When the investigation started with the first approach, the idea was to detect a possible connection between the displacements and the amount of installed columns in the different phases. The results from Phase A, B and C, however, were so inconsistent that it was not possible to discern any relationship between displacements and amount of installed columns. This is most likely because the total displacement due to the installation of a column does not happen instantly and is instead gradual. This means that the measured displacements, in for instance Phase B, included delayed displacements from the columns already installed in Phase A.

Uncertainties in the input parameters used in the calculations are quite few. The largest uncertainty is the assumed density for the binder, the assumed density of 1800 kg/m^3 is not documented during the installation; the assumed density is valid for LC-mortar. When mean values were used, the spread of values are small and therefore the mean value is representative.

An aspect to consider when estimating displacements is the installation order. In section 465/300 the LC-columns were installed from east to west, and since the inclinometer was located on the western side, the LC-columns are installed in the direction towards the inclinometer. This probably contributes to unequal displacement on both sides of the service road. When a column is installed next to another column it seems reasonable to suggest that the older column partly prevents displacement in that direction. Hence, the displacement is probably larger on the western side where the inclinometer is located. To get a better understanding about the displacement it would be favourable to have an inclinometer on both sides of the service road. In this way it would be possible to see if the displacement is equal on both sides or not.

When it comes to uncertainties regarding the measured displacement, it is notable that the measured displacement in section 465/535 has a different pattern in the top 5 m compared to the other investigated sections. This odd pattern could be due to several reasons such as disturbances generated by machines or non-homogenous soil properties. If the displacement in the top 5 m would have continued in the same pattern as the displacement in the other sections, it would comply well with the calculated displacements presented in Figure 8.10. Another aspect to consider when evaluating the measured displacement in section 465/535 is the wooden piles installed approximately 15 m north of the inclinometer. How and if these piles affect the displacement is difficult to assess.

The adapted SSPM for a planar wall used in Approach 5 complies well with the measured displacements in the different sections tested, with an operative correction factor for the wall thickness of 4-6. However, it would be interesting to investigate how the adapted method complies when the installation- and column properties differ from the studied areas in this thesis. The LC-columns in the three sections studied had an injection pressure of 550 kPa and the binder consisted of equal parts of lime and cement. In the adapted method in the final approach, the impact from each parameter is included and accounted for in the operative value. A better approach would be to divide the operative value for the wall thickness into a few more specific variables.

9 Calculated Displacement with PLAXIS

This chapter presents a numerical analysis made in PLAXIS 2D - version 11 for estimating horizontal displacement. The chosen model, input parameters and calculation phases will be described together with a presentation of the results. The model represents section 465/300 and the LC-columns are seen as a planar wall as in the SSPM calculations made in Chapter 8. The results from PLAXIS will be compared to the measured displacement in section 465/300 for Phase D. Further, a parameter study will be performed to investigate the sensitivity of chosen input parameters.

9.1 Model

The model made for this analysis is simplified and is made with a linear elastic model, which is based on Hooke's law for isotropic linear elastic behaviour. This is the simplest material model in PLAXIS and it is a good start for a first analysis of the problem. Stress states are not limited in any way and the model therefore shows infinite strength. The linear elastic model involves two basic elastic parameters, Young's modulus and Poisson's ratio (Brinkgreve, Swolfs, & Engin, 2011).

An undrained total stress analysis with undrained parameters (Undrained C) is applied to simulate undrained behaviour. The stiffness is modelled with undrained Young's modulus (E_u) and undrained Poisson's ratio (ν_u).

In the linear elastic theory, the relation between stress and strain can be divided into a part representing volume change and another representing change of shape. According to Hooke's law (equation (9.1) the use of Poisson's ratio $\nu \approx 0.5$ in undrained analysis implies a bulk modulus (K) approaching infinity. This means that there will be no volume change.

$$K = \frac{E}{3 \cdot (1 - 2 \cdot \nu)} \rightarrow \infty \quad with \quad \nu \rightarrow 0.5 \tag{9.1}$$

However the part representing the change of shape still has an impact. According to equation (9.2), the shear modulus which governs the change of shape is calculated as a function of Young's modulus and Poisson's ratio (Brinkgreve, Swolfs, & Engin, 2011).

$$G = \frac{E}{2 \cdot (1 + \nu)} \rightarrow \frac{E}{3} \quad with \quad \nu \rightarrow 0.5 \tag{9.2}$$

A plain strain model with 15-node elements is used in the analysis, which is two-dimensional and has no displacements or strains in the z-direction. The model is built as seen in Figure 9.1 below, where the loads for the road and railway can be seen together with the topography of section 465/300. The levels for the ground surface, the river and the bedrock are estimated from the geotechnical investigations made in the area. A simple stratigraphy is made where the soil in the section is seen as one homogeneous clay layer with geotechnical properties estimated from the geotechnical investigations made. The LC-columns are represented as a part in the clay symbolising a LC-column wall, which later in the calculations is applied with a volume strain to simulate the volume expansion due to the installation of LC-columns.

A ground water level is applied to the model in order to simulate a water level in Göta Älv and boundary conditions are set. There is full fixity boundaries set for the base line together with the line symbolising the bedrock, while the vertical side is set to horizontal fixity. The model is made for a quite large area so that the boundary conditions do not affect the results. It is also of interest to see if and how the railway, the road, the bedrock and the river affect the displacement results. A mesh is generated in the model and a refined mesh has been made around the LC-wall and around the line symbolising the position of the inclinometer.

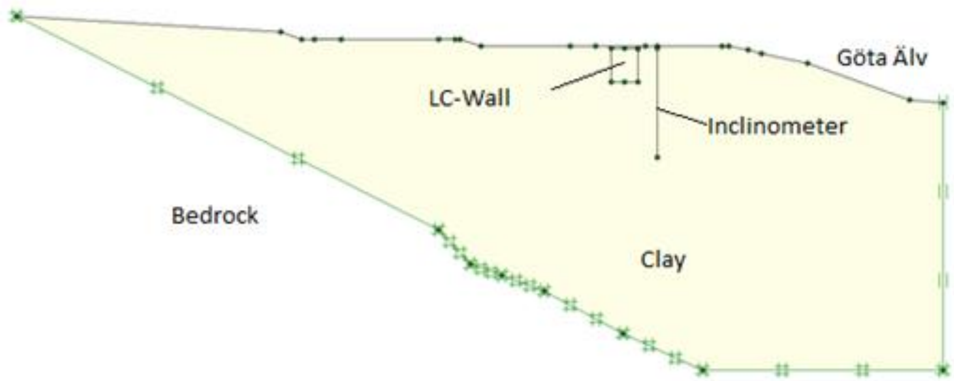

Figure 9.1 Overview of the PLAXIS model with boundaries, LC-column wall and inclinometer position.

9.1.1 Input parameters

The input parameters needed for the analysis are the unit weight of the clay, the undrained shear strength of the clay, the undrained Young's modulus (E_u) the undrained Poisson's ratio (ν_u) together with the applied volume strain. Further, the loads for the road and railway are set to known standards. All the values set for the input parameters can be seen in Table 9.1 below.

Table 9.1 Input data for the PLAXIS model.

Input data	
E_u	2000 kN/m^2
$E_{u,inc}$	250 kN/m^2/m
γ_{unsat}	15.5 kN/m^2
ν_u	0.490
Volume strain	0.74 %

The undrained shear strength and unit weight have been estimated from the geotechnical investigations made, see Appendix 1. The unsaturated unit weight for the clay (γ_{unsat}) is estimated to 15.5 kN/m^2 in the top 10 m of the clay layer; this unit weight is simplified and applied to the entire clay layer. The undrained shear strength is estimated to 8 kPa at surface level with an increase of 1 kPa/m, which is slightly simplified.

The undrained Young's modulus can be estimated on basis of soil type and undrained shear strength, see Appendix 8 (Trafikverket, 2011). The incremental modulus ($E_{u,inc}$) is estimated according to Appendix 8 and the modulus starts increasing directly from the ground surface where the LC-Wall is located. According to Hooke's law, the shear modulus (G) and the oedometer modulus ($E_{u,oed}$) is calculated from known relationships with Young's modulus.

The increase in lateral stress coefficient (K_0) should be 0.89 according to existing empirical relations and the calculations made can be seen in Appendix 8.

Poisson's ratio is chosen in order to symbolise an incompressible material which is true for clay when analysing a short term scenario. Poisson's ratio is set to 0.490 since the undrained Poisson's ratio is used and a value close to 0.5 should be selected in the analysis. A value of exactly 0.5 would lead to singularity of the stiffness matrix and numerical problems. When selecting Poisson's ratio for the elastic model, PLAXIS should generate realistic ratios of K_0, which will be controlled (Brinkgreve, Swolfs, & Engin, 2011).

The volume strain applied on the LC-wall region is set to 0.74 %, calculated in the same way as in Approach 2 in Chapter 8 where the coverage ratio, a, together with the binder/column ratio where introduced, see Appendix 8.

9.1.2 Calculation phases

Different calculation phases are made in order to specify the different construction stages and the installation of the LC-columns. The calculation performed in this analysis consists of four phases and all phases are defined as plastic calculations and uses staged construction as load input.

The **initial phase** describes the initial conditions including the initial geometry configuration and the initial effective stress state. Also the initial groundwater conditions are established in this phase by applying a ground water level to the model. In order to generate initial stresses the K0 procedure is normally used. This procedure can however only be used for horizontally layered geometries with a horizontal ground surface. Due to the geometry of our model with a non-horizontal ground surface, gravity loading is used instead in order to generate the initial stresses. In the initial phase neither the loads nor the volume strain are applied. The initial stresses are then calculated with a finite element calculation by applying the soil weight. The initial stresses were then controlled and verified before the other calculation phases were made. Once the initial stresses have been set up, the displacements were reset before the start of all following calculations performed in order to remove the developed displacement due to the initial stress generation (Brinkgreve, Swolfs, & Engin, 2011).

The **second phase** is a nil step which is introduced in order to solve large out-of-balance forces occurring when using gravity loading. No additional loading is applied and equilibrium is restored.

In the **third phase** the volume strain on the part in the clay symbolising the LC-column wall is introduced.

9.2 Results

The result from the analysis made in PLAXIS can be seen in Figure 9.2 below, showing the horizontal displacements and displacement pattern after the volume strain is applied. From the results it can be seen that neither the Göta Älv nor the bedrock have any significant impact on the horizontal displacement occurring. A more comprehensive picture of the displacement results can be seen in Appendix 9.

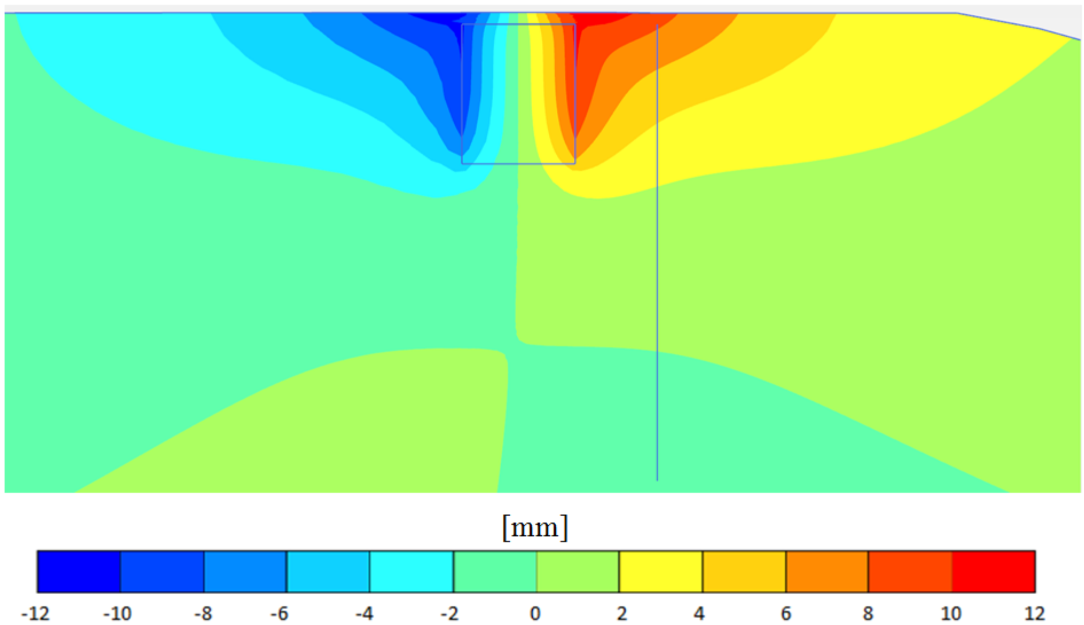

Figure 9.2 View of the horizontal displacements that occur when the LC-column wall is applied with a volume strain of 0.74 % in PLAXIS.

The horizontal displacements that occur at the position of the inclinometer in the model can be seen in Figure 9.3 below, together with the measured displacement in section 465/300 for Phase D. The installation phase where the LC-columns are seen as a wall and applied with a volume strain, 0.74 %, results in underestimated horizontal displacements compared to the measured displacement. The displacement at the column top level, +0.8 m, is 8.3 mm and decreases with the depth as seen in the figure below.

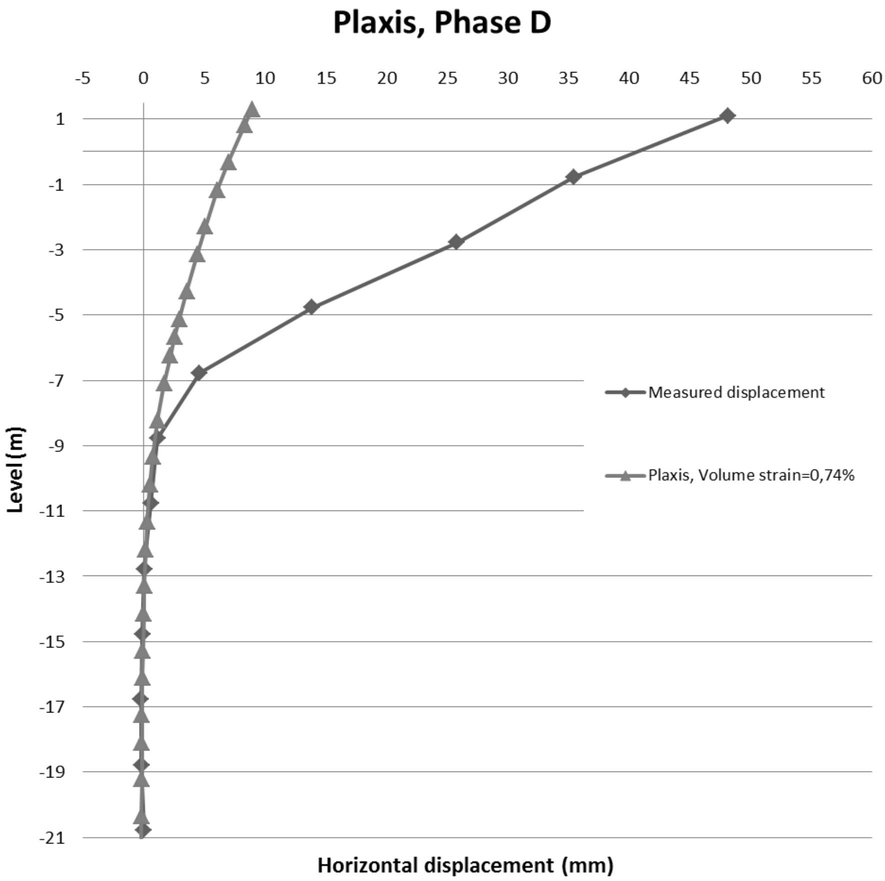

Figure 9.3 *The horizontal displacements calculated in PLAXIS compared to the measured displacement in section 465/300.*

9.2.1 Parameter study

In this section a parameter study is carried out where Young's modulus, Poisson's ratio and the applied volume strain is varied in the PLAXIS analysis. The results from all the analyses will be investigated at different levels at the position of the inclinometer.

The horizontal displacement results from the analysis made where Poisson's ratio is varied and the Young's modulus is 2000 kN/m^2 are presented in Table 9.2 below. The incremental modulus (E_{inc}) has a constant value of 250 kN/m^2/m. Poisson's ratio is set to a decreased value of 0.47 compared to the original input value of 0.490. The decreased Poisson's ratio has small impact on the horizontal displacement in this analysis. The horizontal displacement decreases with approximately 0.1 mm at the top level of the columns and 0.002 mm at the bottom level of the columns.

Table 9.2 Calculated horizontal displacement at different levels at the inclinometer position when Poisson's ratio is varied and Young's modulus is constant.

		Horizontal displacement (mm) at the inclinometer position		
	ν_u	Column top level	Column mid-level	Column bottom level
$E_u = 2000$ (kN/m^2)	0.490	8.30	5,08	2.68
	0.470	8,18	5.01	2.66

The variation of Young's modulus with a constant Poisson's ratio of 0.49 resulted in horizontal displacements which can be seen in Table 9.3 below. Neither the increased nor the decreased Young's modulus resulted in any significant difference in horizontal displacement, compared to the original analysis where E_u =2000 kN/m^2. Moreover, it should be noticed that the displacement increases at the column mid-level and bottom level in the case where E_u =4000 kN/m^2. This is explained by the use of a constant value of 250 kN/m^2/m for the incremental modulus (E_{inc}), in all of the different analyses with varied Young's modulus.

Table 9.3 Calculated horizontal displacement at different levels at the inclinometer position when Young's modulus is varied.

		Horizontal displacement (mm) at the inclinometer position		
	E_u	Column top level	Column mid-level	Column bottom level
$\nu_u = 0.49$ (-)	1000	8.36	5.01	2.49
	2000	8.30	5,08	2.68
	4000	8.29	5.17	2.88

The volume strain used in the original analysis was set to 0.74 %, which resulted in underestimated horizontal displacement compared to the measured displacement in section 465/300. Therefore it is of interest to investigate how an increased volume strain affects the results. The volume strain is varied and is set to 3 %, 4 % and 5 % in the original model and the results are presented in Figure 9.4 below. By applying an increased volume strain of 4 %, the calculated displacement complies rather well with the measured displacement. A volume strain of 3 % underestimates the displacement while a volume strain of 5 % overestimates the displacement. All of the tested volume strains results in overestimated displacement around bottom level of the LC-column wall and a few meters downwards.

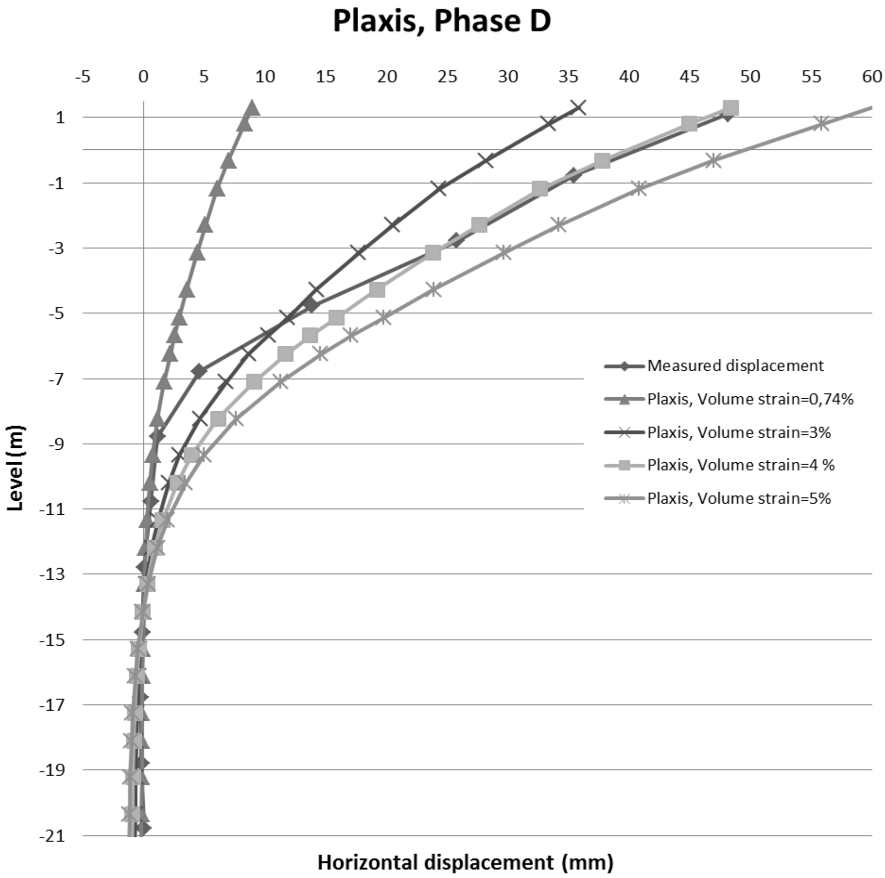

Figure 9.4 Horizontal displacement at the inclinometer position when the volume strain is varied compared to the measured displacement in section 465/300.

9.3 Discussion

The model made in PLAXIS is as mentioned a simplification of the reality and the chosen model should be analysed with an understanding of how the model actually works. The undrained linear elastic model does however reflect the main behaviour regarding how the deformation occurs. It should still be investigated further if a more complex model would lead to better accuracy and description of how the soil behave in the reality. The soil is not linear elastic in reality but it is a good start in order to investigate the deformation occurring when applying a volume strain to the LC-column wall.

The parameter study made in the model shows that the varied Young’s modulus and Poisson’s ratio has no significant impact on the deformation. This can be described by the homogeneous clay layer in the model and even if Young’s modulus and Poisson’s ratio are increased or decreased, the relation in the soil will remain the same. The undrained linear elastic model could therefore be seen as almost independent of the chosen material properties and only the geometry of the model has big influence on the deformation. This is due to the fact that the bulk modulus (K) is very large in an undrained analysis which leads to insignificant volume change according to Hooke's law. There are though change of shape occurring in an undrained linear elastic analysis and it depends on the shear modulus (G), which leads to deformation when the volume strain is applied to the LC-column wall. As seen in equation (9.2), an increase of Young's modulus will only increase the shear modulus by one third of the original value which explains the results in the parameter study.

The volumetric strain applied was also investigated in the parameter study and has big impact on the deformation. This was expected since more added material should lead to more soil displacement. The results from the parameter study where the volumetric strain was investigated were very similar to the results from the calculations made in Approach 3 for the SSPM for a planar wall.

In Approach 3, where the operative values for the wall thickness in the SSPM were investigated, an operative correction factor equal to 5 for the wall thickness results to comply well with the measured displacements. An operative correction factor of 5 is approximately equal to applying a volume strain of 4 % on the LC-column wall in the PLAXIS analysis and the resulting displacement are almost equal for both the SSPM and the PLAXIS analysis, see Figure 9.5 below.

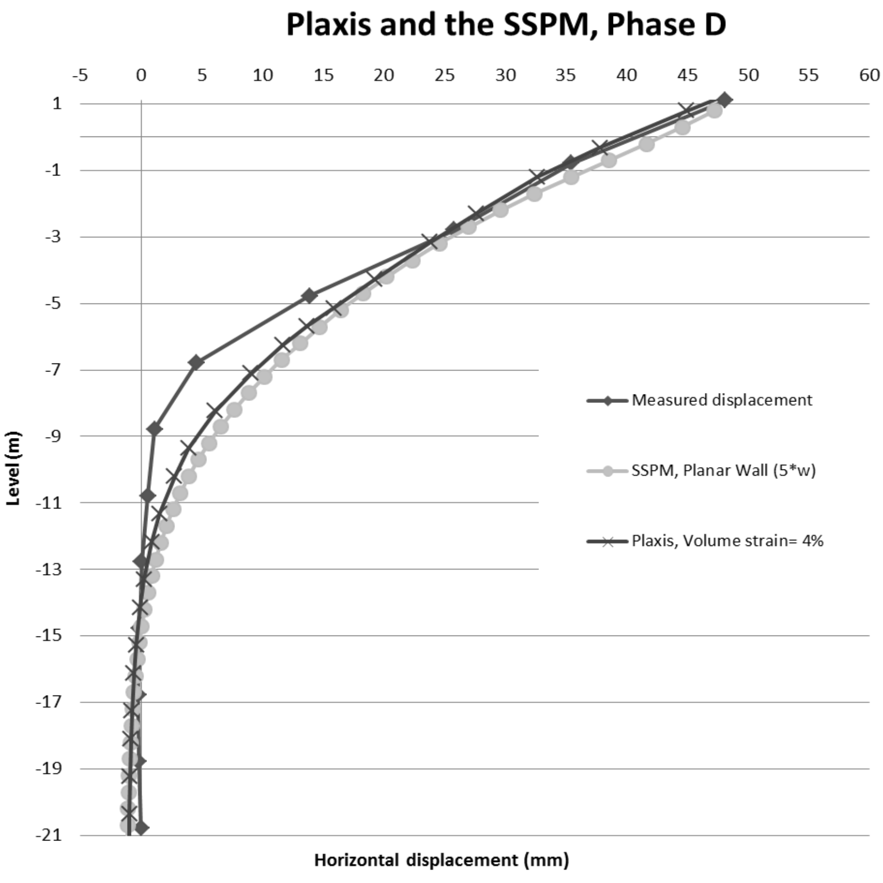

Figure 9.5 Results from PLAXIS compared to results with the SSPM for a planar wall in Approach 3.

As seen in the figure above, the calculated displacements are almost equal and this can be explained by the fact that the SSPM as well as the Plaxis analysis are based on the linear elastic theory. Furthermore, the SSPM for a planar wall only considers the geometrical relations for the LC-columns and neither soil properties nor the topography in the area are considered in the calculations. From the PLAXIS analysis it could be concluded that neither the river nor the bedrock had any significant impacts on the horizontal displacement occurring and due to this fact, the results for PLAXIS and the SSPM are practically equal. If the soil conditions, in the part where the LC-columns are installed, had been less homogenous and the topography less symmetric together with possible impact from the bedrock/river, the results would probably have been different in PLAXIS. The PLAXIS analysis is therefore good to implement in order to investigate if and how the topography, river, bedrock and other possible factors have any impact.

In the final approach of the SSPM for a planar wall, it was concluded that an operative correction factor of 0.8 for the column length/the wall height better described the displacement that occurred. This has not been investigated in PLAXIS but would probably lead to further improved results where the calculated displacement complies better with the measured displacement.

10 Conclusions

Rehnman's method, the SSPM for a cylindrical pile and the SSPM for a planar wall have been investigated in this thesis. Only the SSPM for a planar wall was studied in detail and the investigation indicated that it can be adapted for estimating the soil displacements when installing LC-columns. The initial investigation indicated that Rehnman's method and the SSPM for a cylindrical pile were less suitable.

The SSPM for a planar wall can be adapted for LC-columns by addition of two operative correction factors, one for the wall thickness and another for the column length. In the sections studied in this thesis an operative correction factor for the wall thickness of 4-6 and an operative correction factor for the column length of 0.8 give results that comply very well with the measured displacements. This procedure must however be further investigated at more locations where the geotechnical properties are different from those in the section 465/300. Furthermore, investigations into the effect of chemical reactions and injection pressure on the horizontal soil displacements need to be conducted.

Since the SSPM for a planar wall is only valid for homogenous soil conditions and flat ground, it is recommended to do a numerical analysis with a FEM-software like PLAXIS. The analysis will give an understanding about the external influences, for instance the bedrock or a slope, which may have impact on the displacement.

11 Proposal for further research

A proposal for further research is to investigate how each of the parameters included in the operative value for the wall thickness affects the horizontal displacement. Further, the adapted SSPM for a planar wall should be tested on more sections to get an understanding about how different conditions affect the displacement. For instance how different installation orders and complex installation patterns should be handled. It should also be investigated when the displacement occur; how much occurs instantly and how much occurs during the hardening process of the LC-columns.

It should further be investigated if a more detailed model in PLAXIS is necessary, by comparing the result from a detailed model with a simpler model.

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Appendices

APPENDIX 1: GEOTECHNICAL PROPERTIES

APPENDIX 2: LC-COLUMN PROPERTIES AND INSTALLATION PATTERN

APPENDIX 3: APPROACH 1, INPUT DATA FOR REHNMAN'S METHOD

APPENDIX 4: APPROACH 1, INPUT DATA FOR SSPM, CYLINDRICAL PILE

APPENDIX 5: APPROACH 1, INPUT DATA FOR SSPM, PLANAR WALL

APPENDIX 6: APPROACH 2 - 5, INPUT DATA

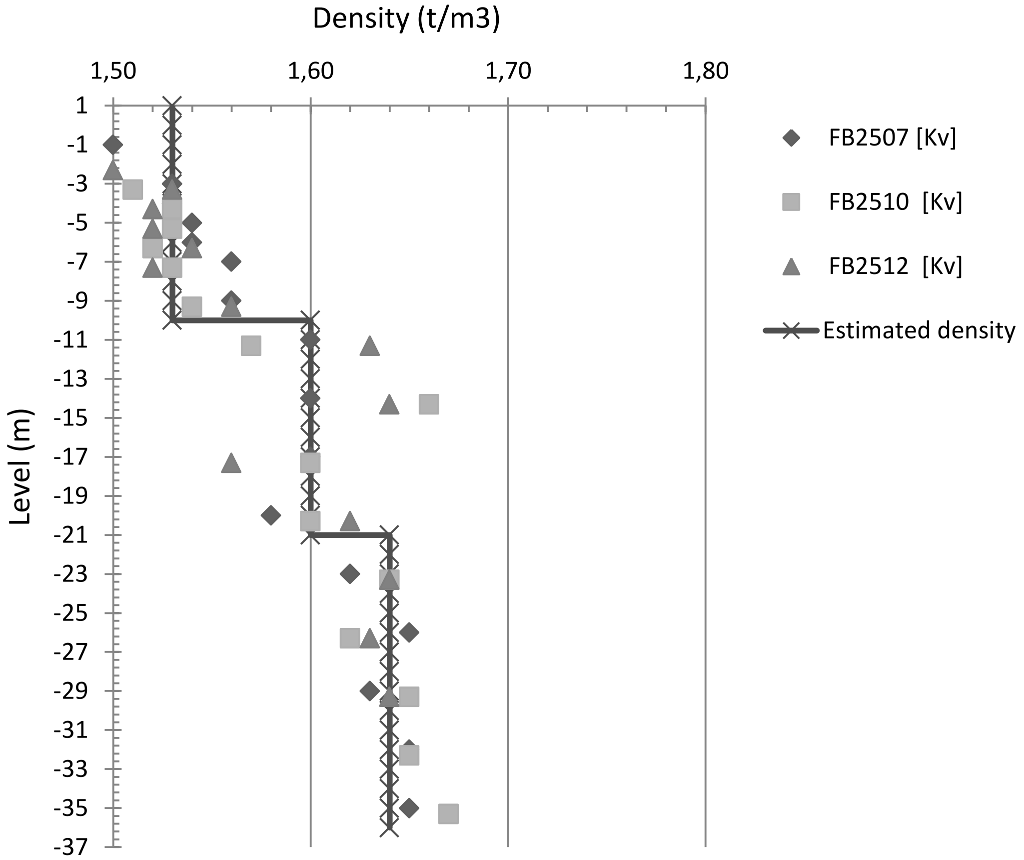
APPENDIX 7: SECTION 465/535 & 465/750, INPUT DATA

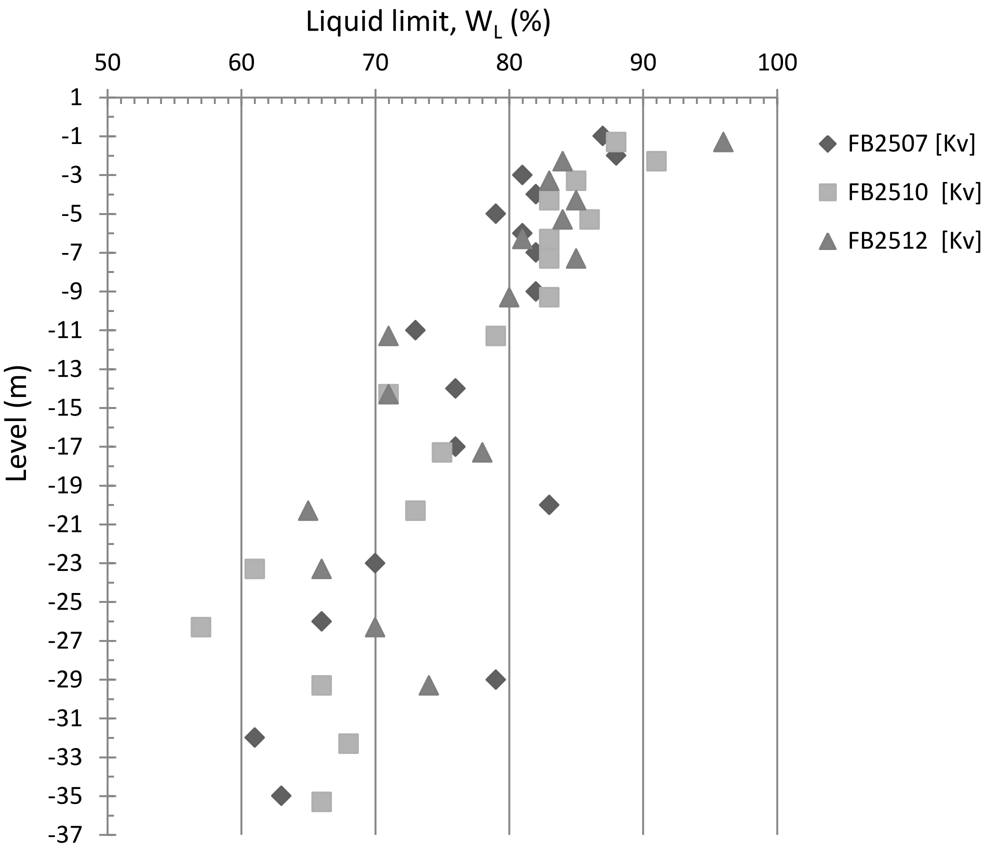
APPENDIX 8: PLAXIS, INPUT DATA

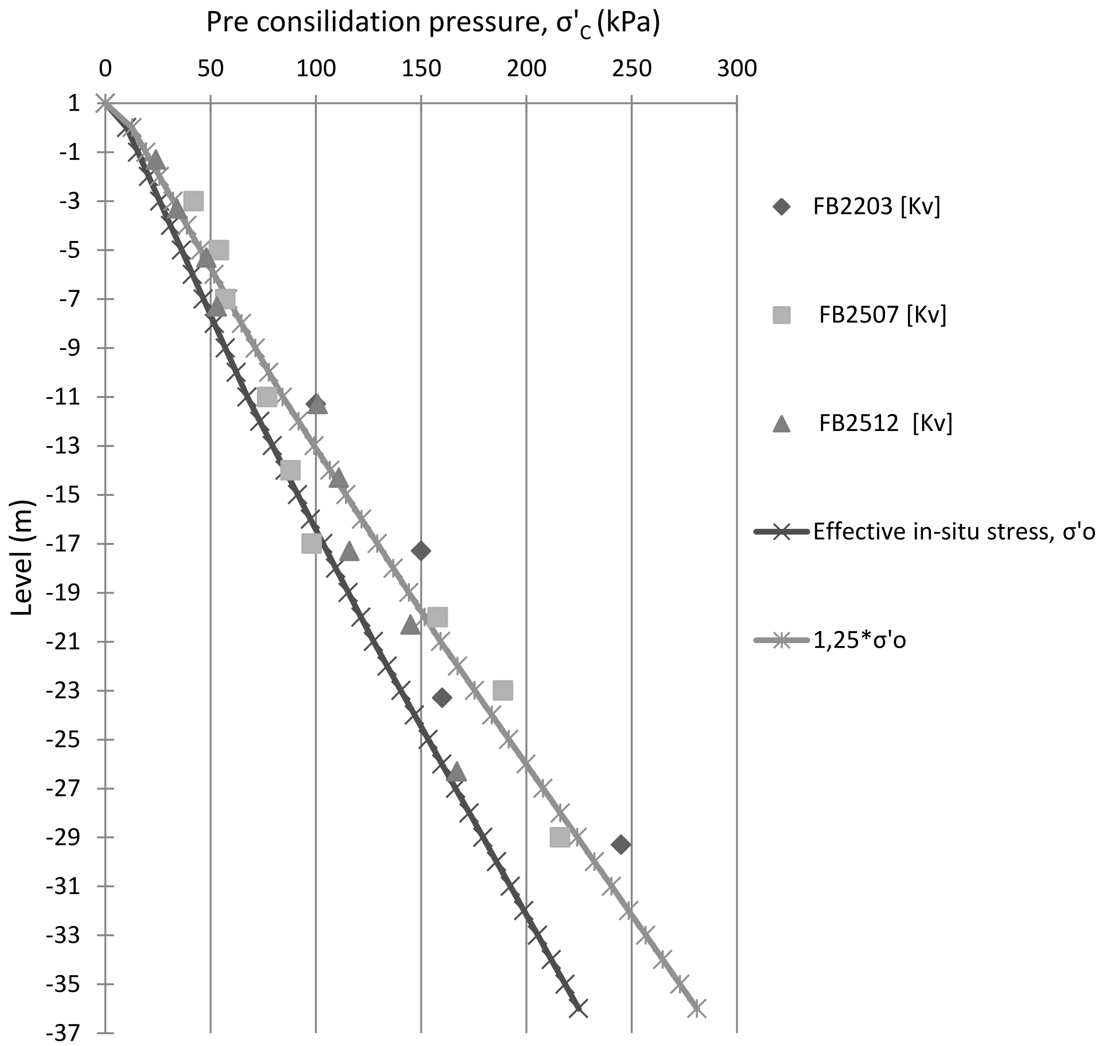
APPENDIX 9: PLAXIS, DISPLACEMENT RESULTS

APPENDIX 10: DRAWINGS

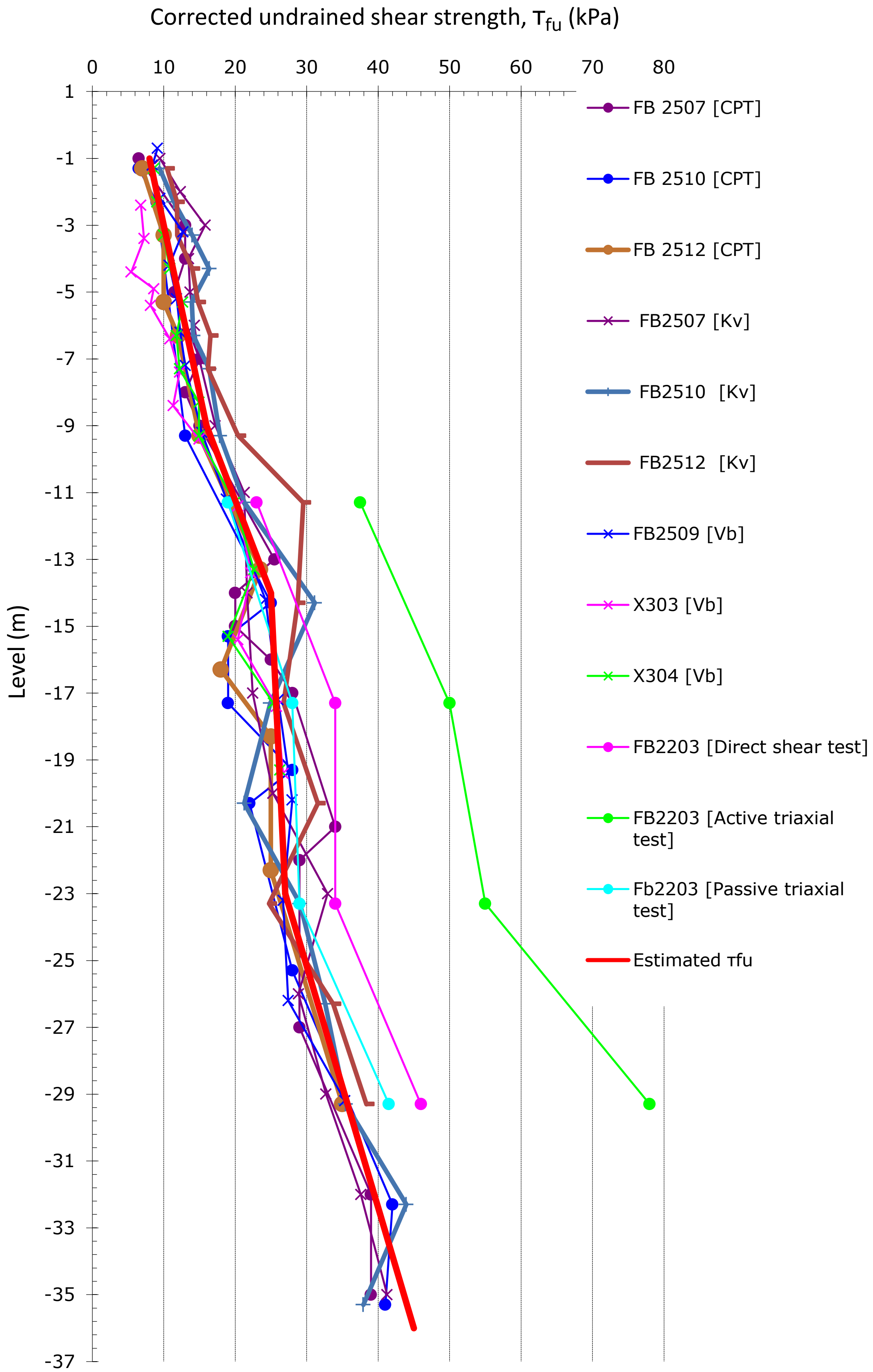
Appendix 1. Geotechnical Properties





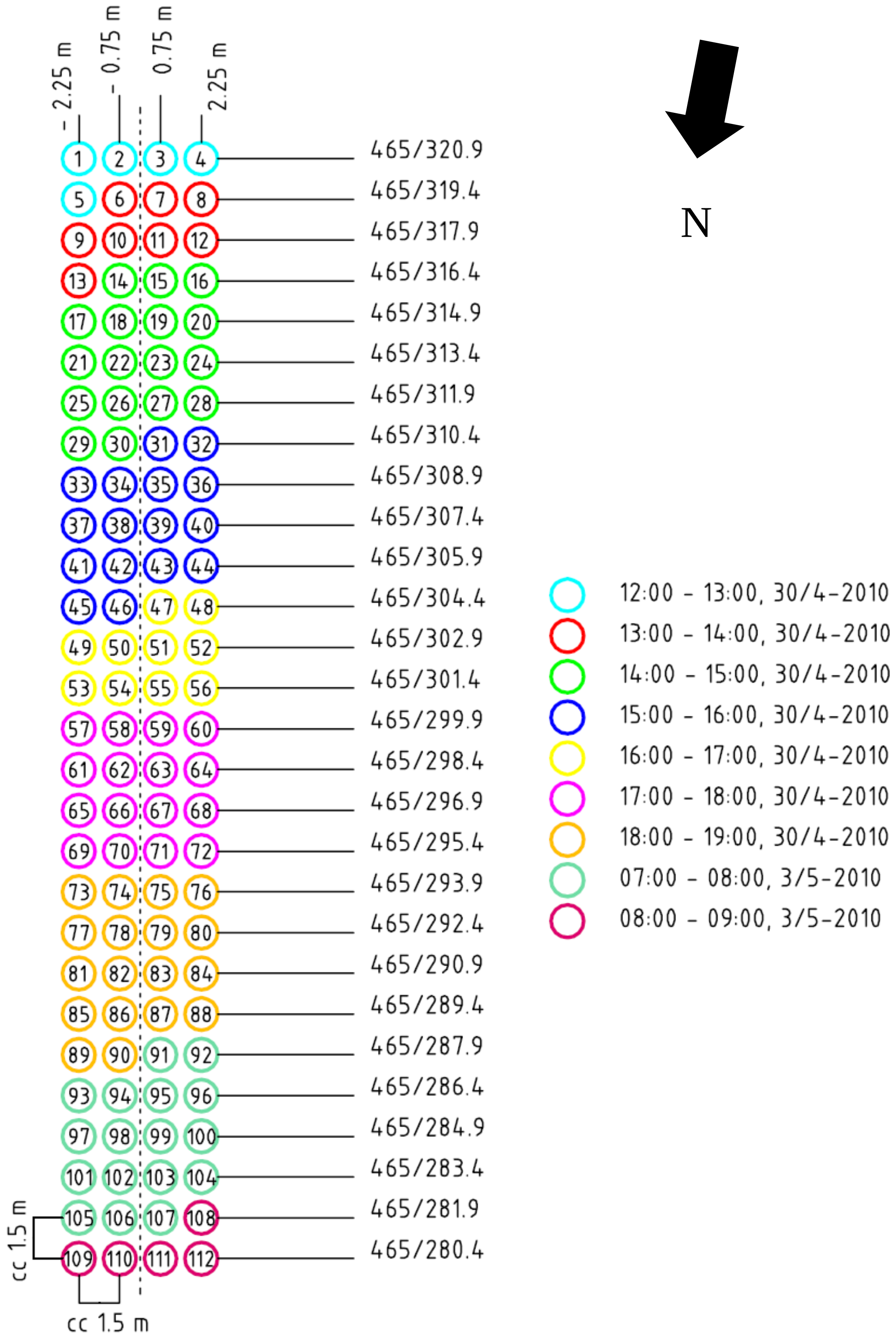


Pre consilidation pressure, σ'c (kPa)
0
50
100
150
200
250
300
1
-1
-3
-5
-7
-9
-11
-13
-15
-17
-19
-21
-23
-25
-27
-29
-31
-33
-35
-37
Level (m)
FB2203 [Kv]
FB2507 [Kv]
FB2512 [Kv]
Effective in-situ stress, σ'o
1,25*σ'o



Corrected undrained shear strength, τ_{fu} (kPa)
0
10
20
30
40
50
60
70
80
Level (m)
1
-1
-3
-5
-7
-9
-11
-13
-15
-17
-19
-21
-23
-25
-27
-29
-31
-33
-35
-37
FB 2507 [CPT]
FB 2510 [CPT]
FB 2512 [CPT]
FB2507 [Kv]
FB2510 [Kv]
FB2512 [Kv]
FB2509 [Vb]
X303 [Vb]
X304 [Vb]
FB2203 [Direct shear test]
FB2203 [Active triaxial test]
Fb2203 [Passive triaxial test]
Estimated τfu

Appendix 2: LC-Column Properties and Installation Pattern



Column Number	Section [m]	Distance from centerline [m]	Date	Time	Installation level [m]	Top level of column [m]	Bottom level of column [m]	Stabilized column length [m]	Average amount of lime/cement [Kg/m]
Uplifting rate = 20 mm/rotation for all installed lime-cement columns									
Mixing tool = "Pinnborr" with a diameter of 600 mm for all installed lime cement columns									
1	465+320.908	-2,25	2010-04-30	12:45	1,3	0,8	-5,7	6,5	31,2
2	465+320.903	-0,75	2010-04-30	12:48	1,3	0,8	-5,7	6,5	30,0
3	465+320.898	0,75	2010-04-30	12:51	1,3	0,8	-5,7	6,5	31,5
4	465+320.893	2,25	2010-04-30	12:55	1,3	0,8	-5,7	6,5	31,8
5	465+319.407	-2,25	2010-04-30	12:59	1,3	0,8	-5,7	6,5	31,9
6	465+319.402	-0,75	2010-04-30	13:02	1,3	0,8	-5,7	6,5	30,7
7	465+319.397	0,75	2010-04-30	13:06	1,3	0,8	-5,7	6,5	31,6
8	465+319.392	2,25	2010-04-30	13:09	1,3	0,8	-5,7	6,5	29,2
9	465+317.906	-2,25	2010-04-30	13:13	1,3	0,8	-5,8	6,6	31,8
10	465+317.901	-0,75	2010-04-30	13:16	1,3	0,8	-5,7	6,5	30,8
11	465+317.896	0,75	2010-04-30	13:19	1,3	0,8	-5,7	6,5	32,7
12	465+317.890	2,25	2010-04-30	13:23	1,3	0,8	-5,8	6,6	31,6
13	465+316.405	-2,25	2010-04-30	13:26	1,3	0,8	-5,8	6,6	31,6
14	465+316.400	-0,75	2010-04-30	14:04	1,3	0,8	-5,8	6,6	35,0
15	465+316.394	0,75	2010-04-30	14:08	1,3	0,8	-5,8	6,6	31,8
16	465+316.389	2,25	2010-04-30	14:11	1,3	0,8	-5,8	6,6	32,0
17	465+314.904	-2,25	2010-04-30	14:14	1,3	0,8	-5,8	6,6	31,9
18	465+314.899	-0,75	2010-04-30	14:18	1,3	0,8	-5,8	6,6	30,3
19	465+314.893	0,75	2010-04-30	14:21	1,3	0,8	-5,8	6,6	31,5
20	465+314.888	2,25	2010-04-30	14:24	1,3	0,8	-5,7	6,5	30,1
21	465+313.404	-2,25	2010-04-30	14:28	1,3	0,8	-5,7	6,5	28,6
22	465+313.398	-0,75	2010-04-30	14:31	1,3	0,8	-5,8	6,6	29,4
23	465+313.392	0,75	2010-04-30	14:34	1,3	0,8	-5,8	6,6	29,9
24	465+313.387	2,25	2010-04-30	14:38	1,3	0,8	-5,8	6,6	28,9
25	465+311.903	-2,25	2010-04-30	14:41	1,3	0,8	-5,7	6,5	29,2

Column Number	Section [m]	Distance from baseline [m]	Date	Time	Installation level [m]	Top level of column [m]	Bottom level of column [m]	Stabilised column length [m]	Average amount of lime/cement [Kg/m]
26	465+311.897	-0,75	2010-04-30	14:45	1,3	0,8	-5,7	6,5	30,1
27	465+311.891	0,75	2010-04-30	14:48	1,3	0,8	-5,7	6,5	29,7
28	465+311.886	2,25	2010-04-30	14:51	1,3	0,8	-5,7	6,5	29,6
29	465+310.402	-2,25	2010-04-30	14:56	1,3	0,8	-5,7	6,5	31,5
30	465+310.396	-0,75	2010-04-30	14:59	1,3	0,8	-5,7	6,5	28,4
31	465+310.390	0,75	2010-04-30	15:15	1,3	0,8	-5,7	6,5	28,9
32	465+310.385	2,25	2010-04-30	15:06	1,3	0,8	-5,7	6,5	31,3
33	465+308.901	-2,25	2010-04-30	15:11	1,3	0,8	-5,7	6,5	28,2
34	465+308.895	-0,75	2010-04-30	15:14	1,3	0,8	-5,7	6,5	29,6
35	465+308.889	0,75	2010-04-30	15:17	1,3	0,8	-5,7	6,5	28,3
36	465+308.883	2,25	2010-04-30	15:21	1,3	0,8	-5,7	6,5	27,9
37	465+307.400	-2,25	2010-04-30	15:24	1,3	0,8	-5,7	6,5	27,9
38	465+307.394	-0,75	2010-04-30	15:28	1,3	0,8	-5,7	6,5	28,7
39	465+307.388	0,75	2010-04-30	15:32	1,3	0,8	-5,7	6,5	28,1
40	465+307.382	2,25	2010-04-30	15:35	1,3	0,8	-5,7	6,5	29,5
41	465+305.900	-2,25	2010-04-30	15:40	1,3	0,8	-5,4	6,2	29,8
42	465+305.893	-0,75	2010-04-30	15:43	1,3	0,8	-5,7	6,5	30,5
43	465+305.887	0,75	2010-04-30	15:47	1,3	0,8	-5,4	6,2	29,0
44	465+305.881	2,25	2010-04-30	15:52	1,3	0,8	-5,4	6,2	31,6
45	465+304.399	-2,25	2010-04-30	15:55	1,3	0,8	-5,5	6,3	28,6
46	465+304.393	-0,75	2010-04-30	15:59	1,3	0,8	-5,4	6,2	29,2
47	465+304.386	0,75	2010-04-30	16:02	1,3	0,8	-5,4	6,2	28,7
48	465+304.380	2,25	2010-04-30	16:05	1,3	0,8	-5,4	6,2	30,2
49	465+302.898	-2,25	2010-04-30	16:08	1,3	0,8	-5,5	6,3	29,8
50	465+302.892	-0,75	2010-04-30	16:12	1,3	0,8	-5,4	6,2	30,3
51	465+302.886	0,75	2010-04-30	16:15	1,3	0,8	-5,4	6,2	31,0
52	465+302.879	2,25	2010-04-30	16:18	1,3	0,8	-5,5	6,3	30,7
53	465+301.397	-2,25	2010-04-30	16:22	1,3	0,8	-5,5	6,3	32,2
54	465+301.391	-0,75	2010-04-30	16:25	1,3	0,8	-5,4	6,2	30,0
55	465+301.385	0,75	2010-04-30	16:28	1,3	0,8	-5,4	6,2	30,2

Column Number	Section [m]	Distance from baseline [m]	Date	Time	Installation level [m]	Top level of column [m]	Bottom level of column [m]	Stabilised column length [m]	Average amount of lime/cement [Kg/m]
56	465+301.378	2,25	2010-04-30	16:31	1,3	0,8	-5,4	6,2	30,9
57	465+299.897	-2,25	2010-04-30	17:05	1,3	0,8	-5,5	6,3	33,3
58	465+299.890	-0,75	2010-04-30	17:08	1,3	0,8	-5,4	6,2	32,5
59	465+299.884	0,75	2010-04-30	17:11	1,3	0,8	-5,4	6,2	29,0
60	465+299.877	2,25	2010-04-30	17:14	1,3	0,8	-5,4	6,2	29,2
61	465+298.396	-2,25	2010-04-30	17:18	1,3	0,8	-5,4	6,2	29,3
62	465+298.389	-0,75	2010-04-30	17:21	1,3	0,8	-5,4	6,2	30,1
63	465+298.383	0,75	2010-04-30	17:24	1,3	0,8	-5,4	6,2	30,2
64	465+298.376	2,25	2010-04-30	17:28	1,3	0,8	-5,4	6,2	32,8
65	465+296.895	-2,25	2010-04-30	17:32	1,3	0,8	-5,4	6,2	30,8
66	465+296.889	-0,75	2010-04-30	17:35	1,3	0,8	-5,4	6,2	28,5
67	465+296.882	0,75	2010-04-30	17:38	1,3	0,8	-5,4	6,2	29,5
68	465+296.876	2,25	2010-04-30	17:43	1,3	0,8	-5,4	6,2	31,0
69	465+295.395	-2,25	2010-04-30	17:47	1,3	0,8	-5,5	6,3	32,1
70	465+295.388	-0,75	2010-04-30	17:50	1,3	0,8	-5,4	6,2	31,0
71	465+295.381	0,75	2010-04-30	17:53	1,3	0,8	-5,4	6,2	30,7
72	465+295.375	2,25	2010-04-30	17:56	1,3	0,8	-5,5	6,3	30,7
73	465+293.894	-2,25	2010-04-30	18:00	1,3	0,8	-5,5	6,3	29,3
74	465+293.887	-0,75	2010-04-30	18:03	1,3	0,8	-5,4	6,2	30,0
75	465+293.881	0,75	2010-04-30	18:07	1,3	0,8	-5,4	6,2	29,0
76	465+293.874	2,25	2010-04-30	18:10	1,3	0,8	-5,4	6,2	28,4
77	465+292.393	-2,25	2010-04-30	18:13	1,3	0,8	-5,4	6,2	29,5
78	465+292.386	-0,75	2010-04-30	18:16	1,3	0,8	-5,4	6,2	28,4
79	465+292.380	0,75	2010-04-30	18:20	1,3	0,8	-5,4	6,2	33,5
80	465+292.373	2,25	2010-04-30	18:23	1,3	0,8	-5,4	6,2	28,8
81	465+290.893	-2,25	2010-04-30	18:26	1,3	0,8	-5,4	6,2	30,9
82	465+290.886	-0,75	2010-04-30	18:29	1,3	0,8	-5,4	6,2	29,9
83	465+290.879	0,75	2010-04-30	18:32	1,3	0,8	-5,4	6,2	31,0
84	465+290.872	2,25	2010-04-30	18:36	1,3	0,8	-5,4	6,2	32,1
85	465+289.392	-2,25	2010-04-30	18:40	1,3	0,8	-5,4	6,2	30,3

Column Number	Section [m]	Distance from baseline [m]	Date	Time	Installation level [m]	Top level of column [m]	Bottom level of column [m]	Stabilised column length [m]	Average amount of lime/cement [Kg/m]
86	465+289.385	-0,75	2010-04-30	18:43	1,3	0,8	-5,4	6,2	31,6
87	465+289.378	0,75	2010-04-30	18:47	1,3	0,8	-5,4	6,2	29,1
88	465+289.371	2,25	2010-04-30	18:50	1,3	0,8	-5,4	6,2	29,9
89	465+287.891	-2,25	2010-04-30	18:53	1,3	0,8	-5,4	6,2	30,6
90	465+287.885	-0,75	2010-04-30	18:56	1,3	0,8	-5,5	6,3	29,8
91	465+287.878	0,75	2010-05-03	07:03	1,3	0,8	-5,4	6,2	36,4
92	465+287.871	2,25	2010-05-03	07:07	1,3	0,8	-5,4	6,2	29,1
93	465+286.391	-2,25	2010-05-03	07:10	1,3	0,8	-5,5	6,3	29,1
94	465+286.384	-0,75	2010-05-03	07:14	1,3	0,8	-5,4	6,2	28,8
95	465+286.377	0,75	2010-05-03	07:17	1,3	0,8	-5,4	6,2	29,3
96	465+286.370	2,25	2010-05-03	07:21	1,3	0,8	-5,4	6,2	28,5
97	465+284.890	-2,25	2010-05-03	07:25	1,3	0,8	-5,4	6,2	28,9
98	465+284.883	-0,75	2010-05-03	07:28	1,3	0,8	-5,4	6,2	28,0
99	465+284.876	0,75	2010-05-03	07:31	1,3	0,8	-5,4	6,2	28,6
100	465+284.869	2,25	2010-05-03	07:34	1,3	0,8	-5,4	6,2	28,7
101	465+283.390	-2,25	2010-05-03	07:37	1,3	0,8	-5,4	6,2	29,7
102	465+283.383	-0,75	2010-05-03	07:41	1,3	0,8	-5,4	6,2	29,5
103	465+283.375	0,75	2010-05-03	07:44	1,3	0,8	-5,4	6,2	29,2
104	465+283.368	2,25	2010-05-03	07:47	1,3	0,8	-5,4	6,2	30,5
105	465+281.889	-2,25	2010-05-03	07:51	1,3	0,8	-5,4	6,2	29,0
106	465+281.882	-0,75	2010-05-03	07:54	1,3	0,8	-5,4	6,2	27,9
107	465+281.875	0,75	2010-05-03	07:57	1,3	0,8	-5,4	6,2	27,8
108	465+281.868	2,25	2010-05-03	08:00	1,3	0,8	-5,4	6,2	28,6
109	465+280.389	-2,25	2010-05-03	08:33	1,3	0,8	-5,4	6,2	28,2
110	465+280.382	-0,75	2010-05-03	08:37	1,3	0,8	-5,5	6,3	30,1
111	465+280.374	0,75	2010-05-03	08:40	1,3	0,8	-5,4	6,2	30,7
112	465+280.367	2,25	2010-05-03	08:44	1,3	0,8	-5,5	6,3	31,2

Appendix 3: Approach 1, Input Data for Rehnman's Method

η=	0,75	(-)
β=	1	(-)
γ=	1	(-)
δ=	1	(-)
α=	1	(-)
b=	5,1	m
ρ_{binder} =	1800	kg/m^3
m_{binder} =	30	kg/m
r_{column}=	0,3	m

Phase A

Piles=	10		V_{binder}=	0,103	m^3
l=	3,30	m	V_{column}=	1,753	m^3
V,piles =	17,53	m^3	V_{binder}/V_{column} =	0,0589	(-)
d=	6,20	m			
e=	3,80	m			
x=	0,226	m			

$\delta_v(3,8)$=	87,696	mm
δv(3,8), corrected=	5,169	mm

z (m)	Level (m)	δ_h(z) (mm)
0	0,8	5,17
0,5	0,3	4,09
1	-0,2	3,02
1,5	-0,7	1,94
2	-1,2	0,86
2,5	-1,7	-0,22

Phase B

Piles=	26		V_{binder}=	0,103	m^3
l=	9,30	m	V_{column}=	1,753	m^3
V,piles =	45,58	m^3	V_{binder}/V_{column} =	0,0589	(-)
d=	6,20	m			
e=	3,80	m			
x=	0,341	m			

$\delta_v(3,8)$= 132,099 mm
δv(3,8), corrected= 7,787 mm

z (m)	Level (m)	$\delta_h(z)$ (mm)
0	0,8	7,79
0,5	0,3	6,16
1	-0,2	4,54
1,5	-0,7	2,92
2	-1,2	1,30
2,5	-1,7	-0,32

Phase C

Piles=	44		V_{binder}=	0,103	m^3
l=	16,50	m	V_{column}=	1,753	m^3
V,piles =	77,13	m^3	V_{binder}/V_{column} =	0,0589	(-)
d=	6,20	m			
e=	3,80	m			
x=	0,384	m			

$\delta_v(3,8)$= 148,563 mm
δv(3,8), corrected= 8,757 mm

z (m)	Level (m)	$\delta_h(z)$ (mm)
0	0,8	8,76
0,5	0,3	6,93
1	-0,2	5,11
1,5	-0,7	3,28
2	-1,2	1,46
2,5	-1,7	-0,36

Phase D

Piles=	85		V_{binder}=	0,106	m^3
l=	30,60	m	V_{column}=	1,795	m^3
V,piles =	152,61	m^3	V_{binder}/V_{column} =	0,0589	(-)
d=	6,35	m			
e=	3,80	m			
x=	0,449	m			

$\delta_v(3,8)$=	180,177	mm
δv(3,8), corrected=	10,621	mm

z (m)	**Level (m)**	**δ_h(z) (mm)**
0	0,8	10,62
0,5	0,3	8,54
1	-0,2	6,46
1,5	-0,7	4,37
2	-1,2	2,29
2,5	-1,7	0,21
3	-2,2	-1,87

Appendix 4: Approach 1, Input Data for SSPM, Cylindrical Pile

$R=$	0,3	m		(Column 6 - 42)
$L_1=$	6,5	m		(Column 6 - 42)
$L_2=$	6,2	m		(Column 43 - 90)
$V_{binder}=$	0,10	m^3		
$V_{column}=$	1,75	m^3		
$V_{binder}/V_{column}=$	0,0589	=	5,89	%
$\rho_{binder}=$	1800	kg/m^3		
$m_{binder}=$	30	kg/m		
Column top level =	0,8	m		

r = Radial distance to inclinometer

Phase A (Column 47 - 56)

Total displacement = 49 mm

Corrected displacement = 2,90 mm

Phase B (Column 47 - 72)

Total displacement = 125 mm

Corrected displacement = 7,39 mm

Phase C (Column 47 - 90)

Total displacement = 164 mm

Corrected displacement = 9,67 mm

Phase D (Column 6 - 90)

Total displacement = 231 m

Corrected displacement = 13,64 mm

Column No.	r (m)	X (m)	Y (m)	Displacement (mm)
6	20,658	7,1	19,4	0,654
7	20,192	5,6	19,4	0,683
8	19,829	4,1	19,4	0,707
9	19,859	8,6	17,9	0,705
10	19,257	7,1	17,9	0,747
11	18,756	5,6	17,9	0,786
12	18,364	4,1	17,9	0,818
13	18,518	8,6	16,4	0,805
14	17,871	7,1	16,4	0,861
15	17,330	5,6	16,4	0,912
16	16,905	4,1	16,4	0,955
17	17,204	8,6	14,9	0,924
18	16,505	7,1	14,9	0,999
19	15,918	5,6	14,9	1,069
20	15,454	4,1	14,9	1,129
21	15,922	8,6	13,4	1,068
22	15,165	7,1	13,4	1,169
23	14,523	5,6	13,4	1,266
24	14,013	4,1	13,4	1,351
25	14,682	8,6	11,9	1,241
26	13,857	7,1	11,9	1,379
27	13,152	5,6	11,9	1,516
28	12,587	4,1	11,9	1,641
29	13,495	8,6	10,4	1,447
30	12,592	7,1	10,4	1,639
31	11,812	5,6	10,4	1,837
32	11,179	4,1	10,4	2,023
33	12,376	8,6	8,9	1,691
34	11,385	7,1	8,9	1,960
35	10,515	5,6	8,9	2,250
36	9,799	4,1	8,9	2,539
37	11,345	8,6	7,4	1,972
38	10,255	7,1	7,4	2,349
39	9,280	5,6	7,4	2,782
40	8,460	4,1	7,4	3,241
41	10,429	8,6	5,9	2,282
42	9,231	7,1	5,9	2,806
43	8,134	5,6	5,9	3,353
44	7,185	4,1	5,9	4,092
45	9,660	8,6	4,4	2,516
46	8,353	7,1	4,4	3,211
47	7,122	5,6	4,4	4,149

48	6,014	4,1	4,4	5,371
49	9,076	8,6	2,9	2,797
50	7,669	7,1	2,9	3,689
51	6,306	5,6	2,9	5,003
52	5,022	4,1	2,9	6,963
53	8,713	8,6	1,4	2,994
54	7,237	7,1	1,4	4,046
55	5,772	5,6	1,4	5,706
56	4,332	4,1	1,4	8,514
57	8,601	8,6	0,1	3,060
58	7,101	7,1	0,1	4,168
59	5,601	5,6	0,1	5,962
60	4,101	4,1	0,1	9,151
61	8,748	8,6	1,6	2,975
62	7,278	7,1	1,6	4,010
63	5,824	5,6	1,6	5,632
64	4,401	4,1	1,6	8,338
65	9,142	8,6	3,1	2,763
66	7,747	7,1	3,1	3,629
67	6,401	5,6	3,1	4,891
68	5,140	4,1	3,1	6,740
69	9,753	8,6	4,6	2,475
70	8,460	7,1	4,6	3,144
71	7,247	5,6	4,6	4,037
72	6,162	4,1	4,6	5,180
73	10,544	8,6	6,1	2,163
74	9,361	7,1	6,1	2,655
75	8,281	5,6	6,1	3,257
76	7,350	4,1	6,1	3,948
77	11,477	8,6	7,6	1,864
78	10,400	7,1	7,6	2,215
79	9,440	5,6	7,6	2,617
80	8,635	4,1	7,6	3,039
81	12,521	8,6	9,1	1,595
82	11,542	7,1	9,1	1,845
83	10,685	5,6	9,1	2,114
84	9,981	4,1	9,1	2,379
85	13,650	8,6	10,6	1,363
86	12,758	7,1	10,6	1,542
87	11,988	5,6	10,6	1,724
88	11,365	4,1	10,6	1,896
89	14,845	8,6	12,1	1,168
90	14,029	7,1	12,1	1,297

Appendix 5: Approach 1, Input Data for SSPM, Planar wall

Phase D (Column 6 - 90)

$R=$	0,3	m	$V_{soil}=$	990,98	m^3			
$L_{mean}=$	6,35	m	$V_{binder}/V_{soil}=$	0,0091		=	0,91	%
$w=$	2,55	m	$\rho_{binder}=$	1800	kg/m^3			
$x=$	6,35	m	$m_{binder}=$	30	kg/m			
$V_{binder}=$	9,00	m^3	Column top level =	0,8	m			

z= Depth from column top level

z (m)	Level (m)	Displacement (mm)	Corrected Displacement (mm)
0	0,8	1275	11,57
0,5	0,3	1204	10,93
1	-0,2	1124	10,21
1,5	-0,7	1040	9,45
2	-1,2	956	8,69
2,5	-1,7	876	7,95
3	-2,2	799	7,26
3,5	-2,7	729	6,62
4	-3,2	663	6,02
4,5	-3,7	603	5,47
5	-4,2	546	4,96
5,5	-4,7	494	4,48
6	-5,2	444	4,04
6,5	-5,7	397	3,61
7	-6,2	354	3,21
7,5	-6,7	313	2,84
8	-7,2	274	2,50
8,5	-7,7	239	2,17
9	-8,2	207	1,88
9,5	-8,7	177	1,61
10	-9,2	151	1,37
10,5	-9,7	127	1,16
11	-10,2	106	0,97

11,5	-10,7	87	0,80
12	-11,2	71	0,65
12,5	-11,7	56	0,52
13	-12,2	44	0,40
13,5	-12,7	33	0,30
14	-13,2	23	0,22
14,5	-13,7	15	0,14
15	-14,2	8	0,07
15,5	-14,7	2	0,02
16	-15,2	-3	-0,03
16,5	-15,7	-8	-0,07
17	-16,2	-11	-0,11
17,5	-16,7	-15	-0,14
18	-17,2	-18	-0,17
18,5	-17,7	-20	-0,19
19	-18,2	-22	-0,21
19,5	-18,7	-24	-0,22
20	-19,2	-26	-0,24
20,5	-19,7	-27	-0,25
21	-20,2	-28	-0,26
21,5	-20,7	-29	-0,26
22	-21,2	-29	-0,27

Appendix 6: Approach 2 - 5, Input Data

SSPM, Planar Wall, Approach 2

Phase D (Column 6 - 90)

$R=$	0,3	m	$V_{column}=$	1,80	m^3	
$L_{mean}=$	6,35	m	$V_{binder}/V_{column}=$	0,0589	=	5,89 %
$w=$	2,55	m	$\rho_{binder}=$	1800	kg/m^3	
$x=$	6,35	m	$m_{binder}=$	30	kg/m	
$V_{binder}=$	0,11	m^3	Column top level =	0,8	m	

z= Depth from column top level a= 0,126

SSPM, Planar Wall, Approach 3

Phase D (Column 6 - 90)

$R=$	0,3	m	$V_{column}=$	1,80	m^3	
$L_{mean}=$	6,35	m	$V_{binder}/V_{column}=$	0,0589	=	5,89 %
$w=$	2,55	m	$\rho_{binder}=$	1800	kg/m^3	
$x=$	6,35	m	$m_{binder}=$	30	kg/m	
$V_{binder}=$	0,11	m^3	Column top level =	0,8	m	

z= Depth from column top level a= 0,126

$\mu_w=$ 1,4,5,6 (-)

SSPM, Planar Wall, Approach 4

Phase D (Column 6 - 90)

$R=$	0,3	m	$V_{column}=$	1,80	m^3	
$L_{mean}=$	6,35	m	$V_{binder}/V_{column}=$	0,0589	=	5,89 %
$w=$	2,55	m	$\rho_{binder}=$	1800	kg/m^3	
$x=$	6,35	m	$m_{binder}=$	30	kg/m	
$V_{binder}=$	0,11	m^3	Column top level =	0,8	m	

$z=$ Depth from column top level $a=$ 0,126

$\mu_w=$ 5 (-)
$\mu L=$ 0.7, 0.8, 0.9 (-)

SSPM, Planar Wall, Approach 5

Phase D (Column 6 - 90)

$R=$	0,3	m	$V_{column}=$	1,80	m^3	
$L_{mean}=$	6,35	m	$V_{binder}/V_{column}=$	0,0589	=	5,89 %
$w=$	2,55	m	$\rho_{binder}=$	1800	kg/m^3	
$x=$	6,35	m	$m_{binder}=$	30	kg/m	
$V_{binder}=$	0,11	m^3	Column top level =	0,8	m	

$z=$ Depth from column top level $a=$ 0,126

$\mu_w=$ 1, 5, 5.5, 6 (-)
$\mu L=$ 0.8 (-)

Appendix 7: Section 465/535 & 465/750, Input Data

SSPM, Planar Wall, 465/535

R=	0,3	m	V_{soil}=	1,696	m^3
L_{mean}=	7.5	m	V_{binder}/V_{column} =	0,0589	(-)
w=	3,5	m	ρ_{binder} =	1800	kg/m^3
x=	5	m	m_{binder} =	30	kg/m
V_{binder}=	0,1	m^3	Column top level =	0,8	m
a=	0,217				
μ_w=	4,5,6	(-)			
μ_L=	0.8	(-)			

SSPM, Planar Wall, 465/750

R=	0,3	m	V_{column}=	1,311	m3
L_{mean}=	5.8	m	V_{binder}/V_{column} =	0,0589	(-)
w=	3,3	m	ρ_{binder} =	1800	kg/m3
x=	6,65	m	m_{binder} =	30	kg/m
V_{binder}=	0,077	m3	Column top level =	0,9	m
a=	0,202				
μ_w=	4,5,6	(-)			
μ_L=	0.8	(-)			

Appendix 8: PLAXIS, Input Parameters

Young's modulus, E_{50}

$1000*c_u$	For silty clay
$500*c_u$	For low plastic clay
$250*c_u$	For high plastic and muddy clay
$150*c_u$	For mud

(Trafikverket, 2011)

Horizontal stresses

Soil type	**K_o**
Clay	$0.31 + 0.71(W_L\text{-}0.2)$

For over consolidated clay, the increase in lateral stress coefficient = $K_0*OCR^{0.55}$

(Trafikverket, 2011)

Section 465/300

C_u= 8 kPa,

$C_{u,inc}$= 1 kPa/m

W_L= 85%

High plastic and muddy clay is assumed.

OCR≈1.3

V_{binder}= 0.106 m^3

V_{Column}= 1.795 m^3

a = 0.126 %

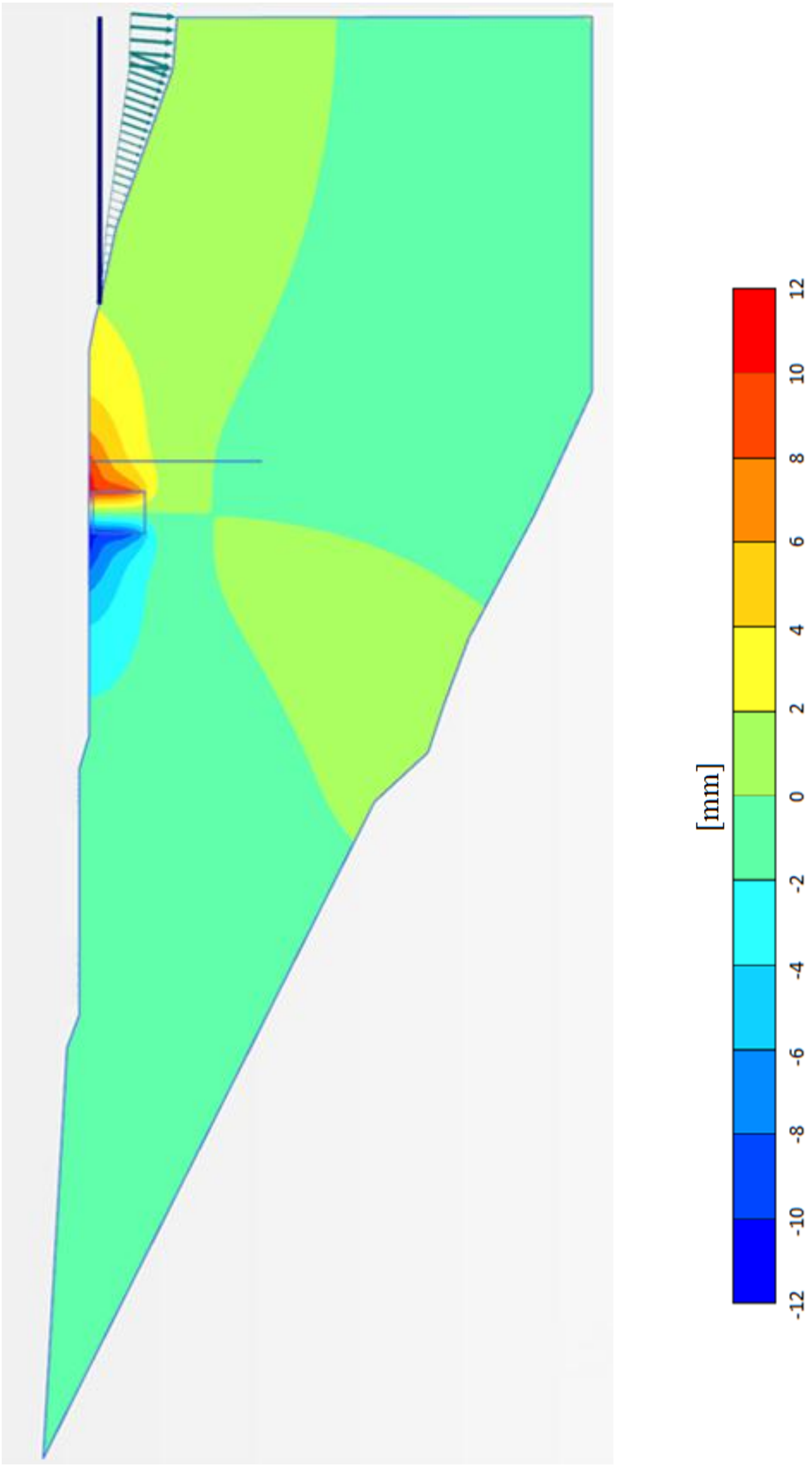
$E_{50}=E_u$= 250*8 = 2000 kPa

$E_{u,inc}$=250*1 = 250 kPa

K_0= (0.31 + 0.71(0.85 - 0.2))*$1.3^{0.55}$ = 0.891

Volume strain = $\frac{V_{Binder}}{V_{Column}} \cdot a = 0.0074 = 0.74\%$

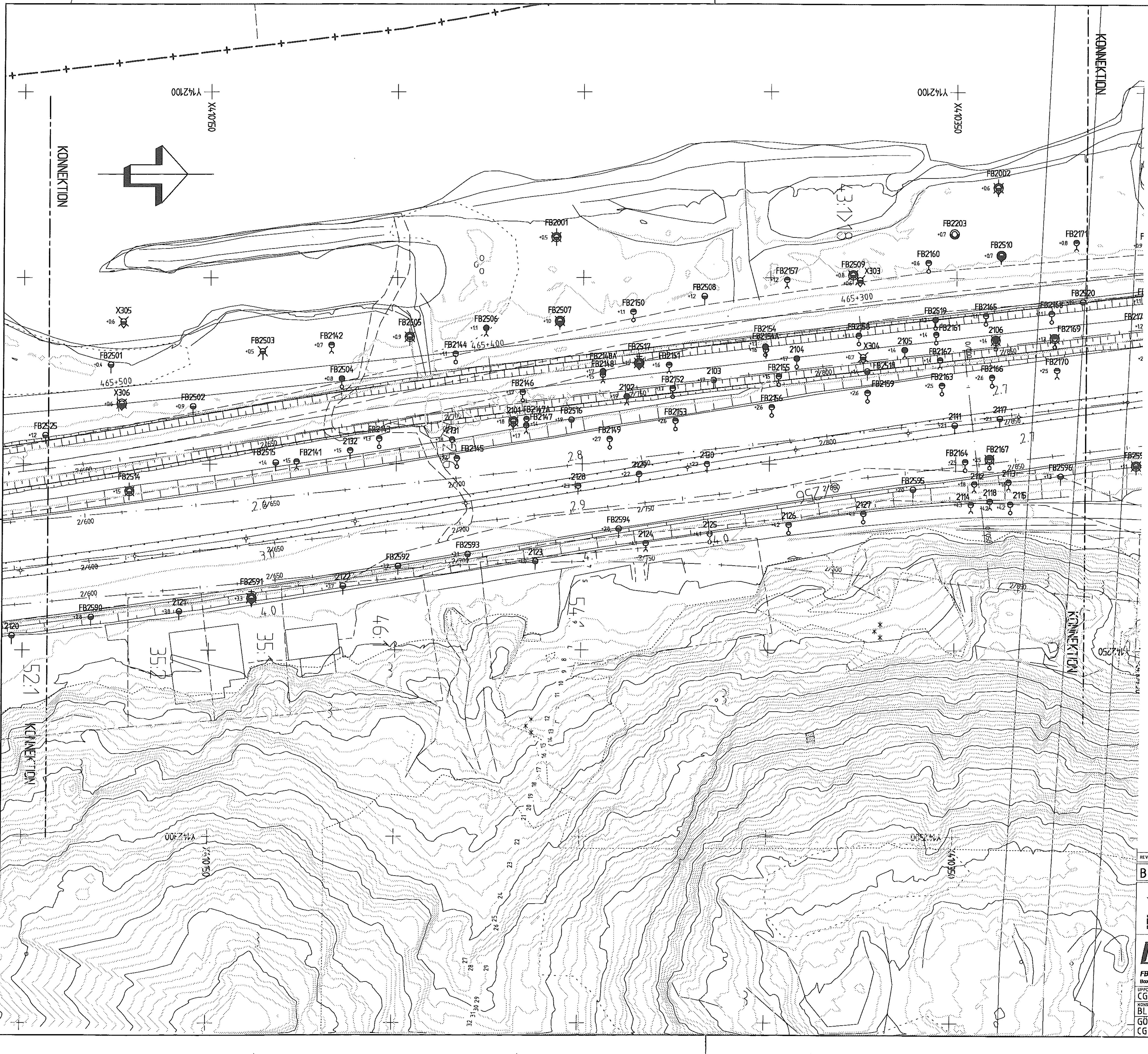
Appendix 9: PLAXIS, Displacement Results



Appendix 10: Drawings

Drawing 1: Plan drawing section 465/300

Drawing 2: Cross-section drawing section 465/300



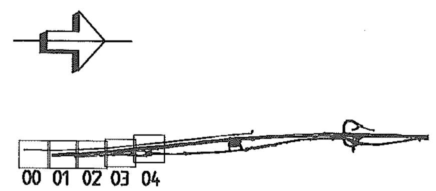
ANMÄRKNINGAR:

KOORDINATSYSTEM: RT 90 7.5 gon V, 60:-1
HÖJDSYSTEM: RHB 70

BETECKNINGAR

GEOTEKNISKA BETECKNINGAR ENLIGT
SGF:S BETECKNINGSSYSTEM, SE www.sgf.net

PLANNYCKEL



SKALSTOCK

Meter, skala 1:500 i A1-format (skala 1:1000 A3-format)

REV	ANT	ÄNDRINGEN AVSER	GODK.	DATUM	VV DATUM	VV DIARIENUMMER

BYGGHANDLING

Vägverket
BANVERKET

BANAVÄG I VÄST

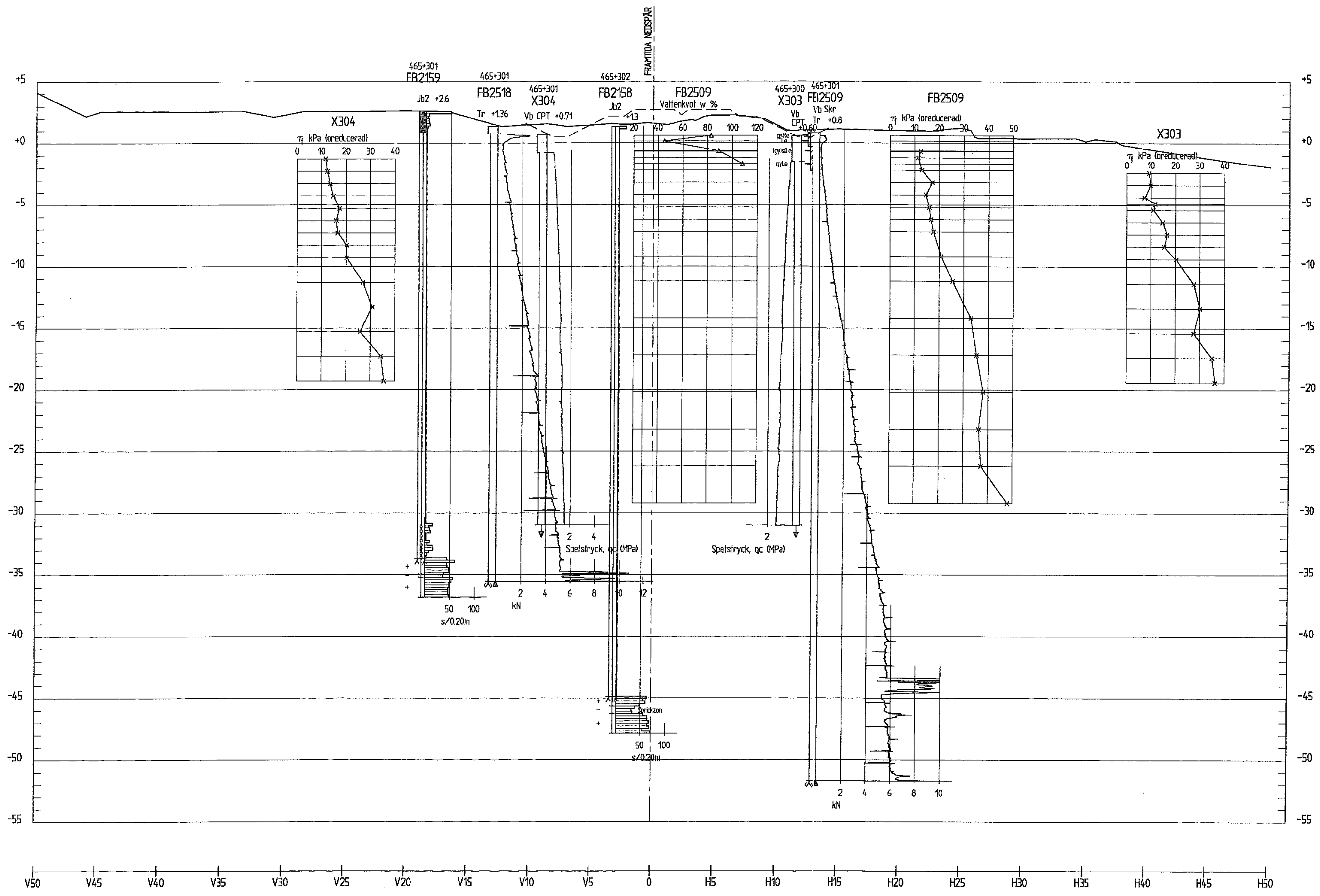
E45 - NORGE/VÄNERBANAN
DELEN AGNESBERG-BOHUS
ENTREPRENAD E41B HUVUDDEL 1

FB FLYGFÄLTSBYRÅN

FB Engineering AB
Box 12076, 402 41 GÖTEBORG
Tel: 031-776 10 00
Fax: 031-776 11 00

GEOTEKNISK UNDERSÖKNING
JÄRNVÄG
KM 465+240 - 465+520
PLAN

UPPDRAGSANSVARIG CG PETERSSON	UPPDRAGSNUMMER 161525			
KONSTR BL	GRANSK CHJU	KONSTRUKTIONSNR	FORMAT A1	SKALA 1:500
GÖTEBORG CG PETERSSON	2009-09-01	OBJEKT NR 54 27 62	RITNINGSNR 502G1103	REV



TVÄRSEKTION 465+300

1: 200

FÖRKLARINGAR

—— Befintligt

— — Blivande

BETECKNINGAR

GEOTEKNISKA BETECKNINGAR ENLIGT SGF:S BETECKNINGSSYSTEM, SE www.sgf.net

KOORDINATSYSTEM RT 90 7.5 gon V 60:-1

HÖJDSYSTEM RHB 70

REV	ANT	ÄNDRINGEN AVSER	GODK	DATUM	VV DATUM	VV DIARIENUMMER

BYGGHANDLING

Vägverket

BANVERKET

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BanaVäg i Väst

E45 - NORGE/VÄNERBANAN

DELEN AGNESBERG-BOHUS

ENTREPRENAD E41B HUVUDDEL 1

GEOTEKNISK UNDERSÖKNING

JÄRNVÄG

KM 465+300

TVÄRSEKTION

UPPDRAGSANSVARIG: CG PETERSSON	UPPDRAGSNUMMER: 161525	KONSTRUKTIONSNR	FORMAT: A1	SKALA: 1:200	
KONSTR: MP	GRANSK: CHJU				
GÖTEBORG CG PETERSSON	2009-09-01	OBJEKT NR: 54 27 62	RITNINGSNR: 502G1316		REV