Deformation analysis of bridge foundation on lime-cement columns

Swelling and consolidation of reinforced soil

Master's Thesis in Infrastructure and Environmental Engineering

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CHALMERS UNIVERSITY OF TECHNOLOGY
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Lime cement column installation (Trafikverket, 2014).

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ABSTRACT

Along the newly built E45 between Gothenburg and Trollhättan a number of bridges were constructed. Some of the bridges have issues with settlement larger than anticipated. The bridge of interest in this report was founded on lime cement columns and was one of the bridges with large settlements. In order to create understanding of the situation an analysis of how swelling and consolidation affect settlement of bridge 15-1806 on E45 has been made. To do this analysis hand calculations and a Plaxis simulation has been made. Input data was developed through literature studies and interviews with involved people. The result of the report shows that swelling has a large effect on total movement of the bridge. The swelling of the soil below the bridge is 29 cm before the construction began. The settlement of the bridge is 10,6 cm after 2 months calculated by consolidation theory. In order to reduce the settlement in the future the swelling should not be overlooked. If the time span between unloading and reloading could be shortened the total impact of the swelling could be reduced. Also with the use of unloading-reloading oedometer tests the soil parameters for swelling and reloading could be estimated in an accurate way.

Key words: Swelling, Lime-Cement, Columns, Settlement, Consolidation, Unloading, Foundation, Bridge, Plaxis, Unloading-reloading, Oedometer, E45, BanaVäg i Väst

Deformationsanalys av brofundament på kalk-cementpelarförstärkt jord
Svällning och konsolidering av kalk-cementpelarförstärkt jord

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SAMMANFATTNING

Längs den nybyggda väg E45 mellan Göteborg och Trollhättan har ett antal broar konstruerats. Några av broarna har efter en tid fått problem med att sättningarna blivit större än förväntat. Den bro som fokus av rapporten ligger på är grundad på kalk-cementpelare och är en av broarna som har utvecklat mest sättningar. För att bilda förståelse för situationen har en analys av hur svällning och konsolidering har påverkat sättningarna i bro 15-1806 på väg E45 gjorts. I denna analys har handberäkningar och en simulering i det finita elementprogrammet Plaxis utförts. In-data har tagits fram genom litteraturstudier samt samtal med personer involverade i projektet och i geotekniska frågor. Resultaten av rapporten visar på att svällning har en stor inverkan på den totala rörelsen av bron. Svällningen innan uppbyggnad av bron uppgår till 29 cm. Sättningen av bron är 10,6 cm och är uträknad med konsoliderings teori över 2 månaders tid. För att minska sättningar i framtiden bör svällning få hög omtanke. Om tidsspannet mellan avlastning av jord och pålastning av bro kan minskas kan den totala inverkan av svällning minskas. Med användning av av-pålastnings ödometertest kan svällnings av pålastning parametrar tas fram på ett pålitligt sätt.

Nyckelord: Svällning, Kalk-Cement, Pelare, Sättningar, konsolidering, Avlastning, Grundläggning, Bro, Plaxis, Av-pålastning, Ödometer, E45, BanaVäg i Väst

Contents

Preface

We wish to express our deepest gratitude to Jan Ekström at Swedish Transport Administration for your commitment and for helping us make this Master's thesis possible. We would like to thank Jorge Yannie, for his invaluable help in Plaxis and settlement theory. We would also like to thank Minna Karstunen for taking time guiding us onto the path when we were lost.

Gothenburg, June 2014

Marcus Andreasson and Jacob Rådberg

Notations

Roman upper case letters

A	$[-]$	Activity of clay mineral
C	$[\%]$	Clay content
D	$[m]$	Diameter
E_{col}	$[\%]$	E-modulus of column
H	$[m]$	Depth down to firm layer
M	[kPa]	Oedometer modulus
M'	[kPa]	Oedometer modulus at increasing stiffness
M_0	[kPa]	Oedometer modulus before pre-consolidation pressure
M_{block}	[kPa]	Oedometer modulus of column and soil composite
M_L	[kPa]	Oedometer modulus beyond pre-consolidation pressure
M_{ul}	$[kPa]$	Unloading modulus
P	$[-]$	Total proportion of columns
S	$[\%]$	Swelling potential of clay
T	$[Nm]$	Torque at failure, Field vane test
T_v	$[-]$	Time factor of consolidation
U_v	$[\%]$	Degree of consolidation

Roman lower case letters

a	$[-]$	Constant, around 250 in calculation of M_{ul}, see equation (5.17)
c	$[m]$	Centre-centre distance between columns
c_{krit}	$[kPa]$	Critical shear strength
c_u	$[kPa]$	Undrained shear strength
$c_{u,col}$	$[kPa]$	Shear strength of column
c_{uFVT}	$[kPa]$	Undrained shear strength from Field vane test
c_{uk}	$[kPa]$	Undrained shear strength of column

c_v	$[m^2/s]$	Coefficient of consolidation
c_{vs}	$[m^2/s]$	Coefficient of swelling
d	$[m]$	Drainage depth of soil
d_{FVT}	[m]	Overall vane width, Field vane test
h_{FTV}	[m]	Vane length, Field vane test
k	$[m/day]$	Permeability
m_v	$[m^2/kN]$	Volume of compressibility
n	$[-]$	Constant value, 0.3, used in calculation of M_{ul}
r_{col}	$[m]$	Radius of column
s	$[m]$	Swelling or settlement of clay due to consolidation
t	$[s]$	Time
u_0	$[kPa]$	In-situ pore pressure
q	$[kPa]$	Pressure from load

Greek letters

γ	$[\mathrm{kN/m^3}]$	Unit weight
$\Delta\sigma$	$[kPa]$	Applied stress
$\Delta\sigma_{block}$	[kPa]	Stress in column and soil composite
$\Delta\sigma_{block,\lim}$	[kPa]	Stress in limit between column and soil
$\Delta\sigma_{ul}$	$[kPa]$	Unloaded stress
σ'	[kPa]	Effective stress
σ'_c	$[kPa]$	Pre-consolidation pressure
σ_f	$[kPa]$	Total pressure at failure
σ'_L	[kPa]	Effective stress beyond pre-consolidation pressure
σ'_{v_0}	$[kPa]$	Vertical effective stress
μ_{col}	$[m/day]$	Permeability of column
η_{LC}	$[-]$	Load distribution factor

λ^*	$[-]$	Modified compression index
κ^*	$[-]$	Modified swelling index
$\tau_{fu,col}$	$[kPa]$	Undrained shear stress of column at failure
$\phi_{u,col}$	$[°]$	Undrained friction angle of column

1 Introduction

Geotechnical engineering is a semi-empirical science which can somewhat be seen as more of an art than other areas within the civil engineering profession. Non-conservative is a word often used to describe the properties of soil. That means that the soil adapts to whatever has happened to it in a previous time. Even though soil is anisotropic and variable, engineers working with soil modelling often assume it to be isotropic and homogenous. To be able make an assumption close enough to reality is why geotechnical engineering can be seen as an art (Holtz & Kovacs, 1981).

Stabilisation of soils with lime-cement columns is something that is used in a wide range of construction applications in Sweden. The project BanaVäg i Väst was one project where this type of stabilisation was used in a great extent.

1.1 Background

In 1980 the idea of building a new and improved transport stretch to expand the communication and transport industry with Norway was discussed. The project, BanaVäg i Väst, was planned and the first part of the construction started in 2004. In December 9th 2012 the traffic on the new road and railroad between Gothenburg and Trollhättan could start. Within this project there where a number of bridges constructed. The settlement measurements of the foundations of these bridges showed that there where varying settlements during the construction phase.

The bridges of interest are situated along E45 and the railway between Gothenburg and Lilla Edet. All of the bridges were founded on lime-cement columns and constructed in a quite similar way, with some local variations. Some of the bridges had larger than expected settlements and the reason for this is unclear.

According to Ekström (2014) the excavations were left open in different lengths of time. Geotechnical experts in the Swedish Transport Administration believe that the large variations in settlements between the bridges might be partly explained by different amount of swelling of the soils, since the measurements started after construction of the bottom analysis plate of the bridge and not from the excavation start.

1.2 Aim

The aim of this report is to analyse how swelling and consolidation affect settlement of the bridge 15-1806 on E45 founded on lime-cement columns. A model in the finite element program Plaxis is to be developed to study this.

1.3 Objectives

- To find an applicable model for making realistic simulations of settlement in bridge foundation due to the effect of swelling and consolidation.
- To evaluate the swelling effects on Gothenburg clay and the effect on bridge displacements.
- To simulate the effects of swelling and consolidation in Plaxis.
- To perform comparative hand calculations to assess the magnitude of swelling and consolidation.
- To compare the hand calculations with the Plaxis model of the area.

1.4 Scope

The bridge along E45 between Tingsberg and Kärra, bridge number 15-1806, was chosen as the main focus of this report since it was one of the typical bridges with settlement problems, being founded on lime-cement column reinforced clay.

The focus of this report is on vertical displacements in soils reinforced with lime-cement columns. Furthermore, the effects of swelling on the total vertical displacements will also be studied.

1.5 Method

A literature study was made to learn more about geotechnical investigations, lime-cement columns as well as swelling and settlement. The choice of bridge was made in collaboration with Jan Ekström at the Swedish Transport Administration.

Data from field investigations and laboratory tests in the surroundings of the bridge on E45 over local road in Kärra was assembled from the Swedish Transport Administration. The data was used to create a hand calculation model in Microsoft Excel. This model was verified by comparison with measured data from the Swedish Transport Administration and by creating a model in the finite element program Plaxis. The input to the models was collected from the Swedish Transport Administration database from the project BanaVäg i Väst with the help of Jan Ekström.

Different models for estimating the unloading modulus for Swedish clays are used to evaluate the swelling of the soil.

Settlement calculations are done using two different methods, one considering plastic failure as described in the paper by Alén, et al. (2005) and the other not considering plastic failure. Both methods are used in order to compare the results and to do an analysis of the reliability of the results. When calculating swelling, the model used a consolidation model with modulus values adapted to swelling rather than compression. This is according to (Persson, 2004) an applicable model for use on swelling. A simulation in the finite element program Plaxis have been made. Input data about the soil properties have been taken from CRS evaluations done by WSP consulting company.

2 Background

This chapter will present relevant background theories relating to the objectives of this report.

2.1 Ground improvement

The clays in the Gothenburg area have been a problem for the construction industry for a long time. Settlements and stability issues are problems that have to be coped with daily. One way to deal with the problem is to use ground improvement methods before the construction is executed.

The method of focus in this report is lime-cement columns. This method is used to increase the strength of the soil as well as transfer parts of the load further down into more firm layers of soil.

2.2 Lime-cement columns

In situ mixing of lime or lime and cement into soft soil is a common ground improvement method in Sweden. The aim with this type of ground improvement is mainly to increase the stability and to reduce settlements. It is commonly used method to improve the ground under road and railroad embankments. The effectiveness of this method has been developed significantly during the years since it was first introduced (Broms, 2004).

Lime-cement columns can be used in organic soils where lime alone is not effective. Lime columns, on the other hand, have a higher permeability and ductility compared with lime-cement columns. As the ground temperature is increased during slaking of the lime introduced into the soil, the water content is reduced and the shear strength is increased significantly (Broms, 2004).

2.2.1 Dry and wet mixing

A lime cement column is a column created by mixing lime and cement into the natural soil in-situ. Two methods that are used when creating lime-cement columns are wet mixture, which includes water, and dry mixture, without added water.

Mixing lime-cement columns are done in five steps, see Figure 2-1. (1) First you make sure the mixing tool is correctly positioned. (2) The mixing tool is driven down to the wanted depth by rotation. (3) When desired depth is reached the tool is pulled towards the surface. At the same time the binder is injected into the soil. (4) The mixing tool is rotated when moving towards the surface and the binder is mixed with the soil. (5) Finishing the column and pulling the tool out of the ground.

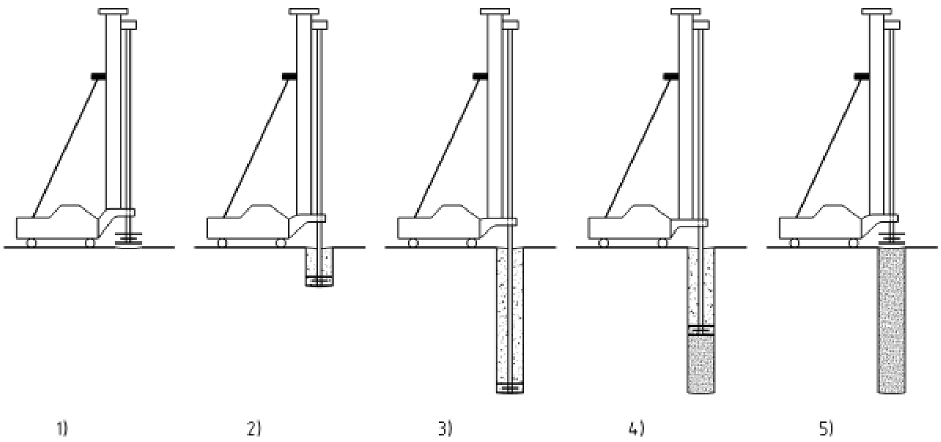

Figure 2-1 Mixing process (British Standard, 2005)

Dry mixing is normally a mixture of dry unslacked lime and cement, and air pressure is used to carry the mixture into the ground (Rydell, et al., 2005).

The two most commonly used techniques for dry mixing is the Nordic technique and the Japanese technique (British Standard, 2005).

The Nordic Technique uses machines able to install columns to 25 meters depth. The columns are usually between 0,6 meters to 1 meter in diameter and can be installed in an angle of 70° from vertical orientation. The machine uses one mixing shaft with the outlet at the mixing tool. The mixing tool is driven down to the desired depth and the binder is pushed into the ground through the outlet at the mixing tool. The mixing tool is rotated and pulled towards the surface, with controlled rotation and speed, while the binder is continuously pushed out of the outlet with a controlled pressure.

In the Japanese dry mixing method a machine with either one or two shafts can be used. The columns can be installed to depths up to 33 meters and are mixed with several blades on each shaft. The diameter of the columns can be between 0,8 meters to 1,3 meters. The binder is pushed down by compressed air into the ground. The outlets are positioned both above and below the mixing tool. The tool is driven down to desired depth and the binder is pushed out with a controlled pressure. The tool is then subtracted with a controlled speed as the soil and binder is mixed. The binder is continuously pushed out of the nozzle and mixed with the soil during the retraction.

In wet mixing water is usually mixed with the lime-cement mixture and then carried in slurry form into the ground (Rydell, et al., 2005).

The two most common techniques for wet mixing are the European technique and the Japanese technique.

The European technique of wet mixing is carried out by either a Flight auger or by the blade mixing method described under dry mixing. The material pushed down into the ground is now however wet and the pressure are built up with water. Into this slurry steel bars or other reinforcements can be inserted to increase the strength of the columns and the reinforced soil.

The Japanese technique of wet mixing is very similar to dry mixing. However the material pushed into the ground is now wet. The column diameter can now increase and have a span of 1 meter to 1,6 meters. The columns can be driven to 48 meters depth. When used as a marine reinforcement method the maximum depth of the sea level is 70 meters (British Standard, 2005).

The undrained shear strength is usually higher in soil stabilised with dry method due to the reduction in water content of the soil (Broms, 2004). According to Broms (2004) the undrained compressive strength of silty clay became 1.7 to 3.2 times higher when using dry method compared to wet method; however this depends on the in-situ water content of the natural soil. The drawback with dry method is difficulties getting an even mixture in the soil, especially if the soil has a low water content and relatively high initial strength. Broms (2004) explains that the friction angle of lime-cement columns is large even at undrained condition due to the fact that the stabilized soil is not fully saturated. Karstunen (2014) however, explains that the friction angle of lime-cement columns is not studied enough to be able to make reliable predictions of the friction angle. The bearing capacity is to a large extent governed by the confining pressure in the soil as well as the axial loading of the lime-cement column (Broms, 2004).

The nozzle is bored into the ground by rotating a mixing tool. When the desired depth is reached the nozzle is retracted. During the retraction the lime-cement mixture is pushed out through the nozzle and into the ground under rotation of the mixing tool. When the surface is reached again the column is done. After the mixing it needs time to set to a firm column (Rydell, et al., 2005).

2.2.2 Mixing tools

There are many tools that could be used for column installation. The most common ones are the tool called "Pinnborr" and the tool called Standard mixing tool (Haglund & Nilsson, 2001).

The tool called "Standard" Mixing tool is a tool that has a half circular loop, see Figure 2-2. The "Pinnborr" tool has one rotation level that mixes the lime-cement mixture with the soil, see Figure 2-3. The visible holes in the tools are used to deliver the lime-cement mix into the ground.

Figure 2-2 "Standard" mixing tool (Haglund & Nilsson, 2001)

Figure 2-3 "Pinnborr" with 2 blades and one rotation level (Haglund & Nilsson, 2001)

2.2.3 Undrained shear strength

The short-term increase of undrained shear strength depends mainly on the chemical reactions of the slaking of unslaked lime that reduce the water content in the soil when mixed with lime-cement. Therefore, the short term effects are connected with a reduction in water content. Long-term effects on the other hand depend more on the pozzolanic reaction, the reaction of the lime-cement with the clay particles in the soil. The shear strength of reinforced soil, and the deformation behaviour, is similar to over consolidated partially saturated soil (Broms, 2004).

Due to the high friction angle, $\emptyset_{u,col}$, of stabilized soil, triaxial tests often show a higher shear strength than in situ shear vane tests (Broms, 2004). The undrained shear strength of the columns, $\tau_{fu,col}$, can be calculated as the sum of the undrained cohesion $c_{u,col}$ and the total pressure σ_f , acting normal to the failure plan, multiplied with tangent of the undrained friction angle $\emptyset_{u,col}$, see Equation (2.1).

$$\tau_{fu,col} = c_{u,col} + \sigma_f tan\emptyset_{u,col} \tag{2.1}$$

The average friction angle of 50/50 lime-cement columns has been estimated somewhere between 30° to 45° (Broms, 2004).

2.2.4 Permeability of lime-cement columns

It can be hard to get an accurate measurement of permeability of the soil in general from small lab samples. This is because of the lack of larger cracks that appear in-situ in the soil and the columns. On the other hand, cracks may appear in the sample of the lime-cement column during the extraction from the ground or the handling of the sample,. The sample will most probably become disturbed. A method to determine in-situ permeability is to measure excess pore water pressure in the columns or back calculate the permeability from measured rate of settlement of the constructed embankment etc. The permeability usually decreases with time (Broms, 2004).

A common notion in Sweden is that the permeability of lime-cement columns is considerably higher than in the surrounding soil. The permeability of lime-cement columns is suggested to 400 to 800 times the permeability of the surrounding soil. This value has appeared to be too high when compared with field tests. The reason for this is probably related to the hydraulic lag that becomes large when the length of the columns is large. The in-situ permeability of lime-cement columns of 8 meters depth, underneath a test embankment at Norrsala, Sweden was found to be roughly the same as the surrounding soil. The excess pore water pressure in this case had dissipated in the 6 meters deep clay layer below the embankment. The recommended estimation of the permeability for lime/cement columns is 40 times the surrounding soil (Broms, 2004).

2.2.5 Installation effects

It has been shown that some theoretical values underestimate the bearing capacity of lime-cement columns compared to laboratory- and field tests (Shen, et al., 2003). It has been suggested by Shen, et al. (2003) that the clay surrounding the column is affected by disturbance from the rotation and from induced pressure during installation. The affected zone is called the plastic zone. The plastic zone may vary depending on shearing force and induced pressure.

The soil is according to Shen, et al. (2003) negatively affected with respect to shear strength in an early stage after installation of lime-cement columns but after about ten days, the clay is restored to strength values exceeding the original value. The strength continues to increase over a period of 70 days. Shen, et al. (2003) explains that this has partly to do with a consolidation process due to the dissipation of induced pore-water pressure caused by the rotation of the mixing tool and injection of lime-cement during the installation.

Shen, et al. (2003) noticed four different factors affecting the property of the soil surrounding the lime-cement columns. These four are soil fracturing, disturbance, thixotropy and consolidation, and cementation effects caused by diffusion of the chemical agents.

Soil fracturing derives from two mechanisms, tensile failure and shear failure. Tensile failure occurs when the effective stress becomes negative with such magnitude that the tensile strength in the soil is exceeded. Fracturing can occur in both horizontal and vertical directions. Tensile fracturing may in most cohesive soils occur in the plastic zone surrounding the column. Shear failure is derived from the rotation of the mixing tool. When the blades rotate they create shearing forces on the columns cylinder wall. Soil fracturing may occur due to the two mechanisms individually or the mechanisms combined (Shen, et al., 2003).

Soil is when mixed disturbed by two forces, one created by expansion of the injected column and one created by the rotation of the mixing blades. These two forces can lead to excess pore-pressures and the formation of the plastic zone around the column. The induced pore pressure is very large but decreases with distance from the column. A large injection pressure contributes to a large induced excess pore-pressure (Shen, et al., 2003).

Clay fracturing is a big factor in property change in the clay surrounding the lime-cement columns. The created fracture channels create drainage paths increasing the rate of consolidation in the clay. Also due to the fracturing in the soil the Cation concentration is spread faster than from diffusion (Shen, et al., 2003).

The pore pressure in the surrounding soil is increased immediately during installation of stone columns with vibrator penetration. The pore-pressure dissipated a while after the installation. Effects such as horizontal stresses and remolding of the surrounding soil were also shown (Gäb, et al., 2009).

2.2.6 Cost

When a soil needs a moderate improvement to reduce settlement or improve stability lime-cement columns is a cost effective reinforcement. It is more cost effective to use a large diameter column than a small diameter. The high torque required to rotate the equipment limits the diameter of the lime-cement column. The high air pressure needed in dry method to evenly distribute the lime-cement mix is also a restraint for the maximum diameter. It has been shown that the cost of using lime columns instead of piles for stabilizing embankments in Sweden is only one third. Lime-cement is usually somewhat more costly than lime columns (Broms, 2004). The price of lime and lime-cement stabilization does of course vary with the current prices for lime and cement.

2.2.7 Settlement reduction

In the 1970s, soil stabilization was mainly focused on reducing uneven settlements causing angular rotation of relatively light constructions, such as houses. Normally consolidated clay or slightly over consolidated clay usually have enough strength to carry a two-storey building without basement without problematic settlement issues. Buildings are usually good at handling large settlements without suffering damages, as long as the settlement is uniform underneath the building. The problems arise when deformation settlement occur leading to rotation, larger on one side than the other. Damage often occurs when the angular rotation becomes greater than 1/300 or the relative deflection is above 1/2000 to 1/4000. The settlement in normally consolidated clay of great depth can usually be reduced with up to 75 per cent or more with 20-meter long lime-cement columns. It is important to remember that the columns may fail if the soil layer directly below the column has higher compression strength than the column. The column may lose much of its original reinforcing properties if this happens. It is therefore preferable to avoid installing the lime-cement columns all the way down to a firm layer. The depth of the columns underneath an embankment is often varied in order to take advantage of the increasing shear strength and compression modulus with depth (Broms, 2004).

2.2.8 Column modulus

The E-modulus of the columns can be estimated to 50-100 times c_{uk}. The value for an organic soil would be 50 times c_{uk} while columns in silty clays would have an E-modulus of about 100 c_{uk} (BRE, 2002).

According to (Swedish Traffic Administration, 2011) the E-modulus of the lime-cement columns can be described by Equation (2.2) where c_{krit}, which is the maximum shear strength of the lime-cement columns allowed for calculation purposes, can have a maximum value of 150.

$$E_{col} = 13 * c_{krit}{}^{1.6} \tag{2.2}$$

2.2.9 Quality control

Quality control is an important step in the construction procedure to ensure the quality of the columns installed. This is usually done in two steps, a general control and an additional control. The general control is performed to ensure that the columns were constructed according to the documents. This involves testing some columns to failure. The additional control is a control that is done by object specific monitoring. Included in this control are monitoring of settlement, movement, pore pressure, continuity and firmness of the columns (Hedman & Kuokkanen, 2003).

Tests that include pushing tools into the columns, taking samples for lab testing or failure loading cannot be done on the actual lime-cement columns that are supposed to be used for reinforcement of the soil. It is therefore common to install testing columns in an area close to the construction area (Hedman & Kuokkanen, 2003).

2.3 Consolidation and Swelling

Vertical deformation of the soil surface due to adding or retracting load is called settlement. It can either be a downward deformation or an upward deformation. Settlement as a word is in this report used to describe a downward movement whereas the upward movement is called swelling or if caused by frost or lime-cement column installation, heave. Heave from installation of the columns is also creating strain between the columns that can later contribute to the settlements.

Settlement consists of three parts, immediate settlement, consolidation settlement and creep settlement, also called secondary compression. The immediate settlement is represented by, in theory, an elastic settlement whereas consolidation and creep are assumed to be permanent to some part and retractable to some part, for example through swelling.

Terzaghi's principle is often used to calculate the stresses in the soil. The theory is built on that the total stress in the soil is a sum between the stresses carried by the soil skeleton, called effective soil stress, and the pore pressure.

2.3.1 Consolidation

As explained earlier soil is a material with a non-conservative function. This means that the material has a memory. When soil is compressed some of the deformations remain permanent (Holtz & Kovacs, 1981).

Consolidation is a process that occurs when a saturated soil is loaded with additional pressure, for example a house, an embankment or some other surplus load. When the soil wants to compress, but due to low permeability the water is not able to flow out immediately, an increase of pore pressure will develop. The increased pressure will want to equalize throughout the soil. The liquid in the pores is slowly pushed out of the soil. When this process occurs, gradually the particles are pushed closer and closer to each other and this will be visible through settlement on the ground level (Holtz & Kovacs, 1981). In 1D situations the process is dependent on the coefficient of consolidation, c_v, the confined modulus, M, the added load, $\Delta\sigma$, pre-consolidation pressure, s'_c, drainage distance of soil, d, thickness of the layer where the settlements will occur, H, time, t, the permeability, k, and the pore pressure, u_0, (Persson, 2004).

Consolidation is a slow process, and the rate of settlement is largely dependent on the permeability and the compressibility of the soil. The longer you wait the more time do you give the pressure in the pores to equalise and the larger part of the total settlement will have occurred. This means that consolidation is a stress-strain-time dependent progression (Holtz & Kovacs, 1981).

2.3.2 Swelling

Swelling can be seen as a reversed consolidation (Persson, 2004). The same formulas can be used in swelling calculations, as in regular consolidation calculations. The same parameters that are used in consolidation calculations are used in swelling calculation, with some modifications. The confined modulus, M, used in this case is, the unloading modulus M_{ul} instead of M_L or M'. The coefficient of consolidation c_v will be dependent on the unloading properties of the soil, and will therefore be called c_{vs}, see Equation (2.3), when concerning swelling and the unloaded weight will give cause to a change in pressure called $\Delta\sigma_{ul}$.

$$c_{vs} = \left(\frac{k * M_{ul}}{\gamma_w}\right) \tag{2.3}$$

k is the permeability, γ_w is the unit weight of water.

Different clay minerals have different swelling potential (potential to absorb water and swell into a larger size). The swelling potential is dependent on the activity of the mineral and the clay content. For example Na-Montmorillonite has an activity of 7.2 whereas the activity of Illite is 0.9 (Larsson, 2008). Since Swedish clays contain mostly Illite the calculations should be done with regard to its swelling potential

(Nayak & Christensen, 1970). These calculations with regard to Swedish conditions are found in Persson (2004).

2.4 Field investigations

Field investigations are tests done in the ground in-situ. There are many different variations of tests and versions of the same test. Below follows some of the most usual tests and tests done in the field investigations of the project by Vägverket.

2.4.1 Field vane test

The field vane test, FVT, is used to determine the undrained strength of clays. The test is suited only for fine grain soils, particularly clays. If used on silts, organic clay and glacial tills the reliability of the result may vary and the test should be backed up with data from other methods. The test is especially useful for determining the shear strength of soft clays, which are often hard to determine in labs because of the samples often being disturbed (Knappet & Craig, 2012).

FVT test are usually only used in clays which have a c_u < 100 kPa. The reliability may be affected in a negative way if the clay is laminated with sand or silts. The test is governed by technical standards, EN ISO 22476, Part 9 for EU and ASTM D2573 in US. The equipment consists of a stainless steel vane attached to a steel rod enclosed in a sleeve packed with grease to reduce friction. The vane is basically four thin rectangular plates arranges in 90 degrees to each other, see Figure 2-4. Those plates are fixed to the high tensile strength steel rod. The vane and the rod are driven into the ground at the bottom of a borehole, a minimum of three times the diameter of the borehole. In soft clays, no borehole is needed; the vane can be pushed directly from ground level. In that case, the equipment needs protection in form of a shoe on the bottom of the vane (Knappet & Craig, 2012).

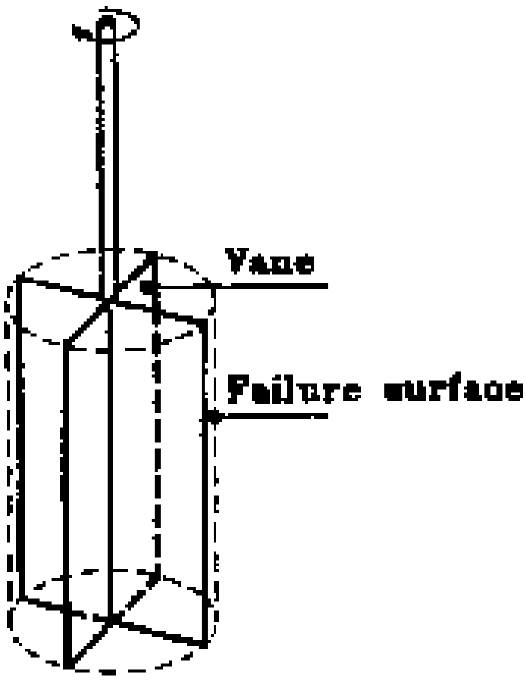

Figure 2-4 Field Vane Test (Information Technology in Construction, u.d.)

Torque is applied gradually from a rotational drive unit connected to the top of the rod. The rotation speed should be between 6-12 degrees per minute. The level of torque required for the soil to fail is used in Equation (2.4) to calculate the shear strength of the soil.

$$\mathrm{T} = \pi\, \mathrm{c_{uFV}} \left(\frac{{d_{FVT}}^2 h_{FVT}}{2} + \frac{{d_{FVT}}^3}{6} \right) \tag{2.4}$$

T is the torque required for soil failure, $\mathrm{c_{uFV}}$ is the undrained shear strength from field vane test, d_{FVT} is the overall vane width and h_{FVT} is the vane lenth.

Shear failure of the clay takes place along the area of the cylinder created by the rotating vane. The shear strength may vary from the sides to bottom and top of the cylinder due to anisotropy in the clay. The shear strength is usually determined at different depths. The vane may also be rotated several times to remould the clay and get the shear strength c_u in the remoulded state in order to have an idea the sensitivity of the clay (Knappet & Craig, 2012).

2.4.2 Cone penetration test

The cone penetration test is done by pushing a cylindrical element, which at the tip is shaped as a cone, into the ground with a specific rate of penetration. A load cell situated between the cone and the body of the instrument is collecting data on the resistance when pushing the rod into the ground. (Knappet & Craig, 2012)

This tool is very versatile as it can be used not only to describe the soil layer sequence but also to determine specific parameters in the soil. The more sophisticated tool Piezocones (CPTU) can give an even more accurate result by also measuring the pore water pressure in the soil as the rod is pushed down into the ground. The tool consists of a cone piece at the end that has two thin layers of permeable material, a friction sleeve and one more permeable layer before the rod attaches to the tool, see Figure 2-5. The permeable layers measure different parameters depending on the placement of the layer. For example the layer u_2, seen in Figure 2-5, can be used to measure the over consolidation of the soil. If the soil is heavily over consolidated the pore pressure of this layer will show negative values. (Knappet & Craig, 2012)

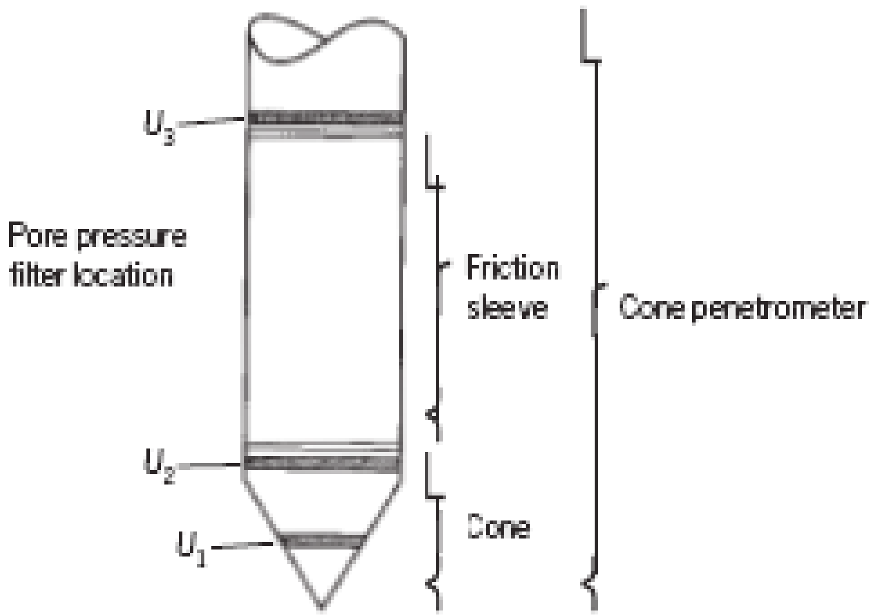

Figure 2-5 Piezocone (Brouwer, 2014)

The CPT test has the advantage of being a quick test that is also cheap and does not leave any large voids in the ground. However the interpretation of the result can be quite difficult if no knowledge of the ground conditions exists and the method is therefore most accurate when used to measure the soil data in the close counter of a soil sample and a vane test (Knappet & Craig, 2012).

2.5 **Laboratory tests**

Some soil tests are too complicated to perform in the field. These tests go under the category laboratory tests and are performed on a sample of soil obtained from the site of interest. It is important that the soil samples are carefully extracted from the soil to reduce sample disturbance which will affect the results obtained from the lab tests.

2.5.1 The oedometer test

The oedometer test is done in order to determine the one-dimensional consolidation or swelling properties of a soil sample. The test is carried out by placing the soil sample, of cylindrical shape, in a confining ring with a porous stone disc on either side. The lower stone is then fixed against the steel ring, while the upper stone, of smaller diameter, is free to move down inside the ring as pressure is applied. The steel ring should be well polished on the inside to reduce friction. As weights are applied the top stone puts pressure on the soil specimen and either a dial gauge or an electronic displacement transducer measures the compression. These components are usually connected to the top stone. The steel ring and both stones sit in an open cell of water so that the soil specimen has free access to water. (Knappet & Craig, 2012)

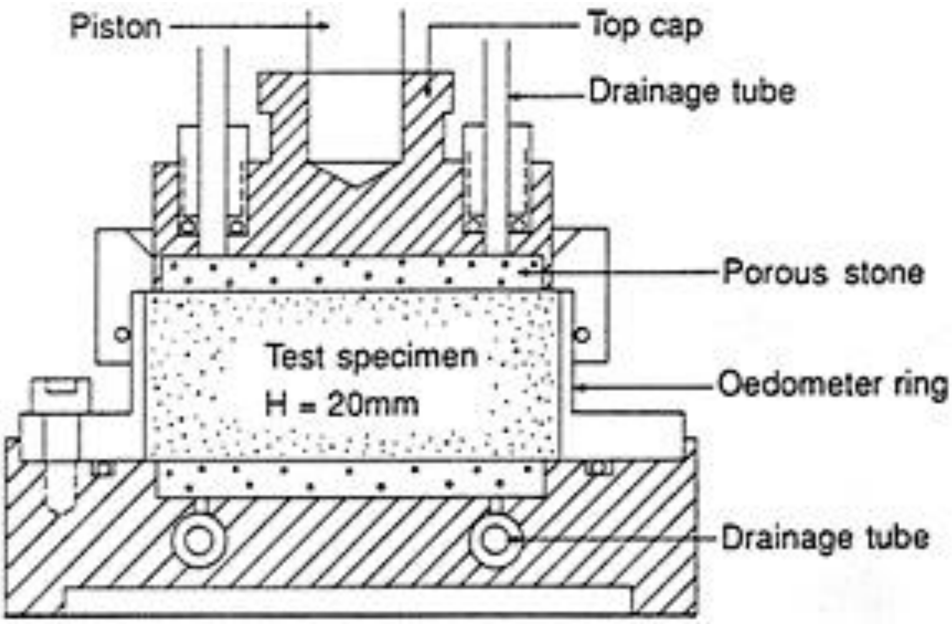

Figure 2-6 Schematic drawing of oedometer (NGI, 2014)/

In Europe, the test procedure is standardised in CEN ISO/TS 17892-5. The soil sample is put under pressure by sequenced addition of weights, each new weight being twice the previous weight. For every sequence the apparatus is left to rest for a period of at least 24 h to allow pore water pressure to dissipate. The effective vertical stress in the soil will then be equal to the applied total stress. The result is then presented as a plot comparing the applied stress to either the thickness of the soil specimen or the change in thickness or the void ratio. The plot is usually done on a logarithmic scale. The expansion of the soil, during unloading of weights, can be measured to obtain the swelling characteristics of the specimen. The void ratio can be calculated at the end of each loading sequence from displacement readings and water content or the dry weight of the specimen. According to Eurocode 7 at least two tests should be done in a given soil stratum. In the case of discrepancy in displacement between the results of these two tests, two more tests should be carried out (Knappet & Craig, 2012).

2.5.2 CRS test

The CRS lab test is popular in Sweden. The name is a short for Constant Rate of Strain and the test can be described as a type of oedometer test in which the sample is deformed with a constant rate of displacement. The sample is set up in a cell with drainage on one side only and a pore pressure meter on the other side, the undrained side. Applied force, deformation and pore pressure are then measured as the sample is put under stress. The CRS test gives information about vertical effective stress and compression. The pre-consolidation pressure can be estimated with, for example, the Sällfors method, which is a graphical method. The compression modulus variation with effective stress is also given by the CRS test. The compression modulus up to the pre-consolidation pressure is called M_{0}, and assumed constant, and the maximum modulus that takes over after the pre-consolidation pressure is exceeded is referred to as M_{L}, also assumed constant. After a further increase of stress, the modulus starts to increase with every increase in effective stress. The modulus is in this state following Equation (2.5).

$$M = M_L + (\sigma' - \sigma'_L)M' \tag{2.5}$$

By knowing pore pressure and rate of deformation, the permeability can be calculated. CRS tests take less time to complete then the oedometer test as it can be done in 24 hours. Compared with the standard oedometer test, which tends to give too low values for the permeability compared to in-situ tests, the CRS test giver higher and more reliable values for the permeability (Sällfors, 2009).

In Sweden it is common practice to use CRS tests instead of the standard oedometer tests to obtain the compression modulus. CRS tests are quicker to perform, but the oedometer test has the advantage that it is possible to get the true unloading modulus. This is because of the measuring procedure by stepwise addition of weights for pressure. Therefore unloading of weights reduces the pressure on the specimen, allowing the soil to swell. Measuring the swelling gives the unloading modulus. This is not possible with a CRS apparatus since it works by deforming the soil with a constant rate. Backwards running of the test would hence create a gap between the soil specimen and the loading cap (Sällfors, 2009). Therefore, for problems involving swelling of clay, it is important to perform either a standard oedometer test or a K_0 triaxial test. Either of these two allows the swelling of the clay to be calculated since they both describe the unloading-reloading modulus.

2.6 Calculating settlement with Chalmers method

In this method the soil is divided into three zones, A, B and C, see Figure 2-7. The upper zone A is a transition zone where the applied weight is large enough to bring the columns into plastic failure state. Below this, in zone B, the columns are intact and assumed to deform equally to the surrounding soil. The columns and soil in this zone can be viewed as a homogenous composite material in which plane sections will remain plane. Zone C represents the area of soil underneath the columns down to the bedrock (Alén, et al., 2005).

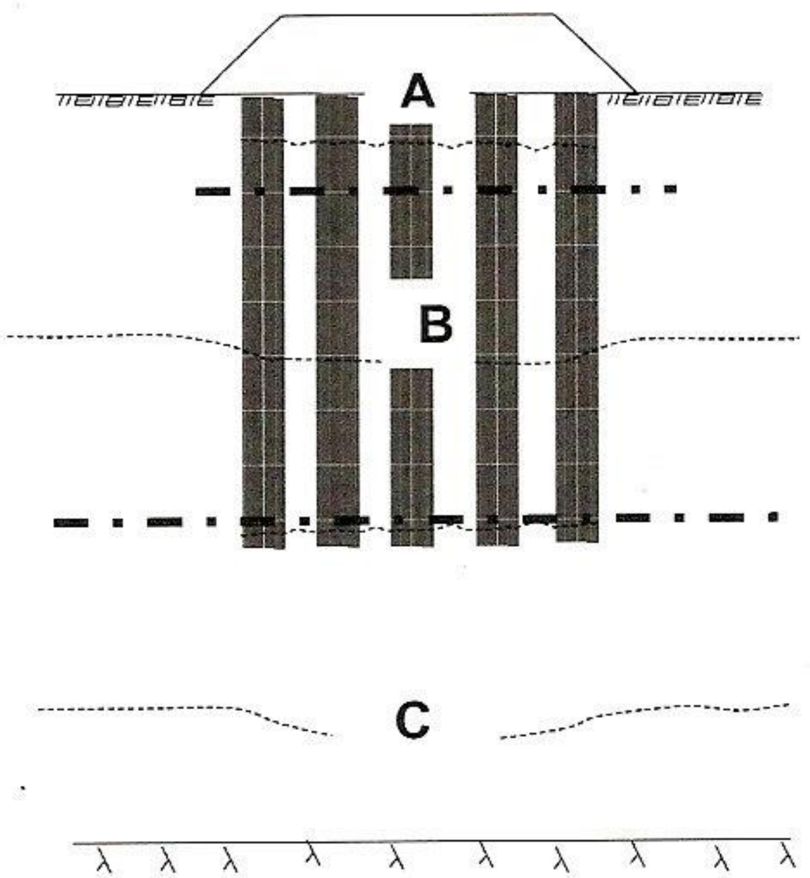

Figure 2-7 Division of zones (Alén & Sällfors, 2012)

Zone A, in this zone the compression strength of the columns are exceeded by the applied load, hence the columns are in a plastic failure state.

Zone B, in this zone the strength of the columns are sufficient to carry the load and the soil and the columns can be treated as a composite material. The properties of this composite material can be calculated by using weighted soil and column data. The ratio depends on the relation between soil and column volume in the studied soil.

Zone C, in this zone normal soil properties and settlement calculations are used. This zone stretches from the end of zone B (the bottom of the columns) down to firm ground. In case of columns going all the way down to firm ground, this zone is ignored.

First the action effect of the applied load should be determined as well as the stress distribution with depth. Thereafter the resistance of the reinforced soil is determined. The action effect is then compared with the resistance and the depth at which the values are equal is the border between zone A and zone B, the neutral plane, see Figure 2-8.

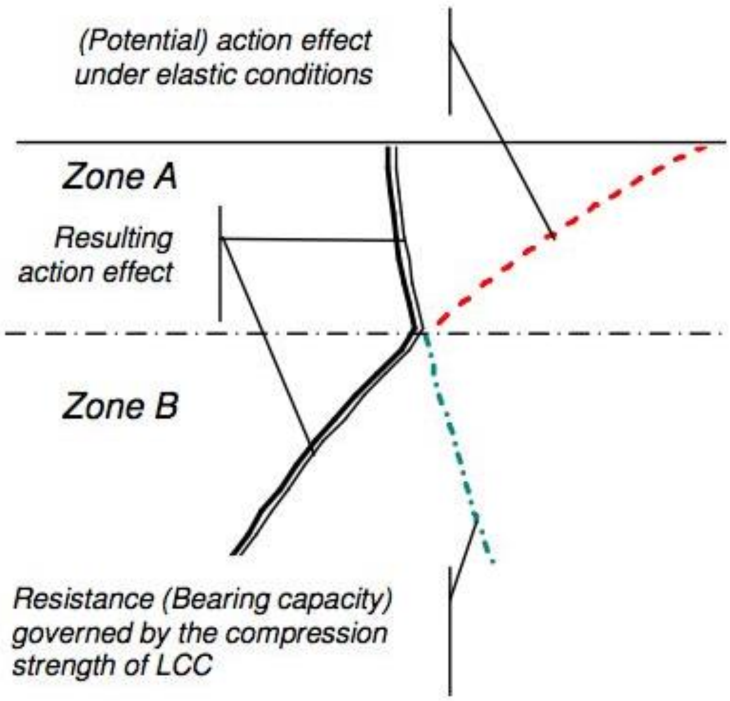

Figure 2-8 Neutral plane (Alén & Sällfors, 2012)

2.1 Simulation in Plaxis

Plaxis is a finite element program used for simulation of soil behaviour under circumstances defined by the user. Plaxis is available for 2D simulations as well as 3D simulations. In this report the name Plaxis refers to the 2D version of the program.

The model used in this report to simulation the behaviour of the reinforced soil underneath the bridge is a single column model. This means that the ambition is to simulate the behaviour of a single column and its surrounding soil, which is applicable for considering large area loading of multiple columns (Vogler & Karstunen, 2008).

The model exploits the axial symmetry of the problem; hence only half the equivalent diameter of the model needs to be plotted. In the axisymmetric analysis, the plotted model is rotated around the y-axis and the forces plotted in the window act on one radian of the circle created. Therefore, calculated forces should be multiplied with 2π to get the total forces. Other output is presented per unit width instead of per radian. The pressure for instance does therefore not need to be multiplied with this factor.

The equivalent diameter, D, of the model, is dependent on the centre-to-centre distance between columns in the area. The area created by the circle with radius $D/2$ should be equal to the area of the centre-to-centre distance squared, see Equation (2.6). Solving Equation (2.6) for *D* gives Equation (2.7). A description of the measurements can be seen in Figure 2-9.

$$\frac{\pi * D^2}{4} = c^2 \tag{2.6}$$

$$\mathrm{D} = 1.13\mathrm{c} \tag{2.7}$$

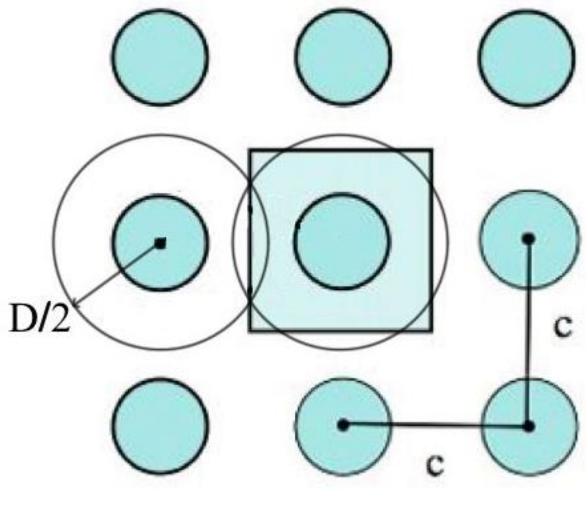

Figure 2-9 Centre to centre distance of columns

2.1.1 Soft Soil material model

In contrary to the more basic Mohr Coulomb model, the Soft Soil model is an advanced model that considers the difference in behaviour of the elastic and plastic state of a soil. Failure of the soil in this model still works in accordance to Mohr-Coulomb. The stiffness parameters for soil used in Soft-soil is called modified compression index (λ^*) and modified swelling index (κ^*) and can be obtained from an oedometer test. λ^* represents the slope of the compression line (after pre-consolidation pressure is exceeded) and κ^* represents the unloading reloading part of an oedometer curve, see Figure 2-10.

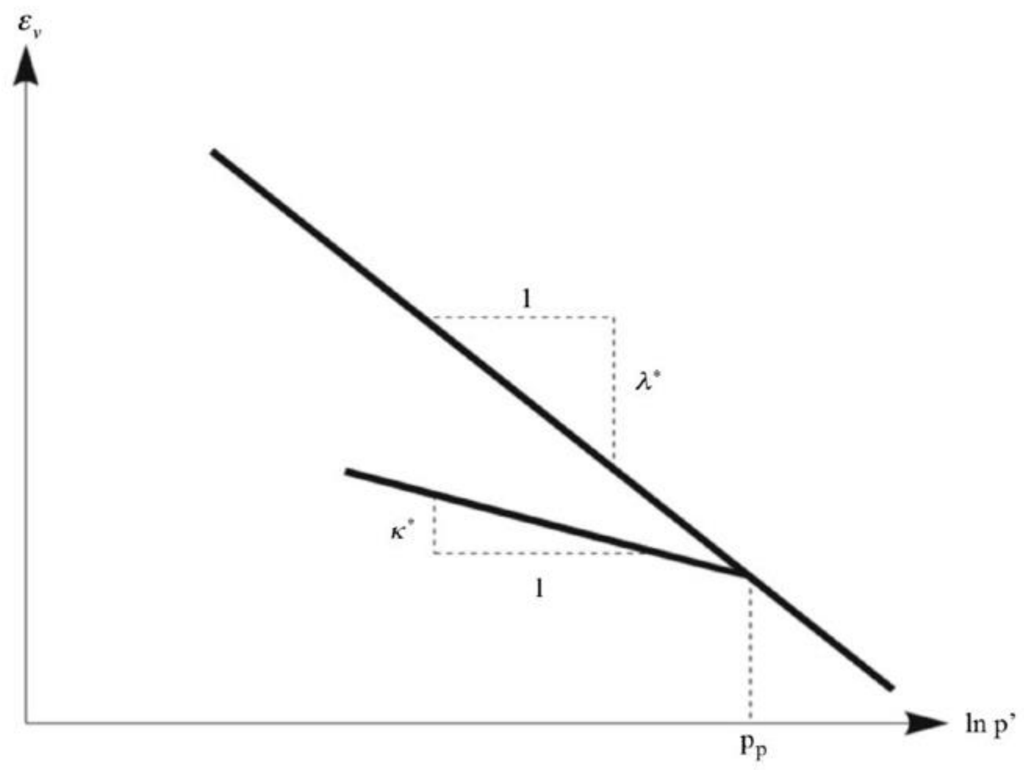

Figure 2-10 Modified compression and swelling modulus

2.1.2 Undrained and drained conditions

In Plaxis, it is possible to choose if the course events will be calculated as undrained or drained. As a rule of thumb, drained condition should be used if the permeability of the soil is great, whereas undrained condition is used when the permeability is low. However drained condition can be useful regardless of the permeability if only a crude approximation of the final and total amount of settlement is of interest and no attention to excess pore water pressure is required. Since no time is needed for pore water pressure to dissipate, the settlement takes place instantaneously, in contradiction to undrained condition.

When a load is applied to a saturated soil body, an increase in pore water pressure will arise. In undrained conditions, the excess pore water pressure will need time to dissipate. The time needed depends in a large extent on the permeability of the soil. The process of soil compression during dissipation of pore water pressure is called consolidation. Consolidation is therefore a mechanism related only to undrained conditions in Plaxis, as only the excess pore pressure can be predicted. With undrained condition and given permeability of the soil, it is possible to assess the expected settlement after a given time.

3 Idealization of area

To determine the expected settlement and the necessary ground improvement on the site, a simplification of the area must be made in order to create a mathematical model. This idealization should be fairly detailed without presenting unnecessary amount of data. Therefore a two dimensional model is often used, with mainly horizontal soil layers. This gives a clear picture of the soil so that the engineer can determine what problems might arise.

3.1 Material Properties

The soil on site needs to be tested in order to determine its geotechnical properties. If any ground improvement is going to be used, for example lime-cement columns, the properties of these are also needed. Site investigations like field vane tests can be used to determine the shear strength of the soil at various depths but complimentary lab tests are also needed. If an oedometer test is applied, the unloading and reloading modulus may also be determined. A CRS test is commonly used to determine the constrained modulus of the soil.

In order to determine the properties of lime-cement columns a laboratory mixture of lime, cement and the soil is done. This is done in order to determine the optimal addition of lime-cement mixture to the soil. These tests are allowed to rest for 7 days before tested in strength. The strength tests are then repeated after 14 and 28 days. The minimum shear strength should not drop below 100 kPa. The hydraulic conductivity of the columns are either tested or assumed (Alen, et al., 2005).

3.2 Mathematical model

A mathematical model should be chosen to represent the reality. Sometimes more than one method is used in order to provide a safety to the calculation. If the results from the different methods have a fairly high discrepancy, it can be suspected that either an error is made in the calculation or that one of the models does not represent the reality in a good way. In such a case further investigations should be made until the results are matching.

3.3 Compression and swelling moduli

When settlements are to be calculated, it is essential to know the correct compression modulus of the soil. The compression modulus up to the pre-consolidation pressure is called M_0 and the modulus that takes over after the pre-consolidation pressure is exceeded is referred to as M_L.

The reinforced soil (soil with installed lime-cement columns) can be viewed as a composite material with a specific modulus. In this report this modulus is referred to as M_{block}. M_{block} is calculated by a weighted value depending on the area of the columns in relation to the total area.

The swelling modulus, M_{ul}, is gained from oedometer testing. If unloading-reloading tests has not been made the modulus can be estimated with different calculation models or approximated to M_0.

3.4 Rate of settlement

Different rate of settlements can be expected depending on whether the lime-cement columns acts as drain or has the same permeability as the soil. By knowing the rate of settlement, calculations of the settlement at a specific time can be made. The settlement decreases exponentially over time.

4 Case study: Bridge on lime-cement columns

The bridge to be considered, is a crossing for E45 over a local road to Kärra close to Lilla Edet, see Figure 4-1. The area around the bridge is mainly relatively flat farmland. The ground on the site of the bridge is +8.5 meters above sea level. In this chapter more of the geotechnical conditions in the area will be described. The first section contains a description of the geology in the area. In the second section the hydrogeology in the area of interest will be explained.

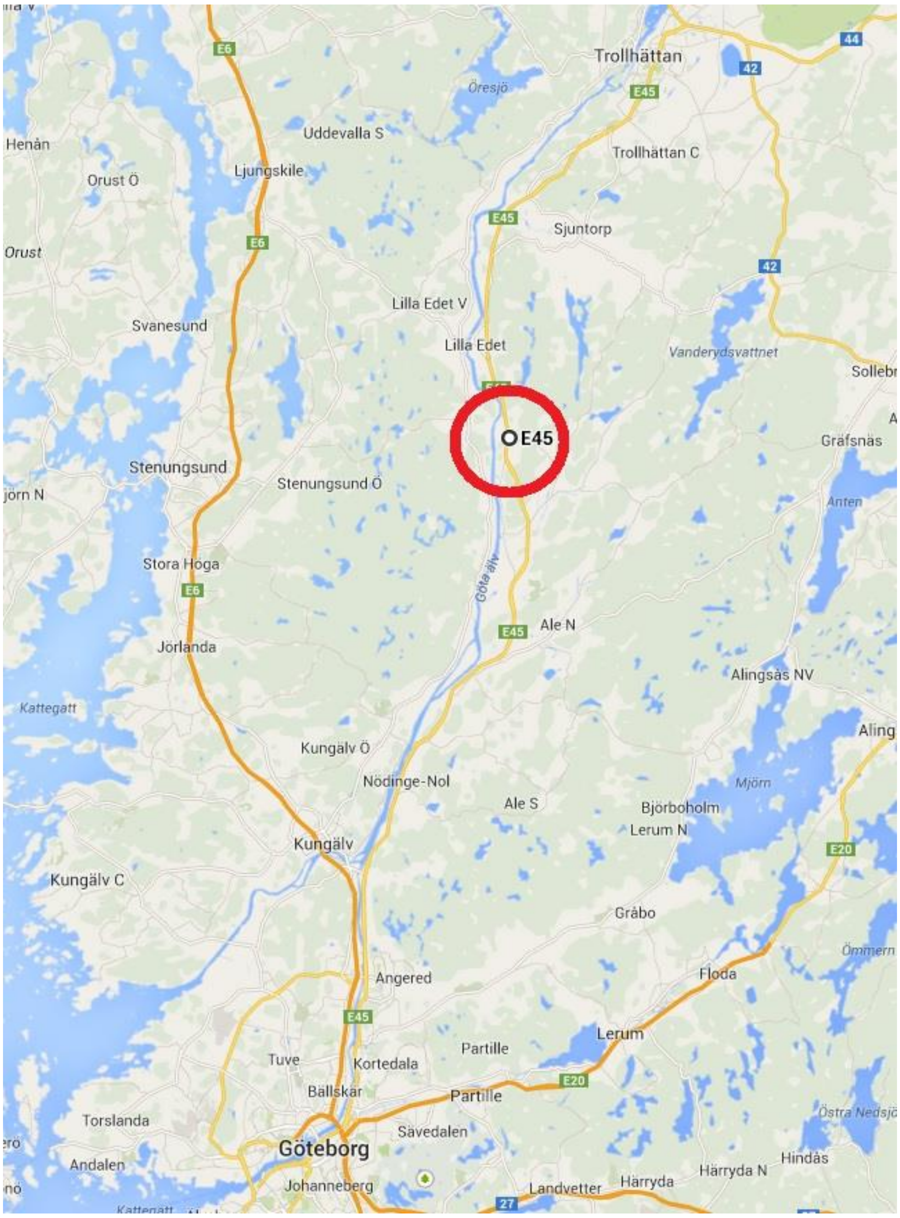

Figure 4-1 Location of bridge (Google, 2014)

4.1 Geology

The Swedish geology has been characterized by the recent glaciation. About 75 % of our country is covered with the glacial sediments such as moraine. The clay exists in the parts of Sweden that where under deep water after the ice retracted (Andréasson, et al., 2006).

Figure 4-2 shows that the area between Stockholm and Gothenburg was once under deep water. The deep water was a settlement point for sedimentation of clay particles. This is why the area of Gothenburg has such a deep clay cover (SGU, 2014).

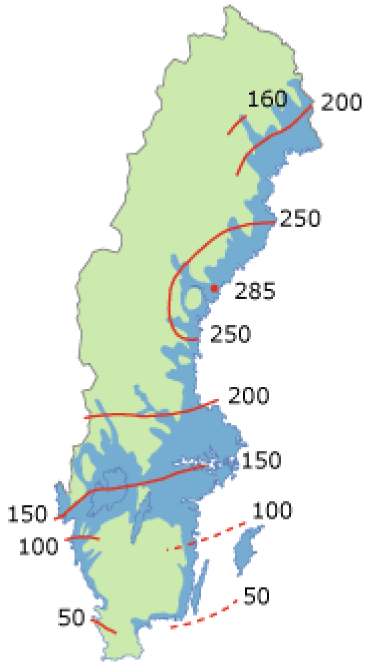

Figure 4-2 Highest sea water level reached since the last ice age (Geological Survey of Sweden, 2014)

The soil in the area of the bridge can be divided into six different zones with different attributes. The input data to the layers was taken from Sweco's field investigations. The stratigraphy can be seen below where the top layers down to 5.4 meters are removed during excavation.

The stratigraphy used in the models for calculations is:

(0-0.5 m)	50 cm of dry crust clay
(0.5-1 m)	50 cm of muddy silt
(1-6 m)	5 m of silty clay
(6-19 m)	13 m of clay
(19-35 m)	16 m of slightly heavier clay
(35- m)	Sand below 35 m

4.2 Description of bridge 15-1806 in Kärra

The bridge is built as a box shaped concrete bridge rather than supporting its weight on bridge columns, see Figure 4-3.

Figure 4-3 Bridge 15-1806 in Kärra

The weight of the bridge was calculated by using the drawing of the bridge, see Appendix 1. Measurements of each element of the bridge were taken and the total volume was multiplied with the density of reinforced concrete, 2500 $\mathrm{kg/m^3}$. The weight of the bridge was calculated to 21295 kN.

When constructing the bridge the excavation was done first. Two months after the excavation, the bottom plate was constructed on the soil and the measurements of the settlements and swelling was started. Two months after this the rest of the bridge was casted and later came the supporting structure on the sides of the bridge and the fill.

The total area of the bottom plate of the bridge is 320 m^2 and the pressure on this surface is 66 kPa. In the pressure is included the weight of the fill that might add weight to the bridge surface calculated with 2:1 method from bridge bottom to bridge top.

4.3 **Lime-cement columns**

The lime cement columns have been arranged in a pattern mixing single columns with lines of columns to reduce settlements, as well as to reinforce the slope in the proximity of the bridge. Various lengths of the columns have been used in order to take full advantage of soils increasing strength with depth. The depth of the columns varies between 5-23 m. The centre- centre distance between single columns is 1.5 m. The pattern of columns is denser directly under the bridge due to installation of rows of columns and only single columns further away. The depth of the columns is greatest directly under the bridge, to account for the additional load it presents, and it becomes gradually shorter further away from the bridge to even out differences in settlement. Additional columns have been installed to reinforce the slope west of the bridge. For an overview of the column pattern, see Figure 4-4.

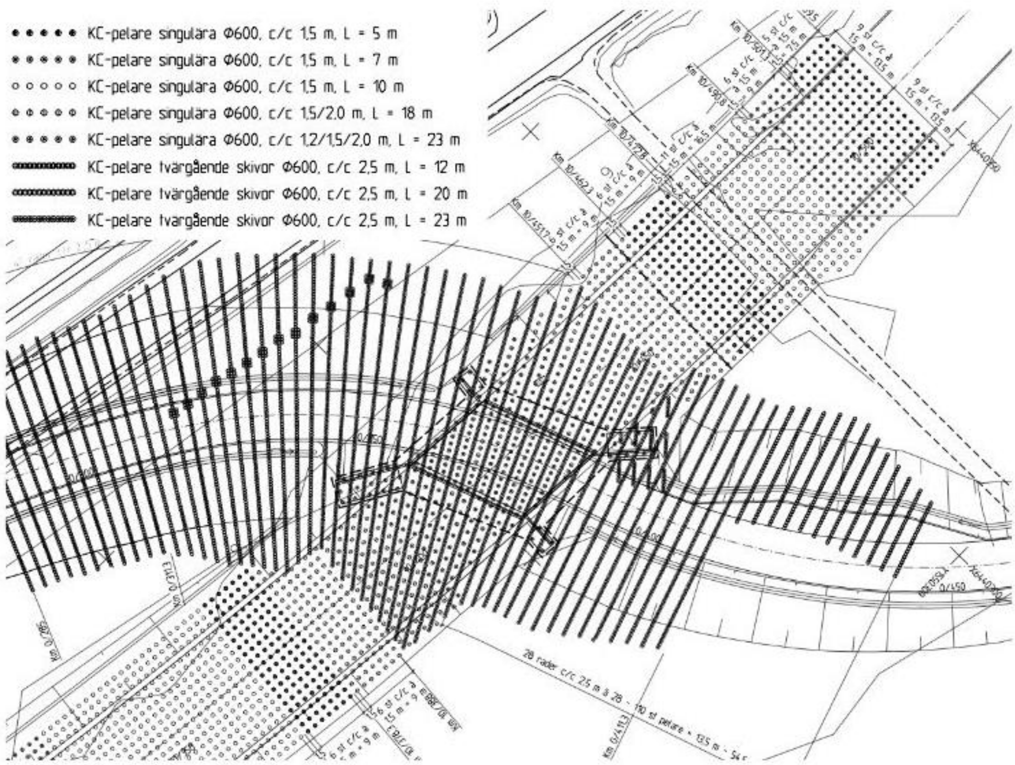

Figure 4-4 Column pattern overview (Sweco Infrastructure AB, 2010)

The mixture used in the Lime-cement columns used in the bridge of focus is 50 % lime and 50 % cement, see Appendix 1.

The shear strength of the columns, $c_{u,col}$, can be set to 100 kPa (Vägverket, 2010). The shear strength appears to be closer to about 250 kPa from in-situ tests. This data is measured on actual columns 12-16 days after construction. Since the columns break in the process, extra test columns are constructed in the area if tests are desirable (Sandberg, 2012).

The amount of lime-cement mixture is according to the drafted documents for installation instructions 26 kg/m column (Planthaber, 2010). The diameter of the columns is 600 mm and the amount of columns is about 20% of the ground area below the bridge.

5 Calculations

In this chapter the calculations and modelling made for settlement as well as swelling are described. For modelling, the software Plaxis was used. The calculations have been made as one-dimensional calculations in Excel and serve as a comparison and validation to the Plaxis model. For details of the input data see chapter 3.

5.1 Proportional area of columns

The proportional area of lime-cement columns under the bottom plate of the bridge can be described with Figure 5-1. As rows of columns and single columns are distributed evenly under the bridge, the total proportion P of lime-cement columns can be calculated as the mean value of the proportion created by the rows and the smaller proportion created by single columns, see Equation (5.1). This proportion of lime-cement column compared to the surrounding soil will be used to calculate M_{block}, the modulus used in the following hand calculations.

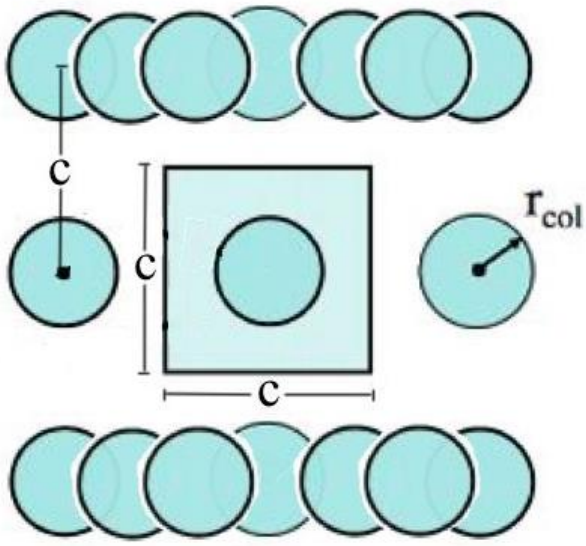

Figure 5-1 Proportion calculation of column area

$$P = \left(\frac{\frac{\pi r_{col}^2}{c^2} + \frac{2 r_{col} * c}{c^2}}{2} \right) = \left(\frac{\pi r_{col}^2}{2c^2} + \frac{r_{col}}{c} \right) \tag{5.1}$$

The bridge reinforcement is a 3D problem. The columns are installed to different depths and in patterns that cannot be described properly in a 2D model. In plane the columns turn underneath the bridge in two different directions, see Figure 4-4. Horizontal stress can be modelled in a 2D model, but only in one direction. How columns affect each other when installed, during settlement and other influence can only be modelled in a 3D model. However to be able to simulate the problem with existing means a 2D model was used. This is a huge simplification but it gives some indication of the true picture.

5.2 Hand calculations on settlement

The settlements in this report are calculated with two different methods. The first method takes into consideration different behaviour at different depths in the reinforced soil. This is done by dividing the soil into different zones. The settlements will then be calculated for each zone (zone A, zone B and zone C) and the sum of the displacements in each zone will be the total settlement. In zone A the compression strength of the columns is too low to carry the elastic proportional part of the load, driving the top parts of the columns into a plastic failure state. In zone B the soil and the columns will be viewed as a composite block with weighted parameters from soil and columns. Zone C will be calculated with normal settlement calculations for the soil. The depth of the clay layer is 35 m and the length of the columns is set to 20 m.

The other method used for calculating settlement in this report is a calculation that does not take into account the potential plastic failure of the columns. This model does only divide the soil into two different parts; one with reinforced soil and one without columns.

5.2.1 Zone A

The depth of Zone A (z_{lim}) is calculated by setting $\Delta\sigma_{\mathrm{blocklim}}$ equal to $\Delta\sigma_{\mathrm{block}}$. Part of the load will be distributed from the surface; the rest will be distributed from the bottom of the columns. A factor decides the proportion of the load is distributed down to the bottom of the columns, in this case the factor was calculated to n_{LC}=0.4, see Equation (5.2) and Equation (5.3), where H is the depth of the clay down to firm layer and D is the length of the columns. This gives a value at the top of the columns of $\Delta\sigma_{\mathrm{block}}$=q=39.6 kPa and at the bottom $\Delta\sigma_{\mathrm{block}}$=q=26.4 kPa, see Equation (5.4). Both loads are then distributed with a 2:1 load distribution; see Figure 5-2 The effective vertical stress σ'_{v_0} can now be solved from Equation (5.5). In this method a weighted value of the compression modulus, of the soil and the columns, M_{block} is used. M_{block} is calculated with Equation (5.6). A and A_{col} are described by Figure 5-3 in 5.2.2 Settlement in zone B. Equation (5.7) shows the components of σ'_{v_0}.

$$n_{LC} = (D/H)^{1/v} \qquad (5.2)$$

$$v = (M_{block}/M_{soil})^{0.1} - (M_{soil}/M_{block})^{0.1} \qquad (5.3)$$

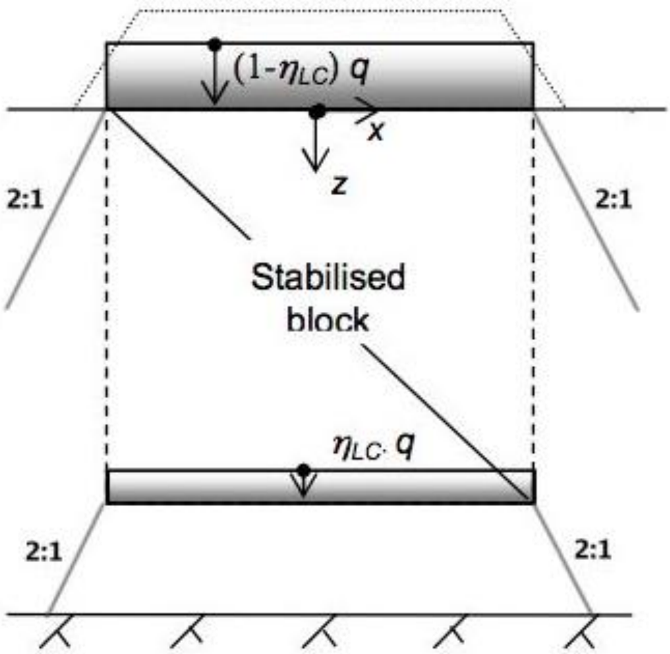

Figure 5-2 Load distribution

$$\Delta\sigma_{block} = n_{LC} \cdot q + (1 - n_{LC}) \cdot \frac{q \cdot B}{B + z} \tag{5.4}$$

$$\Delta\sigma_{block,lim} = \frac{1.5 \cdot c_{u,col} + \sigma'_{v0}}{E_{col} - 1.5 \cdot M_{soil}} \cdot M_{block} \tag{5.5}$$

$$M_{block} = \frac{(A - A_{col}) \cdot M_{soil}}{A} + \frac{A_{col} \cdot E_{col}}{A} \tag{5.6}$$

$$\sigma'_{v_0} = \gamma_{clay} \cdot z - \gamma_w \cdot (z - 1) \tag{5.7}$$

The effective vertical stress at the border between zone A and zone B is calculated with Equation (5.8). Equation (5.9) shows the effective vertical stress the top of the clay. An average value of the effective stress is calculated with Equation (5.10). The settlements for zone A is calculated by putting the average effective stress into Equation (5.11).

$$\Delta\sigma'_{soil}(z_{lim}) = \frac{M_{soil}}{M_{block}} \cdot \Delta\sigma_{block}(z_{lim}) \tag{5.8}$$

$$\Delta\sigma'_{soil}(0) = \frac{2 \cdot q - 3 \cdot a \cdot c_{u,col}}{2 + a} \tag{5.9}$$

$$\Delta\sigma'_{soil} = \frac{\Delta\sigma'_{soil}(0) + \Delta\sigma'_{soil}(z_{lim})}{2} \tag{5.10}$$

$$s_{zone\ A} = \frac{\Delta\sigma'_{soil}}{M_{soil}} \cdot z_{lim} \tag{5.11}$$

5.2.2 Settlement in zone B

The areal proportion, a, of the columns, see Figure 5-3, is calculated with Equation (5.12). Equation (5.13) is used for calculation of settlements.

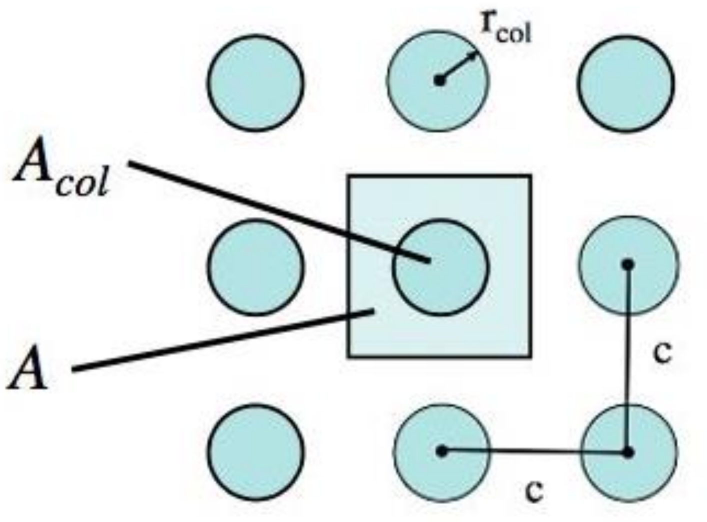

Figure 5-3 Area of columns, $A_{col} = \pi r^2$ and = c^2.

$$a = \frac{A_{col}}{A} = \frac{\pi r^2}{c^2} \tag{5.12}$$

$$S_m = S_{col} = S_{soil} = \sum \frac{\Delta h \cdot q}{a \cdot E_{col} + (1 - a) \cdot M_{soil}} \tag{5.13}$$

5.2.3 Settlements in zone C

This zone represents the clay beneath the columns and above firm ground. The calculations are performed with Equation (5.14).

$$S_{soil} = \sum \frac{\Delta h \cdot q}{M_{soil}} \tag{5.14}$$

5.2.4 Settlement calculation without plastic failure

The plastic failure of the model above does not exist in this model. However the different moduli for reinforced soil and for unimproved soil are still taken into account by using the combined modulus for column and soil, M_{block} for the reinforced soil and M_{soil} for the part underneath the columns, see equation (5.15).

$$S_{soil} = \sum \frac{\Delta h \cdot q}{\mathrm{M}} \tag{5.15}$$

Δh is the layer thickness, q is the load, M is the modulus for the specific layer.

5.3 Swelling Calculations

As described in 2.3.2, the equations for consolidation settlements can be applied for swelling in soils. For calculating the total swelling in the soil Equation (5.16) is used.

$$s = \frac{\Delta\sigma * H}{M_{ul}} \tag{5.16}$$

One way to model the unloading modulus is to use the M_0 as the modulus. This since oedometer unloading-reloading shows that these two moduli are quite similar. The value of M_0 is however expected to be larger than M_{ul} due to sample disturbance.

In Equation (5.17) a model described by Persson (2004) is one model that has been used in comparison with the other models.

$$M_{ul} = a * \sigma'_c * \left(\frac{\sigma'_v}{\sigma'_c - \sigma'_v}\right)^n \tag{5.17}$$

Where a and n are constants that according to Persson (2004) adapts the calculation to Swedish soils. σ'_c is the pre consolidation pressure, σ'_v is the verticle effective stress.

According to Persson (2004) the most reliable unloading modulus is a combination of Equation (5.18) and Equation (5.19).

$$M_{ul} = 1500 * \sigma'_c * \left(\frac{\sigma'_v}{\sigma'_c}\right)^4 \tag{5.18}$$

$$M_{ul} = 400 * \sigma'_v \tag{5.19}$$

Swelling due to consolidation is often calculated over a specific time span. For this Equation (5.20) in combination with Figure 5-4 is used.

$$T_v = \frac{c_{vs} * t}{d^2} \tag{5.20}$$

c_{vs} is the coefficient of consolidation in the swelling calculations, t is the time of consolidation, d is the drainage distance depending on if the drainage is one sided or double sided, see Figure 5-4, (d=h).

Uv is taken from the diagram in Figure 5-4. The Uv factor is then multiplied with the total settlement to get the settlement transpired.

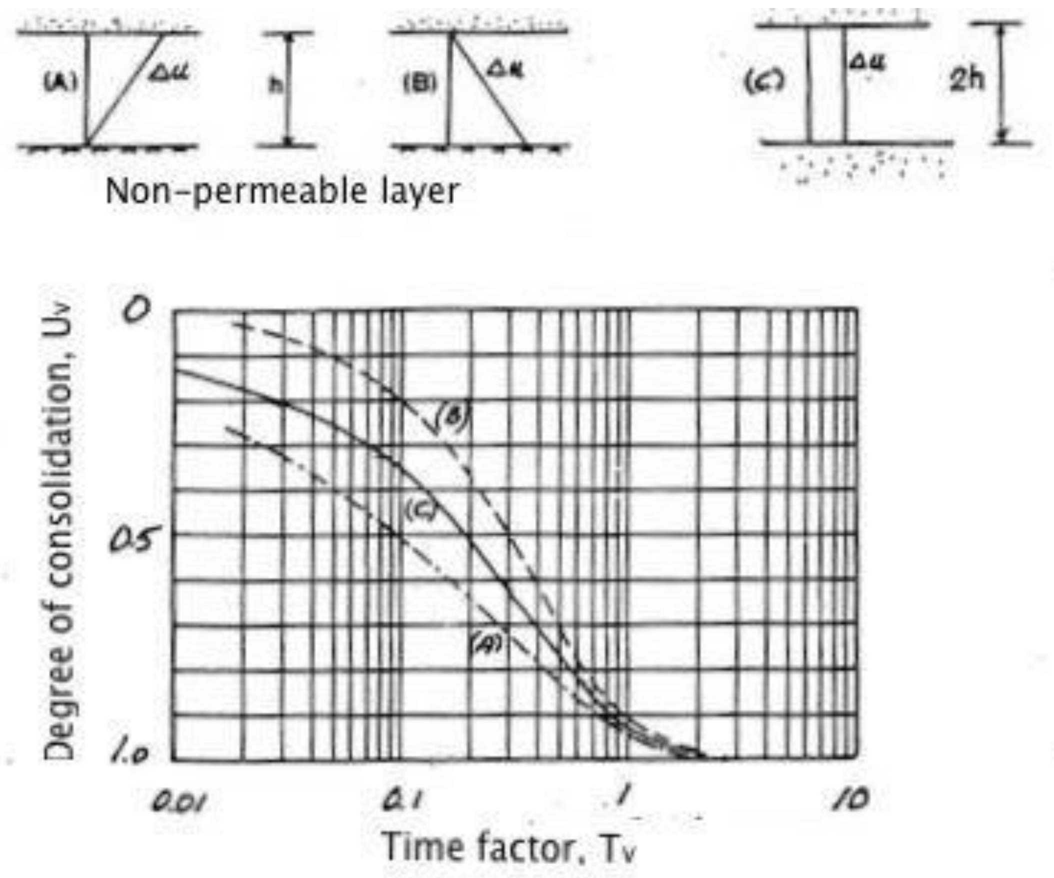

Figure 5-4 Time factor and degree of consolidation (Sällfors, 2009).

(A) is showing the drainage from bottom of the soil, (B) is drainage from top, (C) is drainage from both directions.

5.3.1 Water conditions

The water conditions are assumed hydrostatic. The groundwater level is according to (Vägverket, 2010) 0.5 meters below the ground surface. In the hand calculations the permeability of the columns is chosen to be the same as the permeability of the surrounding soil according to the in-situ tests described in 2.2.4 Permeability of lime-cement columns. The coefficient of permeability for clay is, according to the CRS tests 1.5×10^{-9} m/s.

5.3.2 Load distribution

The load is assumed to be distributed with the 2:1 method, both from the top and from the bottom of the lime-cement columns. This method of distributing the load is beneficial since it is more realistic when the columns are relatively long. The method of distributing the load either only from the bottom of the piles or only from 2/3 of the pile length into the ground has been proved too conservative (Alén, 2012). Because of this the method used is the one described by Alen, et al (2005) where the load is distributed partly from the top of the columns and partly from the bottom of the columns. The parts size is decided by using the factor η_{LC}.

5.4 Plaxis

This chapter describes how a model in the finite element program Plaxis was done. The model was analysed with two different conditions, drained and undrained. Both conditions simulate the behaviour of one single lime-cement column and its surrounding soil when exposed to excavation and loading with the result of swelling and consolidation respectively. First, the model is done with drained condition. Secondly, the model is slightly changed to simulate the undrained behaviour of the soil. The simulations were made as axisymmetric soft soil models.

5.4.1 Input phase Drained

The model created is 35 meters deep; this is the depth of the clay layer below the bridge at Kärra. The total width of the model is set to 0.7 meters to represent half the centre-centre distance between two columns, see Figure 5-5.

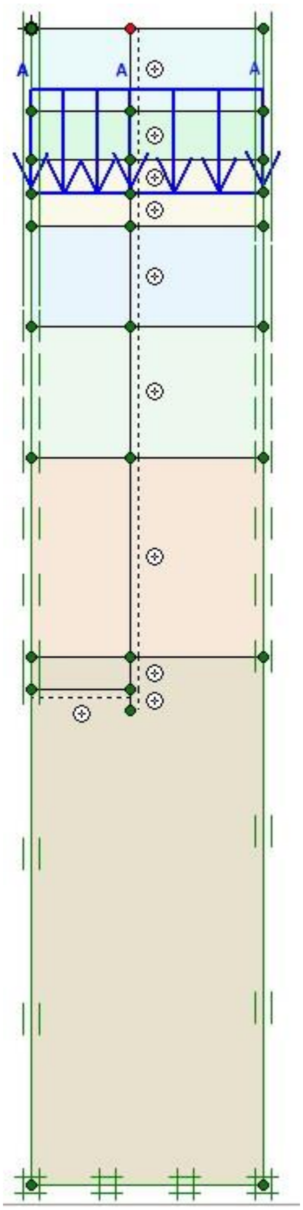

Figure 5-5 Geometry of the plaxis model (not in scale)

The soil body is divided into seven different layers corresponding to different stiffness parameters obtained from CRS analyses of the soil in the area of the bridge. To get the right stiffness parameters from the CRS analysis a program called “Graph Click” was used. This software takes out the X and Y values at any given position in a graph so that the right slope of the curve can be calculated. This allowed for good estimations of the λ^* and κ^* values at different depths.

Table 1 Input parameters used for the different soil layers

Parameter		0-2.5 m	2.5-4 m	4-6 m	6-9 m	9-13 m	13-18 m	18-35 m
Unit weight [kN/m^3]	**γ**	16.3	16.3	16.3	15.5	15.5	15.5	16
Modified comp-ression index [-]	**λ***	0.075	0.102	0.13	0.188	0.2	0.125	0.09
Modified swelling index [-]	**κ***	0.046	0.043	0.038	0.037	0.029	0.025	0.019
Friction angle [°]	**φ**	30	30	30	30	30	30	30
Dilatency angle [°]	**Ψ**	0	0	0	0	0	0	0
Over-consolidation ratio [-]	**OCR**	1.3	1.3	1.3	1.3	1.3	1.3	1.3
Permeability [m/day]	**k**	1.296*10E-4	1.296*10E-4	1.296*10E-4	1.296*10E-4	1.296*10E-4	1.296*10E-4	1.296*10E-4
Poissons ratio [-]	**ν**	0.2	0.2	0.2	0.2	0.2	0.2	0.2
K0 [-]	**K_0**	0.54	0.59	0.62	0.64	0.65	0.64	0.33

5.4.2 Calculation phase Drained

In this configuration the soil is considered drained, which makes Plaxis neglect pore water pressures and allow displacements to take place instantaneously. Input data for the different soil layers was assigned according to Table 1. A mesh was generated with the setting “Medium” and was then refined along the column. The calculations were then carried out in four different calculation steps as described below:

1. **Initial phase:** The initial stresses are created in the soil. All the seven different soil layers are activated and a phreatic water table is created 0.5 meters below ground surface. Initial stresses were calculated by K_0 procedure, using the in-situ K_0 value stated in Table 1. Displacements are reset to zero.

2. **Installation of column:** The lime-cement column is created. This is simulated by the activation of a new type of soil 0.3 meters wide (half the width of the LCC) next to the Y-axis, stretching 20 meters deep into the soil. The lime-cement column is created with Hardening soil model. This model is based on tri-axial tests, which are not available for the lime-cement columns under the bridge. Therefor the necessary parameters have been approximated, see Table 2.

Table 2 Column parameters (Volger & Karstunen, 2007)

Parameter		Value
Unit weight [kN/m^3]	**γ**	18
Secant stiffness modulus [MPa]	$\mathbf{E_{50}}$	30
Unloading reloading modulus [MPa]	$\mathbf{E_{ur}}$	90
Oedometer modulus [MPa]	$\mathbf{E_{oed}}$	30
Reference pressure [kPa]	$\mathbf{P_{ref}}$	100
Friction angle [°]	**Φ’**	37
Dilatency angle [°]	**Ψ**	0
Permeability [m/day]	**k**	5,18*10E-3
Poissons ratio [-]	**ν**	0.2
Power [-]	**m**	0,7
K_0^{NC} [-]	$\mathbf{K_0^{NC}}$	0,398

3. **Excavation:** In this step the top 5.4 meters of the soil and the lime-cement column is deactivated to represent the excavation. Since this is done with a drained condition, the swelling of the soil due to unloading will take place immediately. The water table is set equal to the bottom of the excavation.

4. **Loading:** The total settlements caused by the applied load due to the bridge can be approximated with a drained analysis. In this calculation step a load of 66 kN is applied to the soil. The load is represented by a pre-described load system. A concrete slab of high stiffness and no self-weight is created to distribute the load evenly onto the soil and the column.

5.4.3 Input phase Undrained

The model created is the same as in the drained case, but with the difference that all soil layers have the setting undrained instead of drained. Hence the model still is 35 meters deep and 0.7 meters wide.

5.4.4 Calculation phase Undrained

After the proper input data was assigned to the soil, a mesh was generated with the setting “Medium” and was then refined along the column. The calculations were then carried out in six different calculation steps as described below:

1. **Initial phase:** The initial stresses are created in the soil. A phreatic water table is created 0.5 meters below ground surface. Initial stresses were calculated by K_0 procedure, using the in-situ K_0 value stated in Table 1.

2. **Installation of column:** The lime-cement column is created. This is simulated by the activation of a new type of soil 0.3 meters wide (half the width of the LCC) next to the Y-axis, stretching 20 meters deep into the soil. The lime-cement column is created with Hardening soil model. This model is based on tri-axial tests, which are not available for the lime-cement columns under the bridge. Therefor the necessary parameters have been approximated, see Table 2.

3. **Excavation undrained:** As in the drained analysis, the first 5.4 meters of soil and column is deactivated, but this time with an undrained condition. The water table is set equal to the bottom of the excavation. The swelling gained in this step does not represent the total swelling. The swelling of the excavation bottom after given time will take place in the following step, swelling.

4. **Swelling:** The excavation at Kärra was left open for about 30 days before the construction of the bridge began. This step is therefore applied to give the soil from the undrained excavation phase time to swell for 30 days.

5. **Loading undrained:** In contrast to drained loading, this phase takes into consideration the need for excess pore water pressure to dissipate before total displacements can take place. The settlements gained in this phase can be viewed as initial, or elastic, settlements. The same force, 66 kN, is used as in the drained phase. The load is still represented by a pre-described load system and a concrete slab of high stiffness and no self-weight is created to distribute the load evenly onto the soil and the column.

6. **Consolidation:** In this phase the excess pore water pressure created in the previous loading phase is allowed time to dissipate, the setting is undrained. As the water dissipates, consolidation takes place with resulting settlement. The time is set to 365 days to get the displacement a year after the bridge was built.

6 Results

This chapter presents the results obtained from the hand calculations and the Plaxis simulations. The results from Plaxis will be presented as both drained and undraind, where the undrained analysis is more representative to the reality. The results from the hand calculations are meant to serve as a comparison to Plaxis and swelling as well as settlement is presented separately.

6.1 Plaxis

Drained: The drained analysis gave a swelling of 16.4 cm and a total settlement of 24.5 cm. As seen in Figure 6-1, the column, represented by green colour, buckles out under the load.

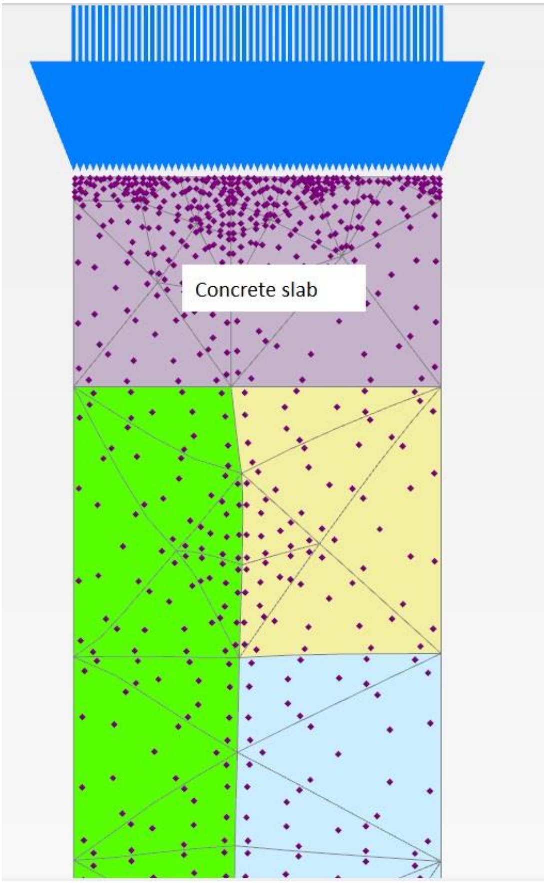

Figure 6-1 Drained analysis: Loading, scaled up 5 times

Undrained: In the undrained analysis, the total swelling becomes 16.6 cm after 30 days. In the loading step the soil settles 15.7 cm instantaneously and during the consolidation phase it settles another 10.4 cm due to consolidation. This gives a total settlement of 26.1 cm. As seen in Figure 6-2 the column, represented by tan colour, buckles out under the load.

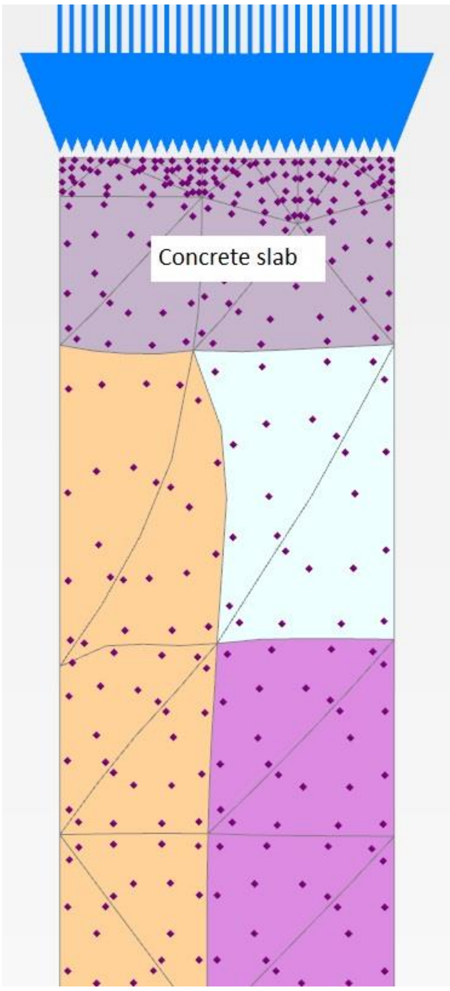

Figure 6-2 Undrained analysis: Loading, scaled up 10 times

6.2 Hand calculations

6.2.1 Swelling

The total depth, from the top of the embankment to the bottom of the excavation, is 6.7 m see Appendix 1. The height of the embankment is 1.3 m see Appendix 1 for detailed view. This gives an excavation depth of 5.4 m. The weight of the excavated soil gives cause to a change in effective vertical stress of 87 kPa at the bottom of the excavation when removed. This unloading will cause swelling of the soil.

The calculations show a total swelling of 29 cm when using M_0 as swelling modulus, see Figure 6-3. When the time factor is used in the calculations and the swelling is given a timespan of two months, the total swelling will have occurred within that given time.

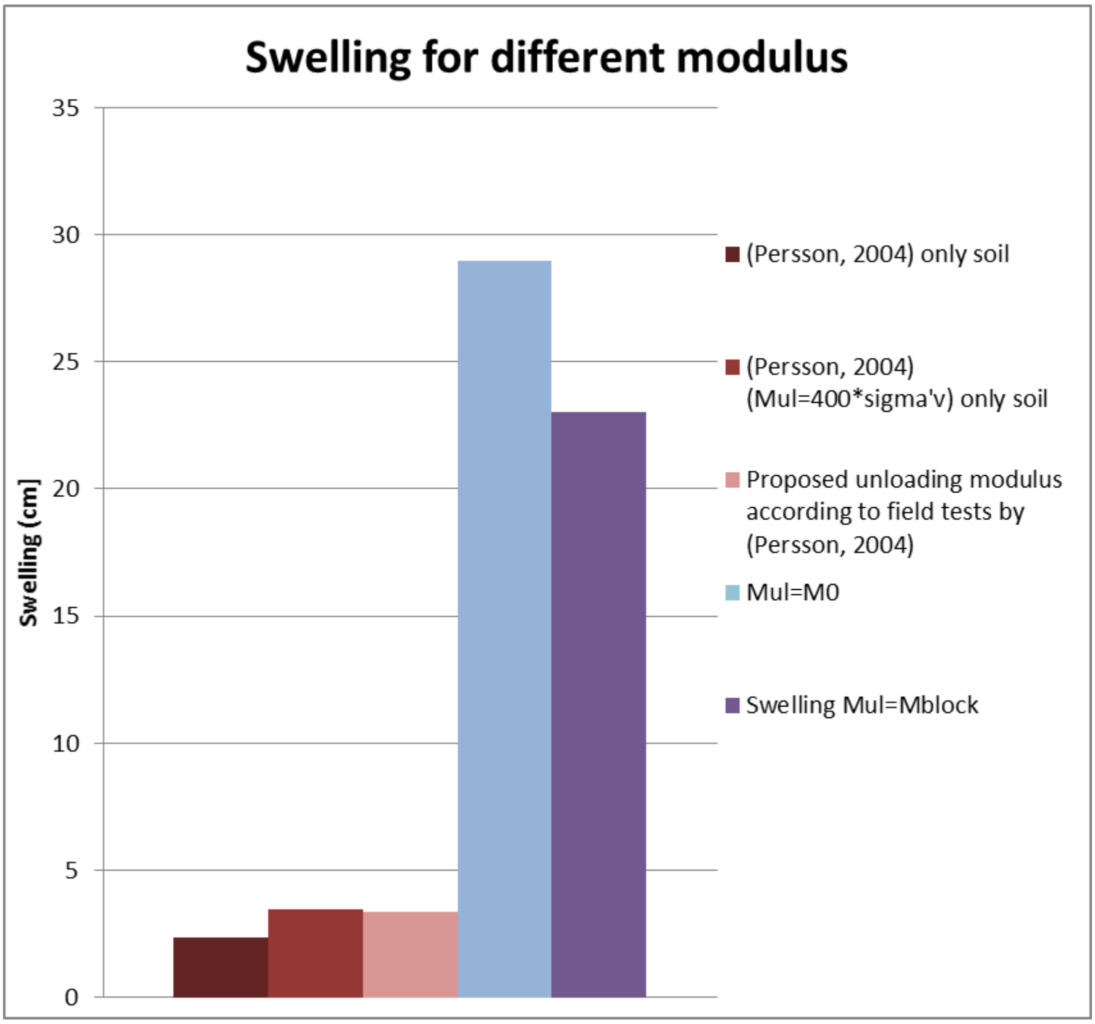

Figure 6-3 Total swelling with different models

6.2.2 Settlement

The stress distribution is approximated with a model proposed by (Alen, et al., 2005), were the load is divided into two parts. One part of the load, $(1 - h_{LC})$, spreads from the surface and the other part, h_{LC}, transfers down along the piles and spreads from the bottom of the piles. When taking into account plastic failure of the lime-cement columns, the depth at which the lime-cement columns turn into plastic failure, zone A, will stretch down to a depth of 3.6 m as shown in Figure 6-4. This calculation method gives a total settlement of 25,3 cm. When not considering plastic failure in the columns the total settlement becomes 16.4 cm, see Figure 6-5

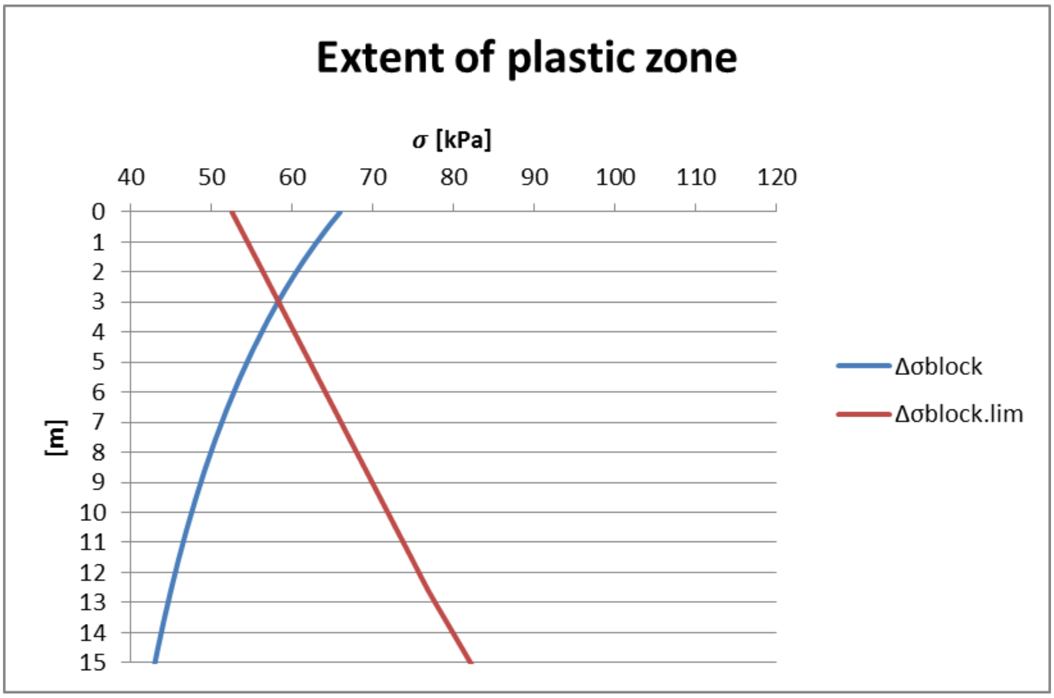

Figure 6-4 The depth of zone A

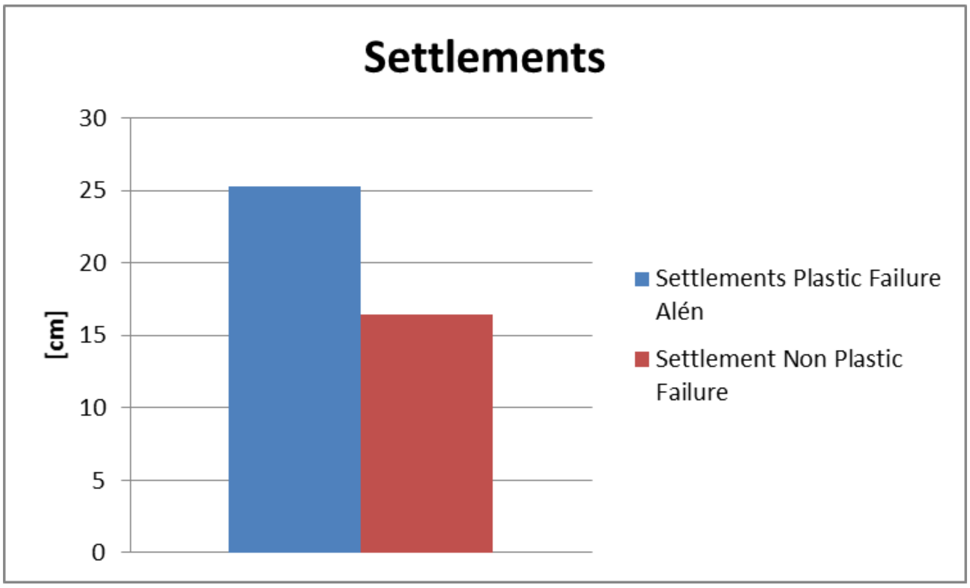

Figure 6-5 Settlement calculation

7 Discussion

A large part of this thesis has been dedicated to study the effects of swelling of the soil. The question of how important swelling is in settlement calculations for a structure may arise to the reader. Swelling is important because if, for example an excavation bottom was to be loaded instantly after excavation with a load less than the soil weight removed, the soil would theoretically start to swell and the structure on the soil would move upwards. If the soil instead would be given time between the excavation and loading the soil would experience the effect described as swelling. When the soil then is loaded it starts to compress back to a level depending on the weight of the applied load. Which of the two movements (swelling or compression) that would be larger depends on the weight added and this is why the swelling calculations of a specific case is important. An easy way to reduce the amount of swelling would be to minimize the time between excavation and loading.

Measurements of the settlements for the bridge considered in this study started when the bottom plate was installed. This means that the soil has had time to swell for about two months before the measurements were started. The amount of swelling that took place during these two months is unknown.

Since no unloading-reloading oedometer tests were done on the clay in the area of the bridge, no unloading modulus is available. Hence the moduli had to be approximated in the calculations for the swelling of the clay. Different approximations have been made. The best approximation of the unloading-reloading modulus in this case is assumed to be equal to the oedometer modulus, M_0. This is because studying stress-strain curves, gained from oedometer tests of Gothenburg clay, show similar slope of the unloading-reloading part of the curve as the M_0 part of the curve. The calculated swelling using M_0 also seems to correlate with the measurements done by Vägverket.

Another method for approximating the unloading-reloading modulus is using the other models described in Chapter 5.3. When swelling was calculated with these moduli, there were significantly smaller effects of the swelling than anticipated. These results did not match the indications from displacement measurements done by Vägverket. The unloading-reloading modulus approximated with Perssons method was about ten times the value of the oedometer modulus M_0, and at some depths even higher. The sum of this is that in the circumstances present, M_0 is probably a decent approximation of the unloading-reloading modulus. To get precise results from the swelling calculations oedometer unloading-reloading tests of the clay at the site should be performed.

The amount of settlement calculated with Plaxis and by hand calculations are very similar. This is a positive sign that the Plaxis simulations are performed in a correct way. There is a difference in total settlement between the two different hand calculation models used. This difference is anticipated since one of the methods takes into account plastic failure of the top part of the columns in a separate zone called zone A. Considering plastic failure gives cause to additional settlements compared with calculations without.

Since the results show such a great correlation to the measurements performed by Vägverket they can be seen as quite reliable. The input data though is the risky variable in the calculations. For example the weight of the bridge proved hard to estimate.

This report did not focus on the effects of installation of a lime-cement column. This effect might however be larger than anticipated and could therefore be an important factor in settlement calculations. Strain between the columns created by the installation in combination with the pore water pressure changes described by Shen (2003) would have been interesting to investigate further.

The permeability for each layer of the soil is given from CRS results evaluated by SWECO. The permeability used in the calculations of this report, 1.5×10^{-9} m/s, seems to be a reasonable value since this is taken from the CRS tests that represents the larger parts of the clay. The permeability has a large influence on how much of the swelling and settlement that transpires during the given time of two months. One order of magnitude higher permeability allows for all settlement to take place within two month, whereas one order of magnitude lower permeability will allow for almost no settlements during two months.

The Plaxis model used in this report exploits the axis symmetry available. The advantage with using an axisymmetric model, when modelling a single column, is that it captures the cylindrical shape of the column. The drawback with trying to simulate multiple columns in Plaxis 2D is that the plane strain model has to be used, in which columns get simulated as infinite rows stretching perpendicular to the plane of the window. In reality these rows consisted of columns with different lengths, something that is not possible to simulate in Plaxis 2D. The fact that single columns are installed between the rows makes the matter of simulation in 2D even worse. A more realistic way to model the displacements under the bridge in a finite element program would therefore be to use Plaxis 3D. The columns are installed with different installation lengths and different c/c-distances. The column pattern is not perfectly aligned and consists of single columns as well as rows of columns. The 3D pattern cannot be simulated in an accurate way with a 2D model. The Plaxis model in this report is a simplification that gives an indication of the settlement but the model could be improved by using 3D simulation.

The Soft soil material model is regarded as a good model for simulations of settlement in clay. The limits with this model are that it does not take into account creep settlements. This is however not of importance since this report only focuses the consolidation settlements and swelling, which are of greater importance in the relatively short time span in which the displacements of the bridge has been measured by Swedish traffic administration. The Soft Soil model has settings to account for eventual pre-consolidation of the soil. However some weaknesses of the model are that it does not take into account creep, anisotropy of the soil and it is most suitable for soil compression.

The moduli κ^* and λ^* were evaluated by reading values from a graph, the CRS tests. This method with the help of a computer program was used in order to get the most accurate results possible. The graph used was a CRS-test which is not ideal in this case. To get a better result an unloading-reloading oedometer test should be used. This gives room for some uncertainty in the values.

The input data to Plaxis is a possible source of error. The most important parameters affecting displacement; λ^*, κ^*, permeability and the overconsolidation ratio, have been obtained from CRS analyses. The modified swelling index, κ^*, should in the ideal case be obtained from an oedometer unloading-reloading test, instead M_0 was used, as discussed earlier. This is of course a source of error, but in the circumstances it is not thought to have any mayor impact on the results obtained. Greater deviations from the reality are thought to come from the inability to set up a representative geometry in the model.

The input parameters are in some cases uncertainty since many are not site specific. Some of the input parameters are standard values and some are assumed by Swedish Traffic Administration, SWECO, COWI and the writers since the actual value from the specific location is not available. The disturbance of samples taken from the site can also affect the results since the calculations are built on the parameters from in-situ tests and lab tests.

The calculated swelling in Plaxis appears to be in the right order of magnitude. The calculated compression however is larger than expected. A somewhat larger settlement in the Plaxis simulation is expected due to the fact that the simulation is based on a single column when in fact there are rows of columns directly underneath the bridge.

None of the models used take installation effects into account. The assumption that the heave from installation of the columns can be neglected because of the excavation after the column installation was partly true but not entirely. Ekström (2014) explains that the clay between the columns is deformed due to the installation of the columns and, that this will affect the settlement.

The installation effects could have quite a large effect on the settlement of the bridge. This has been described in the chapter about installation effects. Shen, et al. (2003) describes that during installation there are some fracturing, disturbances, consolidation and chemical transportation between soil and columns taking place that could affect the total settlement outcome. These factors are however difficult to involve into calculations.

The increasing pore-pressure due to installation is a factor that could have large effect on the total settlement as well. This is a factor that could be easy to measure during installation and could therefore be used in control calculations. The increased pore-pressure could be controlled by the induced pressure during lime-cement column installation.

In our case where the columns are installed in a tight pattern the effects of the plastic zone could be large. Almost all clay between the columns has been affected by installation disturbance and a large part has most probably been affected by fracturing.

Freezing of water allows the volume of water in the soil to expand into lenses. This expansion can lead to a heave of the soil and when melted to an increase of settlement. This can also be a factor to why the settlement in the case of the bridge in Kärra, since it was constructed partly during winter time, has larger settlement than what can be calculated in simple hand calculations.

In this report the pore water pressure is described as hydrostatic, even after the excavation. However Ekström (2014) explains that the pore water pressures at depths present in this scenario often stay constant at the bottom regardless of any change in the top of the clay. The pore pressure between these points will change accordingly and can be calculated by interpolation between the top and the bottom of the clay. This deviation from initial pore water pressure distribution has not been taken into account but could be an affecting factor of the total swelling and settlement. This could be taken into account in future numerical studies.

If the results from the different methods have a fairly high discrepancy, it can be suspected that either an error is made in the calculation or that one of the models does not represent the reality in a good way. In such a case further investigations should be made until the results are matching. In this report the correlation between Plaxis calculations and hand calculations are good. That indicates the calculations are reasonable but the results of the calculations are still dependent on the given input parameters and the estimated weight of the bridge. The calculations are also a good match to the actual measurements of the settlements on the bridge. However, given the lack of specific information and simplifications in the analysis (2D and 3D, no installation effects) this could be attributed to pure luck.

8 Conclusions

Swelling of the clay has an impact on the settlement of the bridge that should not be overlooked. Unlike compression, the modulus involving swelling and heave are not well understood.

The settlement of the bridge does not exceed the total swelling, because the unloading of the soil is actually larger than the additional weight of the bridge. The settlement is therefore calculated from the time when the bridge was added as a weight on the soil. The settlement was calculated to 10,6 cm after 2 months consolidation time.

The heave from installation of the lime-cement columns is most probably affecting the swelling and settlement of the bridge and is probably the cause of the extra centimetres swelling and settlement in the measured data.

The use of unloading-reloading oedometer tests could increase the knowledge of the clay behaviour in the specific area. This as an additional test would lower the uncertainties and make it easier to understand how the reinforcement can be made as effective as possible in the area.

An easy way to lower the effect of swelling would be to decrease the time between unloading the soil and building the bridge.

9 Recommended further investigations

For future projects handling settlements in combination with swelling and for deeper investigations in this project some further investigations are recommended. Firstly an oedometer unloading-reloading test would be interesting to perform in order to get a better understanding of the unloading and reloading properties of the soil. Measurements of installation heave in projects with lime-cement columns would be an interesting aspect to take into account. Also the effects of installation, fracturing, disturbances in the surrounding soil and pore-pressure changes would be interesting areas to investigate more. The long time settlements due to creep may be of interest for a more precise investigation of the settlement.

The simulations could preferably be redone in soil models that can handle installation effects. Also a 3D modeling program could make the simulation more realistic.

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Appendix 1

Drawings

Instructions for lime-cement columns, (Drawing 401G1500, Trafikverket 2010-06-10)

Drawing of cross section km 420, (Drawing 401G0989, Trafikverket 2010-06-10)

Drawing of bridge construction, (Drawing 443K2222, Trafikverket 2011-10-28)

(Drawing 401G1500, Trafikverket 2010-06-10)

UTFÖRANDE AV KALKCEMENTPELARE

Om inte annat anges på respektive förstärkningsritning gäller följande:

NOMENKLATUR
I föreskrifterna enligt nedan förekommer tre benämningar på kalkcementpelare med definition enligt följande:

1. Produktionspelare: Pelare som installeras löpande under produktionen
2. Förprovningspelare: Pelare som installeras och kontrolleras före det att installation av produktionspelare påbörjas
3. Kontrollpelare: Pelare som installeras och kontrolleras löpande under produktionen och som utgör produktionskontroll

DIMENSION
Ø = 600 mm

PLANLÄGE
Utsättning av kalkcementpelare görs utifrån redovisade baslinjer eller koordinatsatta pelare i separat lista. Entrepenören tar fram ett beteckningssystem där den egna beteckningen anges vid redovisning för varje enskild pelare tillsammans med beställarens pelarbeteckning i tabell 1 enligt nedan.

LUTNING
Pelare skall installeras vertikalt inom angivna toleranser.

PELARLÄNGD
Pelare inom ett område kan ha olika längd och avslutas på olika nivåer. För pelare som inte når fast botten framgår pelarnas längd av respektive ritning. För pelare till fast botten kan bedömda ungefärliga stoppnivåer avläses/interpoleras från nivå- eller djupkurvor på resp. ritning.

STABILISERINGSMEDEL
Kalk och cement 50/50 vikt-% (KC 50/50).

Kalk:
Kornstorlek 0-0,2 mm
CaO-aktiv halt >80 % enligt ASTM C25
Flytbarhet >70 enligt SS134005

Cement:
Sammansättning: Kontrollerad och godkänd enligt europeisk standard.
Kornstorlek 0-0,2 mm
CEM I eller CEM II/A-LL enligt SS-EN 197-1
Flytbarhet >40 enligt SS134005

INBLANDNINGSMÄNGD
Kontrollobjekt 1 (se tabell 4):
Nominellt 90 kg/m3, motsvarande 26 kg/m pelare
Kontrollobjekt 2 (se tabell 4):
Nominellt 90 kg/m3, motsvarande 26 kg/m pelare
Kontrollobjekt 3 (se tabell 4):
Nominellt 90 kg/m3, motsvarande 26 kg/m pelare

BLANDNINGSVERKTYG
Blandningsverktyget skall vara av typen pinnborr med minst 3 blandnivåer alternativt ett likvärdigt blandningsverktyg som ger motsvarande blandningsarbete.

ROTATIONSHASTIGHET
Maximalt 160 varv/minut

STIGNING
15-20 mm/varv. Använd stigning skall redovisas i dokumentation av utförda pelare enligt tabell 1.

TOLERANSER
Planläge:
För singulära pelare ± 0,10 m och för pelare i skivmönster ±0,05 m.
Angivet avstånd mellan två pelare får för singulära pelare avvika med som mest 0,10 m och för pelare i skivmönster med som mest 0,05 m.

Lutning:
För singulära pelare ± 0,02 m/m.
För pelare i skivmönster ± 0,01 m/m.

Inblandningsmängd:
± 20 % av nominell mängd inom flytande 1 m-intervall av enskild pelare
± 10 % av nominell mängd för varje pelare
± 2 % av total mängd inom varje kontrollobjekt (kontrollobjekten redovisas i tabell 4 nedan)

Blandningsförhållande:
±10 %-enheter.

MÄTNOGGRANNHET
Gejderlutning:
± 0,6 % i två vinkelräta riktningar.

Bindemedelsmängd:
± 2 kg/m.

Rotationshastighet:
±10 varv/minut.

Stigning:
±2 mm/varv.

FÖRBEREDANDE ARBETEN
Innan pelarinstallationen påbörjas skall hinder såsom befintliga kablar och ledningar, grundrester, pålar och liknande inom förstärkningsområdet schaktas bort och ersättas med krossmaterial av lämplig fraktion. Matjord och ev. befintlig vägkropp schaktas bort.

Arbetsyta:
Arbetsytans nivå, efter avschaktning av matjord och ev. befintlig vägkropp, skall mätas in och dokumenteras, se tabell 1.

ARBETSFÖRFARANDE VID PELARTILLVERKNING
Arbetsförfarandet skall följa SGF Rapport 2:2000, med följande tillägg/förtydliganden:
Samtliga pelare installeras från markytan. Utmatning av stabiliseringsmedel skall avslutas 0,5 m under arbetsytan. Utmatningstryck skall anpassas efter pelarlängd och installationsförhållanden.

Arbetsutförandet skall anpassas så att homogena och hållfasta pelare åstadkoms både över pelarens tvärsnitt och längs pelaren. Pelare skall utföras på sådant sätt att kratrar inte uppstår. Om kratrar, trots försiktighetsåtgärder, ändock uppstår skall dessa fyllas med förstärkningslagermaterial och packas med skopa när kraterns djup understiger 1,50 m. Om kraterns djup överstiger 1,50 m skall kompletterande pelare installeras och kratern fyllas med friktionsmaterial. Rörpelare betraktas på likadant vis som kratrar och skall åtgärdas enligt ovan. Krater betraktas som en följd av felaktigt utförande och åtgärder till följd av detta skall därför normalt ingå i á-pris. Uppträngande lera skall schaktas bort.

KONTROLL AV KALKCEMENTPELARE

GRUNDKONTROLL
Grundkontroll enligt SGF Rapport 2:2000 kap 8.4 sidorna 51-52. Dessutom skall följande dokumenteras:

- Klockslag för pelartillverkning
- Maskinförare
- Arbetsytans nivå

Uppgifter från grundkontroll skall även ingå i rapporteringen av tilläggskontroll.

TILLÄGGSKONTROLL - PELARE
Kontroll av pelares kontinuitet och hållfasthet skall, förutom genom grundkontroll, utföras enligt följande:

- Kontrollsondering med förborrad pelarsondering, FTPS, i givet antal pelare

Kontrollsondering med FTPS
Vid FTPS-sonderingen skall bandvagnen ha en effektiv tryckkraft av minst 35 kN och en dragkraft av minst 80 kN. Grävmaskin skall assistera med framgrävning av pelarens överyta och som mothåll.

Pelare skall kontrollsonderas genom FTPS (Förborrad Traditionell Pelarsondering med Spetskraftregistrering). Kontroll utförs med avseende på hållfasthet genom pelarsondering med separat registrering av spetstrycket mot vingen. I utrustningen skall även ingå en inbyggd inklinometer, för att möjliggöra bedömning av om sonden styrt ur pelaren eller inte. Om sonden tenderar att styra ur pelaren skall analys utföras på vilket djup detta sker och möjliga åtgärder vidtas. Innan pelarsondering utförs skall förborrning utföras med jordbergsonderingsutrustning (58 mm krondiameter) genom att använda tryck och rotation. Förborrningen skall utföras noggrant i pelaren, centrerad och vertikalt instyrd.

Sondens tvärsnittsarea väljs enligt SGF Rapport 2:2000. Om sondering avbryts på grund av för stort sonderingsmotstånd (> 30 á 35 kN) skall ny sondering utföras i samma pelare med mindre tvärsnittsarea på sondens vingar och med sonden roterad 90 grader jämfört med avbruten sondering.

Strategi för kontrollsondering
Kalkcementpelarproduktionen inom entreprenaden delas in i ett antal kontrollobjekt enligt tabell 4.

Pelare som skall kontrollsonderas skall utföras på samma sätt som produktionspelarna. Kontrollsonderingen skall utföras för pelare tillverkade av samtliga kalkcementpelarmaskiner som avses användas inom entreprenaden. Kontrollsondering utförs både i pelare som ingår i skivmönster och i singulära pelare. Sonderingen skall drivas minst en meter ned i leran under svävande pelare.

Pelare som kontrollsonderas skall fördelas jämnt mellan olika förekommande pelarlängder inom respektive kontrollobjekt. Kontroller skall utföras enligt tabell 4.

Inom varje kontrollobjekt skall kontrollsondering av pelare utföras i två grupperingar enligt följande:

1. Förprovningspelare:
Alla kontrollsonderingar skall utföras och redovisas 7 dagar före det att arbetet inom varje kontrollobjekt får påbörjas. Kontrollsonderingen skall omfatta ett visst antal förprovningspelare och utföras inom en begränsad provsträcka enligt tabell 4. Förprovningspelarna skall installeras i verkliga lägen. Ca 70 % av förprovningspelarna skall kontrollsonderas 12-16 dygn efter installation av pelarna och ca 30 % skall kontrollsonderas 26-34 dygn efter installation av pelarna. Förprovningspelare skall utföras med 2-3 lika stora grupper med inblandningsmängder enligt beställarens anvisningar.

2. Kontrollpelare:
Kontrollsondering skall utföras löpande under produktionen (produktionskontroll). Kontrollsondering skall omfatta ett visst antal pelare som skall vara väl fördelade inom varje kontrollobjekt enligt tabell 4. Kontrollsonderingen skall utföras 12-16 dygn efter installationen på samtliga pelare. Produktionskontrollen skall påbörjas snarast möjligt.

Utvärdering av pelarhållfasthet
Vid utvärdering av pelarnas skjuvhållfasthet beaktas att sambandet mellan det registrerade spetsmotståndet mot pelarvingarna och pelarens skjuvhållfasthet beror på antagen storlek på bärighetsfaktorn och hur stor andel av vingarna som har direktkontakt med pelaren. För sond med 500*15 mm eller 400*20 mm och med förborrning med 58 mm jordbergsonderingskrona kan omräkningsfaktorn mellan spetsmotstånd (kN) och skjuvhållfasthet (kPa) sättas till 12,5.

Förväntad pelarhållfasthet
För varje kontrollpelargrupp med minst 14 st kontrollpelare kan hållfasthetsvärden enligt tabell 2 och 3, verifierade genom sondering med FTPS, således förväntas vid angivna tider. Observera dock att tabell 2 och 3 nedan inte beskriver dimensionerande hållfasthet i pelarna utan förväntad hållfasthet med spridningsmått vid angivna tider.

Kompletteringar vid brister
Vid brister i tillverkningsprocess eller avvikelser från den förväntade i hållfastheten i pelarna skall kompletteringsåtgärder enligt SGF Rapport 2:2000 kap 8.42 utföras. Entreprenören skall snarast påtala och föreslå nödvändiga åtgärder för beställaren.

MILJÖKRAV
Skyddshuv skall användas för att förhindra att damm och lerstänk sprids. Skyddsskärm med höjden 2 m skall användas där problem med stänk kan befaras för närboende och trafikanter.

Entreprenören skall innan arbetena påbörjas upprätta en miljöplan som visar vilka skyddsåtgärder som avses vidtagas vid arbetet.

Plats för tankningsanordning av kalk skall godkännas av beställaren innan arbetena påbörjas.

Kalkcementpelarinstallation skall utföras på ett sådant sätt att omgivande mark ej påverkas av arbetena.

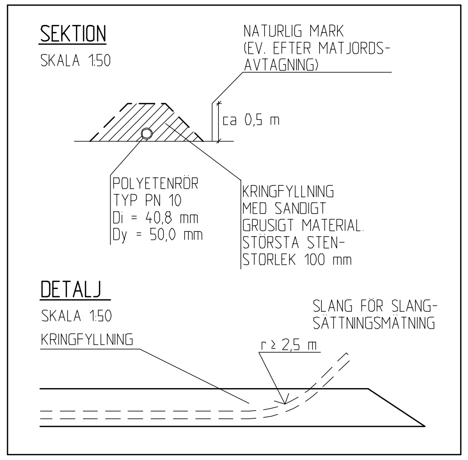
Särskilda skyddsåtgärder måste vidtas vid tankning av maskiner med kalk och cement. Åtgärder skall redovisas för beställaren.

Utsläpp av stabiliseringsmedel genom maskinhaveri eller olyckshändelse skall vattenbegjutas och blandas med lera. Händelsen rapporteras till beställaren.

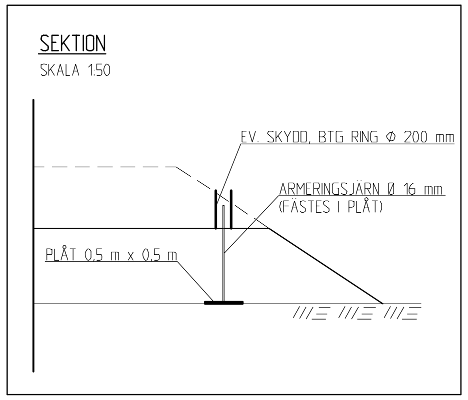
Vid avslutning i pelarens överkant (0,5 m under arbetsytan) skall allt stabiliseringsmedel blandas med jord innan flyttning sker till nästa pelare.

SÄTTNINGSKONTROLL

Principutformning slangsättningsmätare



Principutformning peglar



DOKUMENTATION AV KALKCEMENTPELARE

RAPPORT GRUNDKONTROLL
Rapportering av grundkontroll skall göras i samband med redovisning av mätsedelsystem, dels med utförda pelare sedan förra rapporteringstillfället och dels ackumulerat. Rapportering skall göras digitalt enligt tabell 1.

RAPPORT TILLÄGGSKONTROLL
Rapportering skall göras i rapporter för förprovningspelare respektive för kontrollpelare enligt nedan:

Rapportering av förprovningspelare:
För varje kontrollobjekt skall resultaten från kontrollsondering efter 12-16 dygn redovisas. Tryckkraft vid förborrning skall redovisas tillsammans med pelarsonderingen. På samma sida som pelarsonderingen skall, förutom data från grundkontrollen, även entreprenör, entreprenaddel, kontrollobjekt, sonderingsdatum och pelarsondens mått redovisas. Beställarens pelarbeteckningar (dvs sektion och sidomått) enligt koordinattabell skall användas vid redovisningen. Planer skall bifogas som visar de kontrollerade pelarnas läge.

Utöver grafisk redovisning av kontrollsonderingarna redovisas sonderingarna digitalt i Excel-format. Data från pelarsonderingarna förutsätts bli registrerade med 25 mm djupintervall. På det godtyckliga djupet z skall beräkningarna utföras baserat på sonderingsdata inom djupintervallet z-0,25 m till z+0,25. För glidande 0,5 m intervall beräknas och redovisas i tabell- och digitalform för varje 25 mm medelvärde, standardavvikelse, variationskoefficient och undre 15 % -fraktil. Dessa värden jämförs sedan med tabell 2. Om godtagbara värden uppnås kan efter beställarens godkännande den egentliga produktionen påbörjas Om förväntade värden enligt tabell 2 inte uppfylls skall detta redovisas till beställaren och åtgärder föreslås.

Rapportering av sonderingsresultat, tillsammans med övrig tillverkningsdokumentation, skall göras snarast efter att kontrollsonderingarna i förprovningen slutförts, dock senast 7 dagar efter första sonderingens utförande inom förprovningen.

Rapportering av kontrollpelare:
Produktionskontroll skall rapporteras på samma sätt som förprovning. Rapport skall löpande överlämnas till beställaren, Första rapporten skall överlämnas senast 14 dagar efter att första produktionskontrollen utförts inom entreprenaden. Varje rapport skall omfatta minst 6 sonderingar.

Avrapporteringen skall då entreprenaden slutförts göras digitalt med textdokument och med mätvärden, beräkningar och diagram i Excel-format inom en månad efter att kalkcementpelarproduktionen inom entreprenaden avslutats.

SLUTDOKUMENTATION
Efter att hela kalkcementpelarinstallationen slutförts skall ett gemensamt övergripande slutdokument utarbetas med slutsatser och erfarenhetsredovisning. Hänvisning skall göras till underliggande provningsrapporter och till övergripande utmatningsdata för samtliga pelare i projektet. Dokumentation av grundkontroll för samtliga installerade pelare upprättas. Redovisningen skall göras digitalt.

TABELL 2. KONTROLLSONDERING EFTER 12-16 DYGN

Djup under arbetsyta (m)	Medelvärde Cpelare (kPa)	Undre 15% fraktil (kpa)	Miniminivå för enskilt värde (kPa)
0-0,5	---	---	---
0,5	≥75	≥30	---
0,5-2,0	Rätlinjig interpolation	Rätlinjig interpolation	---
>2,0	≥100	≥80 *	50

* Motsvarar en variationskoefficient av ca 20%

TABELL 3. KONTROLLSONDERING EFTER 26-34 DYGN

Djup under arbetsyta (m)	Medelvärde Cpelare (kPa)	Undre 15% fraktil (kpa)	Miniminivå för enskilt värde (kPa)
0-0,5	---	---	---
0,5	≥100	≥50	---
0,5-2,0	Rätlinjig interpolation	Rätlinjig interpolation	---
>2,0	≥125	≥100 **	50

** Motsvarar en variationskoefficient av ca 15%

TABELL 4. KONTROLLOBJEKT

Kontroll-objekt nr	Delsträcka ca km	Provsträcka för förprovningspelare	Förprovnings-pelare efter 12-16 dygn	Förprovnings-pelare efter 26-34 dygn	Kontrollpelare
1	8/165-8/333	8/200-8/260	9 st	4 st	9 st
2	10/280-10/510	10/340-10/490	16 st	7 st	16 st
3	10/840-10/930	10/865-10/905	2 st	1 st	2 st

TABELL 1. PRINCIP FÖR LITTERERING OCH REDOVISNING AV KC-PELARE
Redovisning skall göras i Excel-format med innehåll och utseende enligt tabell. Tabell tillhandahålls av beställaren och fylls i av Entreprenören

Löp-nummer	Sektion (B)	Tvärmått (B)	Entr. Pelarnr (E)	X-koord (B)	Y-koord (B)	Ök Arbyta (E)	Ök Pelare (E)	Uk Pelare enl. ritn. (B)	Verklig uk pelare (E)	Stabiliserad pelarlängd (E)	Tomborr-ning m (E)	Genomsnitt Kg/m (E)	Stigning mm/varv (E)	Inbl. Verktyg (E)	Pelare till fast botten (E)	FTPS-kontroll (B)	Under-rättelse (E)	Under-rättelse (B)	Anm (B/E)	A-Pris/löpmeter (Ekonomi)	Styckepris (Ekonomi)

(B)= BESTÄLLAREN ANVARAR FÖR IFYLLNAD.
(E)= ENTREPRENÖREN ANSVARAR FÖR IFYLLNAD.

REV	ANT	ÄNDRINGEN AVSER	GODK.	DATUM	VV DATUM	VV DIARIENUMMER

BYGGHANDLING
FÖRFRÅGNINGSUNDERLAG

Vägverket
BANAVÄG I VÄST

SWECO
SWECO INFRASTRUCTURE AB
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403 14 Göteborg
Telefon 031-62 75 00,
Fax 031-62 77 22

VÄG E45
GÖTEBORG-TROLLHÄTTAN
DELEN HÖNEBÄCK - GLÄSSNÄS

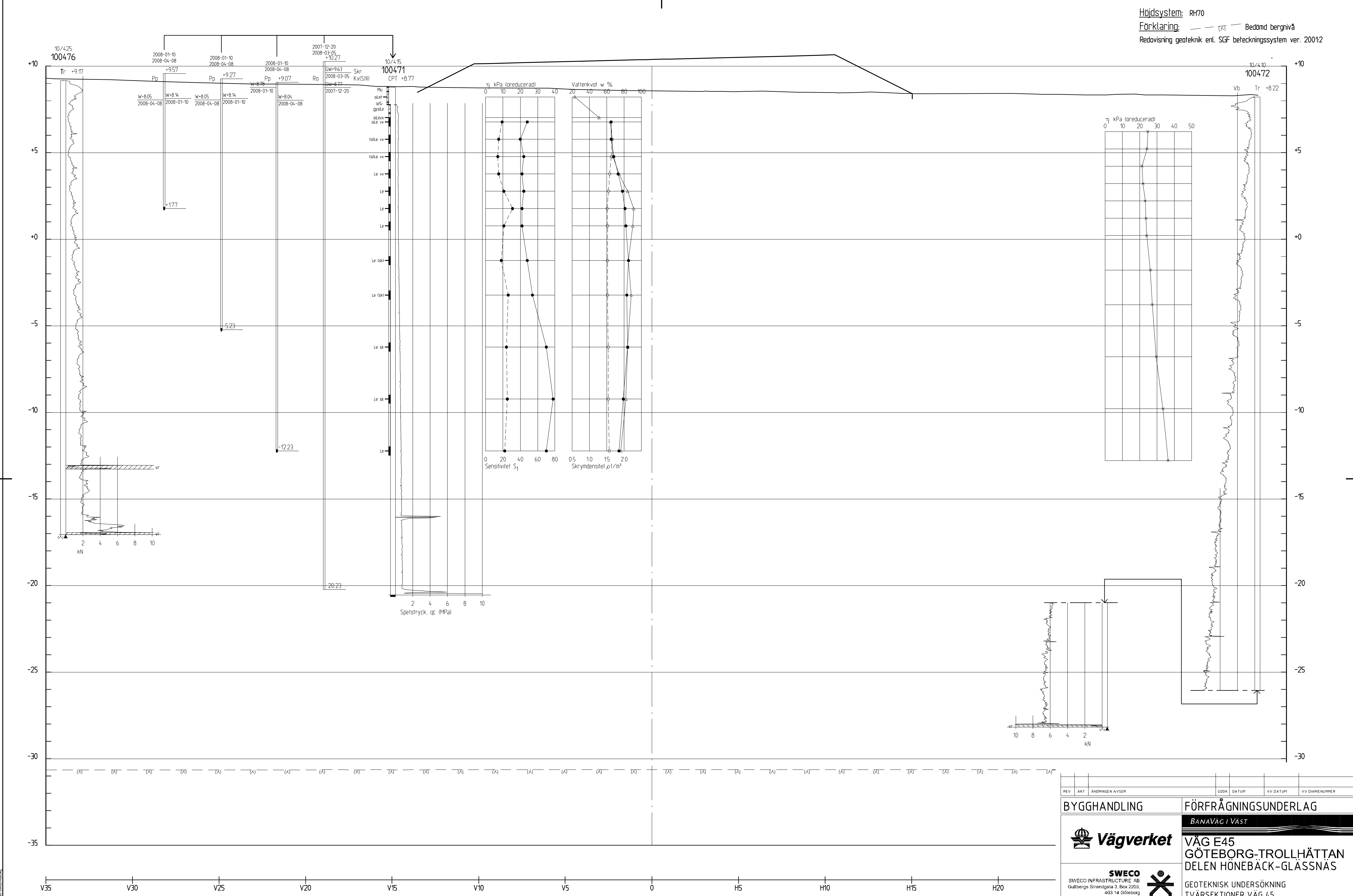
GEOTEKNISKA FÖRSTÄRKNINGSÅTGÄRDER
ANVISNINGAR KC-PELARE

TEXT

UPPDRAGSANSVARIG: A. PLANTHABER	UPPDRAGSNUMMER: 2300651
KONSTR: P.SJÖGREN	GRANSK: U. HÖGSTA
GÖTEBORG U. HÖGSTA	2010-06-10
KONSTRUKTIONSNR	FORMAT: A1
SKALA	
OBJEKT NR: 54 53 04	RITNINGSNR: 4 01 G 15 00
REV	

2010-10-15; 18:04; P:\2341\2300651_E45_Bygghandling\15-RITN\FrånChaps\4\G\Ritdef\401G1500.dwg

(Drawing 401G0989, Trafikverket 2010-06-10)



TVÄRSEKTION 10/420

1: 100

REV	ANT	ÄNDRINGEN AVSER	GODK.	DATUM	VV DATUM	VV DIARIENUMMER

BYGGHANDLING

FÖRFRÅGNINGSUNDERLAG

BANAVÄG I VÄST

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VÄG E45
GÖTEBORG-TROLLHÄTTAN
DELEN HÖNEBÄCK-GLÄSSNÄS

GEOTEKNISK UNDERSÖKNING
TVÄRSEKTIONER VÄG 45
SEKTION 10/420

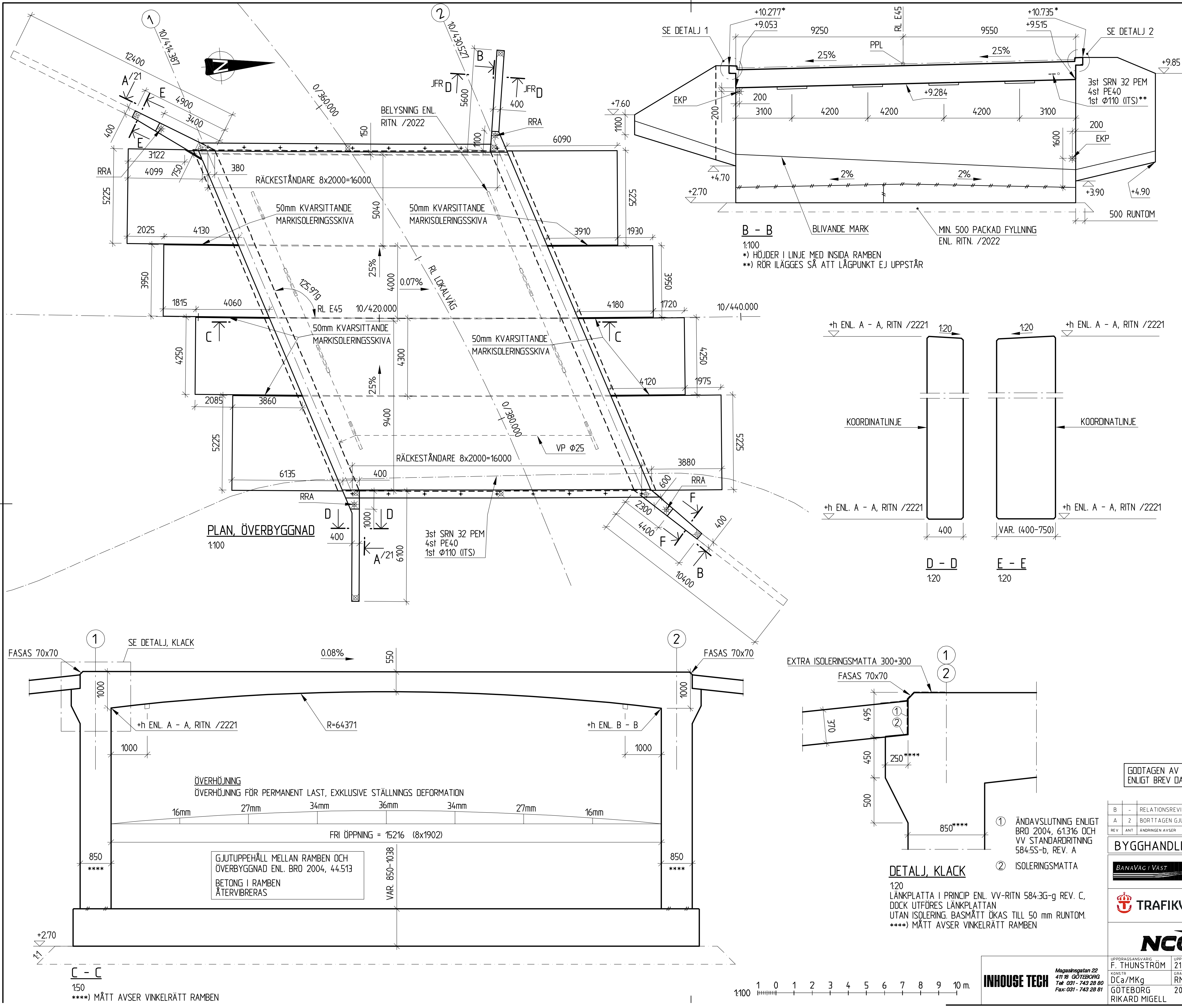
TVÄRSEKTIONER

UPPDRAGSANSVARIG	UPPDRAGSNUMMER	KONSTRUKTIONSNR	FORMAT	SKALA
A. PLANTHABER	2300651		A1	1:100
KONSTR: A. ANDERSSON	GRANSK: U. HÖGSTA	OBJEKT NR: 54 53 04	RITNINGSNR: 4 01 G 09 89	REV
GÖTEBORG U. HÖGSTA	2010-06-10			

2010-10-13; 12:31; P:\2341\2300651_E45_Bygghandling\15-RITN\FrånChaos\4\G\Ritdef\401G0989.dwg

VVA545300BY (19) 2011-01-20:1525

(Drawing 443K2222, Trafikverket 2011-10-28)



ANVISNINGAR

SE RITNING /2022

BETECKNINGAR

- AVVÄGNINGSDUBB
- EKP/RRA
- \+ RÄCKESTÅNDARE
- = = DRÄNERINGSKANAL

HÄNVISNINGAR

SAMMANSTÄLLNING 1 (2)	SE RITNING /2021
SAMMANSTÄLLNING 2 (2), FÖRESKRIFTER	SE RITNING /2022
MÅTT 1 (2)	SE RITNING /2221
ARMERING 1 (5)	SE RITNING /2223
ARMERING 2 (5)	SE RITNING /2224
ARMERING 3 (5)	SE RITNING /2225
ARMERING 4 (5)	SE RITNING /2226
ARMERING 5 (5)	SE RITNING /2227

15-1806-1 b

GODTAGEN AV TRAFIKVERKET, TEKNIK OCH MILJÖ, ENHET BYGGNADSVERK ENLIGT BREV DATERAT 2011-11-16 MED DIARIENUMMER TRVAT 2011/3150

REV	ÄNT	ÄNDRINGEN AVSER	GODK	DATUM	TRV DATUM	TRV DIARIENUMMER
B	-	RELATIONSREVIDERING	FTm	2012-11-18	-	-
A	2	BORTTAGEN GJUTFOG, RAMBEN OCH ÖVERB.	RMI	2012-03-07	2012-04-03	TRVAT 2012/465

BYGGHANDLING | RELATIONSHANDLING

BANAVÄG I VÄST
TRAFIKVERKET
NCC
INHOUSE TECH — Magasinsgatan 22, 411 18 GÖTEBORG, Tel 031 - 743 28 80, Fax: 031 - 743 28 81

VÄG E45
GÖTEBORG-TROLLHÄTTAN
DELEN HÖNEBÄCK-GLÄSSNÄS

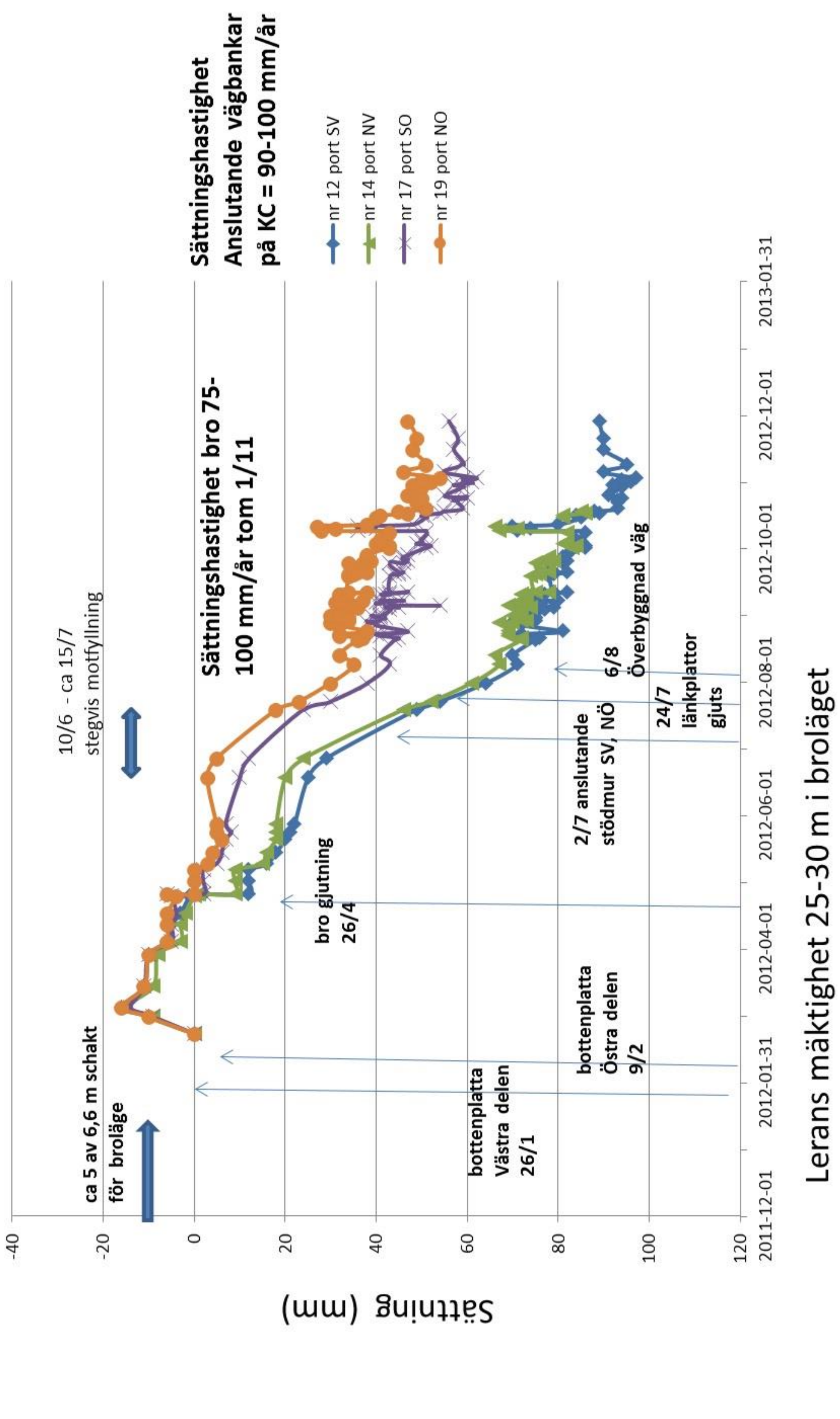
BRO FÖR VÄG E 45
ÖVER LOKALVÄG, KÄRRA, KM 10/425

MÅTT 2 (2)

UPPDRAGSANSVARIG: F. THUNSTRÖM	UPPDRAGSNUMMER: 210730			
KONSTR: DCa/MKg	GRANSK: RMI	KONSTRUKTIONSNR: 15-1806-1	FORMAT: A1	SKALA: SE FIGUR
GÖTEBORG, RIKARD MIGELL	2011-10-28	OBJEKT NR: 54 53 04	RITNINGSNR: 4 43 K 22 22	REV: B

Appendix 2

Measurements of bridge movement



Sättning (mm)
-40
-20
0
20
40
60
80
100
120
2011-12-01
2012-01-31
2012-04-01
2012-06-01
2012-08-01
2012-10-01
2012-12-01
2013-01-31
ca 5 av 6,6 m schakt
för broläge
10/6 - ca 15/7
stegvis motfyllning
Sättningshastighet bro 75-
100 mm/år tom 1/11
Sättningshastighet
Anslutande vägbankar
på KC = 90-100 mm/år
nr 12 port SV
nr 14 port NV
nr 17 port SO
nr 19 port NO
bottenplatta
Västra delen
26/1
bottenplatta
Östra delen
9/2
bro gjutning
26/4
2/7 anslutande
stödmur SV, NÖ
24/7
länkplattor
gjuts
6/8
Överbyggnad väg
Lerans mäktighet 25-30 m i broläget