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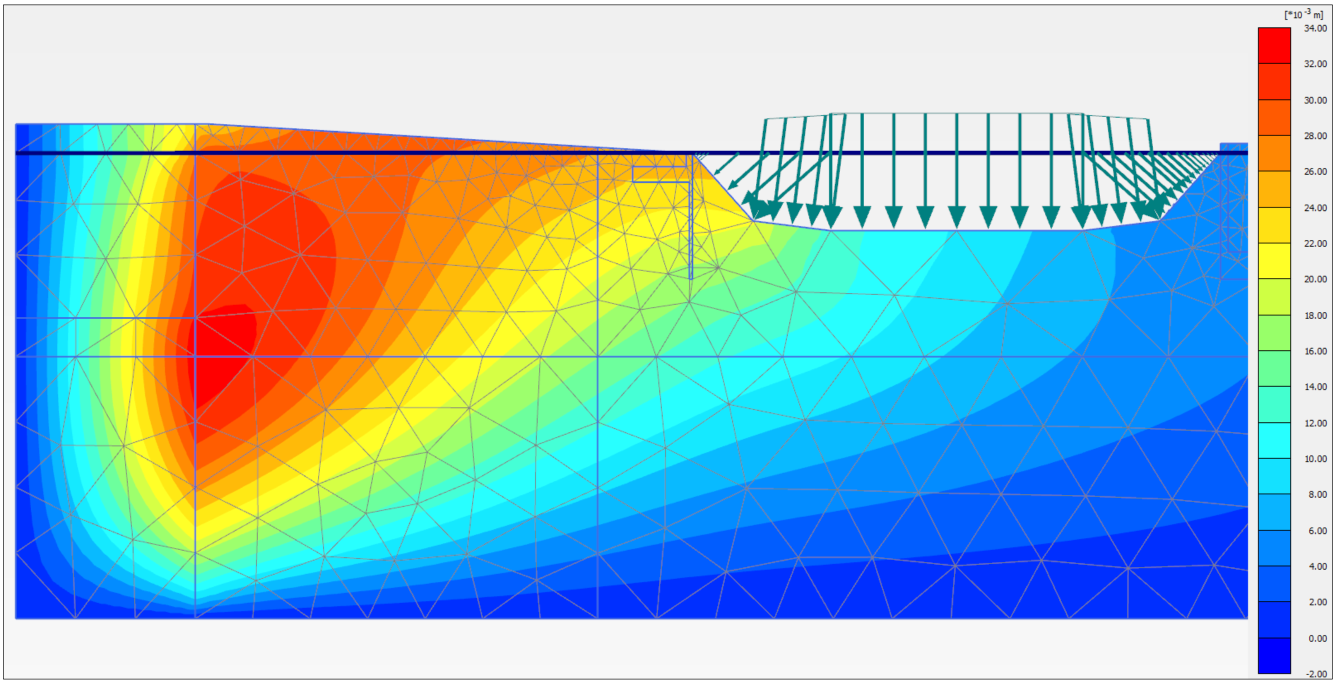


# Analysis and FE-modelling of soil displacement associated to pile driving

## A case study of pile installation at Gamlestadstorget

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

HENRIK HERNQVIST
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Department of Civil and Environmental Engineering
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Master's Thesis BOMX02-16-50

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Examensarbete BOMX02-16-50 / Institutionen för bygg- och miljöteknik,
Chalmers tekniska högskola 2016

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Cover:
PLAXIS plane strain model showing the horizontal displacements at section 100/130 at Gamlestadstorget. For more information see Chapter 5.1.3.

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ABSTRACT

The aim of the master thesis is to investigate ground movements due to pile installation at the Gamlestadstorget construction project and to compare different methods used for analysis and prediction. The thesis was conducted through a literature study, a case study of a part of Gamlestadstorget Spårskede 4, and a FE-modelling with the numerical tool PLAXIS 2D. The scope was to investigate section 100/130, with regard to horizontal- and vertical displacements caused by the installation of hammered pre-cast concrete piles and bored steel pipe piles. For the FE-modelling, a reference project of the Partihall highway bridge was investigated to identify a suitable pile modelling method. With the use of volumetric expansion, to simulate the total installed pile volume, together with a 50 per cent pre-auger efficiency, it was possible to achieve good agreement compared to measured data. From the results of the section 100/130 model, it was possible to conclude that a volumetric expansion equal to 50 per cent of the installed pile volume, together with a 50 per cent pre-auger, complied best with measured data. In the comparison with the empirical method by Rehnman and the semi-analytical method by Sageseta, PLAXIS had the best agreement to measured data with depth. It can also be concluded that it is possible to model the soil displacements from a pile group installation in a plane strain model. However, due to several insecurities in the case study, a general advice cannot be established until the method have been verified in several other cases. The movement caused by the bored steel pipe piles was irrelevant and thus determined as being infeasible to model in the scope of this thesis. Future research could be to study the installation effects of bored steel pipe piles by more extensive measurements or further studies to validate the pile modelling in PLAXIS.

Key words: Pile installation, soil displacement, heave, FEM, PLAXIS 2D, Sagaseta, Rehnman

Analys och FEM-modellering med avseende på markdeformationer kopplade till pålning.

En fallstudie av pålinstallation vid Gamlestadstorget

*Examensarbete inom masterprogrammet Infrastructure and Environmental Engineering*

HENRIK HERNQVIST
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SAMMANFATTNING

Målet med examensarbetet är att undersöka markdeformationer som uppkommit vid pålinstallation vid projekt Gamlestadstorget och jämföra olika metoder för analys och prognostisering. Examensarbetet genomfördes genom en litteraturstudie, en områdesbeskrivning av Gamlestadstorget Spårsekde 4 och en FE-modellering med det numeriska verktyget PLAXIS 2D. Inom ramarna av examensarbetet studeras sektion 100/130 med avseende på horisontella- och vertikala förskjutningar orsakade av installation av slagna förtillverkade betongpålar och borrade stålrörspålar. I FE-modelleringen användes Partihallsbron som referensprojekt för att undersöka och identifiera en lämplig metod för att modellera massundanträngning. Genom att expandera jordvolymen, för att simulera den totala volymen av installerade pålar, tillsammans med en förborrning med 50 procents effektivitet, kan man uppnå en god överenstämmelse mot uppmätta värden. Utifrån resultatet av modellen av sektion 100/130 var det möjligt att dra slutsatsen att en expandering av volymen som motsvarar 50 procent av den totala volymen installerade pålar, tillsammans med 50 procent förborrning, gav bäst överenstämmelse mot uppmätt värden. I jämförelsen med den empiriska metoden av Rehnman och den semi-analytiska metoden av Sagaseta visade det sig att PLAXIS hade bäst samstämmighet med uppmätt data mot djupet. Det går också att dra slutsatsen att det är möjligt att modellera jordförskjutningar, från en installation av en pålgrupp, i en två-dimensionell model. På grund av osäkerheter kan inte en generell slutsats för modellering dras förrän metoden blivit verifierad för fler fall. Markrörelsen som de borrade stålrörspålarna orsakade var försumbar och det bedömdes därför vara för svårt att modellera dessa inom avgränsningen för denna studie. Framtida forskning skulle kunna bestå i att studera installationseffekter av borrade stålrörspålar genom utförligare instrumentering eller att validera pålmodelleringen utförd i PLAXIS genom fler studier.

Nyckelord: Pålning, markdeformationer, hävning, FEM, PLAXIS 2D, Sagaseta, Rehnman

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# Preface

In this study, horizontal- and vertical displacements due to the installation of hammered pre-cast concrete piles and bored steel pipe piles have been investigated.
The study aims to compare different methods used for analysis and prediction such as Rehnman's method, Sagaseta's method and the finite element method.

The thesis has been carried out at the division of GeoEngineering, Chalmers University of Technology, in collaboration with Skanska Väg- och Anläggning Väst and Skanska Teknik during spring 2016. The thesis was initiated by Skanska but modified during the progress of the thesis. It was supervised by Mats Karlsson, assistant professor at the Division of GeoEngineering at Chalmers, Fredrik Olsson at Skanska Väg- och Anläggning Väst, Johannes Tornborg and Peter Claesson at Skanska Teknik. Mats Karlsson was the examiner of the thesis.

Firstly, we would like to thank our supervisor and examiner Mats Karlsson for the guidance and help throughout the thesis. You have challenged us to dig deeper into the literature and to fully understand the ideas behind the theory.

We would also like to thank Fredrik Olsson for his insights of the project of Gamlestadstorget and for his help managing the thesis. We are also indebted to Johannes Tornborg and Peter Claesson for taking the time to supervise us and their help with providing all necessary material for the project at Gamlestadstorget. We thank Anders Kullingsjö for contributing with his knowledge about pile modelling and Sagaseta's method.

Lastly we would like to express our sincere gratitude to all colleagues, friends and family who have endured our company during this last five years of study.

Gothenburg, June 2016

Henrik Hernqvist and David Nguyen

# Notations

**Roman upper case letters**

| | |
|---|---|
| $E$ | Young's modulus |
| $E_{eod}$ | Oedometer modulus |
| $E'$ | Effective stiffness |
| $K_0$ | Earth pressure coefficient |
| $K_{0NC}$ | Earth pressure coefficient for normally consolidated soil |
| $L$ | Pile length |
| $R$ | Pile radius |
| $V_{pb}$ | Volume of clay removed with pre-boring |
| $V_{piles}$ | Volume of driven piles |

**Roman lower case letters**

| | |
|---|---|
| $b$ | Width of piling area |
| $c$ | Cohesion |
| $c_u$ | Undrained shear strength |
| $d$ | Depth of piles below ground surface |
| $l$ | Length of piling area |
| $r$ | radial distance from pile |
| $w_L$ | Liquid limit |
| $x$ | Heave within piling area |

**Greek letters**

| | |
|---|---|
| $\alpha$ | Relative load of building in Rehnman's method |
| $\beta$ | Relative load of building in Rehnman's method |
| $\gamma$ | Relative load of building in Rehnman's method |
| $\delta$ | Relative load of building in Rehnman's method |
| $\delta_h$ | Horizontal displacement |
| $\delta_v$ | Vertical displacement |
| $\varepsilon$ | Strain |
| $\eta$ | Heave factor |
| $\nu$ | Poisson's ratio |
| $\rho$ | Density |
| $\sigma'_c$ | Pre consolidation pressure |
| $\sigma'_0$ | In-situ effective stress |
| $\varphi$ | Friction angle |
| $\psi$ | Dilatancy angle |

**Abbreviations**

| | |
|---|---|
| c/c | Centre to Centre |
| CPT | Cone Penetration Test |
| CRS | Constant Rate of Strain |
| DTH | Down-The-Hole |

| | |
|---|---|
| FEM | Finite Element Method |
| $HHW^{100}$ | Highest High Water, 100 years |
| $HHW^{50}$ | Highest High Water, 50 years |
| LE | Linear Elastic |
| LLW | Lowest Low Water |
| MC | Mohr-Coulomb |
| MHW | Mean High Water |
| MLW | Mean Low Water |
| MWL | Mean Water Level |
| NC | Normal Consolidated |
| OC | Over Consolidated |
| OCR | Over Consolidation Ratio |

*"To acquire competence in the field of earthwork engineering one must live with the soil. One must love it and observe its performance not only in the laboratory but also in the field, to become familiar with those of its manifold properties that are not disclosed by boring records" – Karl von Terzaghi*

# 1 Introduction

With a general trend of increased urbanisation, where increasingly more people are moving into cities, there is also a large demand for new buildings and infrastructure. But denser cities lead to new constructions often being built adjacent or even upon older ones. This requires extensive foundation structures and is a prerequisite for constructions with long lifespans.

One of the most common techniques used for ground improvements are to use piles, more specifically pre-cast concrete piles which are displacement piles. Driving a pile into the soil creates displacements that could affect the function of, or be detrimental to nearby structures and ground reinforcements (Sagaseta and Whittle, 2001). The displacements occur both on the ground surface and below, which is why they could affect buildings and underground constructions alike. As more constructions are built in a specific area or confined space, it thus becomes increasingly important both to understand the processes involved but also to be able to predict the displacements caused.

To be able to analyse and predict the effect of a new construction, a variety of different methods are used depending on the complexity of the problem. The soil displacements could be assessed through empirical-, analytical-, or numerical methods such as a finite element method. Earlier methods developed have often been simpler and formulated based on empirical relations, giving a generalised and rough estimate of the displacements expected. Advanced analytical and semi-analytical methods have also been developed with an increased complexity but also accuracy. The use of numerical analysis, such as the finite element method, is however becoming increasingly widespread for predicting displacements and the effects of installing piles into the ground.

## 1.1 Background

At Gamlestadstorget, north east of central Gothenburg, Skanska have been assigned a contract to reconstruct a tramway junction. The task involves replacing the old tramway as well as two bridges, creating a new square along with a new quayside and to prepare for further development in the area. The geotechnical conditions are complicated and large amount of ground reinforcements is needed. The main methods used are hammered pre-cast concrete piles and bored steel pipe piles, where concrete piles are known to create soil displacements and bored piles are supposed to not give any large deformations.

Different ways of analyzing and predicting soil displacements have previously been used in Sweden. The common practice of calculating displacements has been through the empirically based method redeveloped by Sven-Erik Rehnman (Olsson and Holm, 1993). In Skanska’s prognoses for the project, the more advanced theoretical Shallow Strain Path Method developed by Sagaseta and Whittle (2001) have been used. Finite element analysis is also increasingly common to use and have been investigated by several projects. The different methods are also compared in several publications, including *Ground displacements due to pile driving in Gothenburg clay* by Edstam and Kullingsjö (2010) and the master thesis *Environmental impact of pile driving* authored by Nenonen and Ruul (2011).

Regarding the finite element analysis, the program PLAXIS, both the 3D and 2D version, is frequently used for geotechnical problems. The 3D version is more resource demanding but more realistically captures real world behaviour. In the 2D version, different methods of modelling soil displacements can be performed. One is using an axisymmetric model, which simulates 3D behaviour but only is suitable for single piles or smaller pile groups. The 2D plane strain model can be utilised when modelling larger pile groups or elongated pile groups, it however does not realistically capture the 3D behaviour of the soil.

## 1.2 Aim and objectives

The aim of this master thesis is to investigate ground movements due to pile installation at the Gamlestadstorget construction project and to compare different methods used for analysis and prediction.

The aim can be divided into the following objectives:

- Study the measured displacements caused by piling of hammered pre-cast concrete piles and bored steel pipe piles.
- Perform a numerical analysis, of one section, in the finite element program PLAXIS 2D and to identify a pile modelling method in compliance with measured data.
- Understand and apply the two common methods, proposed by Rehnman and Sagaseta respectively, used for estimating soil displacements.
- Compare the results obtained from the different methods.

## 1.3 Method

A literature study for the theory part was conducted to commence the work. It included a review of research articles and master theses in the field to assess what had already been accomplished. It also included a factual analysis in which a deeper understanding of piling and piles was established. The literature study focused on theories on how the soil behaves and the effects of piling as well as information about the different piling techniques of hammered pre-cast concrete piles and bored steel pipe piles. It also included research about different methods for calculating soil displacement and FE-modelling.

The case study at the Gamlestadstorget construction project was chosen in collaboration with the contractor Skanska, from which most of the information was obtained. The information consisted of interviews, blueprints, documentation, protocols, measurement data but also a study visit. The focus was on attaining information needed for the modelling e.g. ground condition data, piling order, amount of piles, dates of piling, soil movement measurements.

The FE-model was created using the program PLAXIS 2D version AE.02. Theories about constitutive models and documented cases on how to model were investigated. A more extensively monitored reference research project, the Partihall highway bridge, was modelled and the key lessons from this were implemented in the FE-model of section 100/130 of Gamlestadstorget. Geometry and parameters for the model was based on data from the project site, field measurements, pre-investigations and the FE-modelling theory.

## 1.4 Scope

The thesis is investigating the horizontal and vertical soil displacements of section 100/130 at the Gamlestadstorget construction project. The main focus is how the pile driving of pre-cast concrete piles in pile group -13 effects the displacements, but the effect from bored steel pipe piles in pile cap foundation -6615 and -6616 are also considered. The investigation is mainly performed by using the numerical analysis program PLAXIS 2D combined with both the linear elastic and Mohr-Coulomb material models. However, a semi-analytical method by Sagaseta and an empirical method by Rehnman for calculating soil displacements are also used for comparison.

# 2 Literature Review

When driving a pile into the soil, different mechanisms occur. Some of these can lead to damage on nearby structures and ground reinforcements, predominantly caused by soil displacement. The choice of piles and pile installation technique is largely affecting the sum of soil displacement that occur. By the use of analytical methods, estimations can be made on the magnitude of the displacements. Thus it can be assessed whether the soil displacements are within allowed limits and if the choice of piles and installation technique is reasonable.

The following chapter is a review including; what effects the soil experiences during piling, the differences between hammered pre-cast piles and bored piles, the methods of Rehnman and Sagaseta to predict soil displacement and FE-modelling of piles.

## 2.1 Effects from piling

There are several different effects caused by pile driving, this chapter includes soil displacement, physical processes occurring when driving a pile into the soil as well as the effects from soil displacement on adjacent piles.

### 2.1.1 Soil displacement

Driving a pile into clay will cause both horizontal and vertical movements (Massarsch and Wersäll, 2013). The movements, also called displacements, can appear as both heave on the ground surface and lateral displacements in the ground. However, the lateral displacements require more effort to monitor than heave but could be more harmful to nearby foundations and installations in ground.

Several studies have been performed on pile driving in clay. Hagerty and Peck (1971) stated that about half of the heave volume can be seen inside the pile foundation area while the second half is found outside. Wersäll and Massarsch (2013) state that heave close to the pile are relatively small and that the lateral soil displacements is depending on the cross section and spacing of the piles. Sagaseta and Whittle (2001) utilises the assumption that the volume of the surface heave is equal to the volume displaced by the pile in undrained conditions. They also mention that even though the heave is small beyond one to two pile lengths distance, it still ads up to about 30 to 40 per cent of the total displaced volume.

Massarsch (1976) presented a theory about the displacement field, consisting of six areas, adjacent to a single pile in soft clay in order to understand the soil disturbance for a driven pile, see Figure 2.1. Disturbance zone one, at the pile toe, is a pressure zone and is the most important one with regard to ground movements in incompressible soil. The zone, created when driving the pile downwards, have a size extending approximately three times the pile diameter below the pile toe, one pile diameter above and a width of about three pile diameters. The soil is displaced lateral from the pressure zone. Zone two is the zone between the pile shaft and the adjacent soil. A thin zone called the smear zone is created, due to the structure of the soil being destroyed, by the movement of the shaft. Zone three describes the disturbance zone within one pile diameter from the pile shaft where mechanical disturbance occurs and the displacement is primarily in lateral direction. In zone four the displacement is initially lateral but slowly turns toward the surface further away from the pile shaft. Zone five is the heave zone, where the displacements close to the pile is small. The maximum heave is reached at a distance of 0.3 to 1.0 times the pile length and decreases further away from that distance. At the top of the pile, zone six, it is common to find a gap or depression between the shaft and the surrounding soil which is caused by the pile toe at the initial phase of driving.

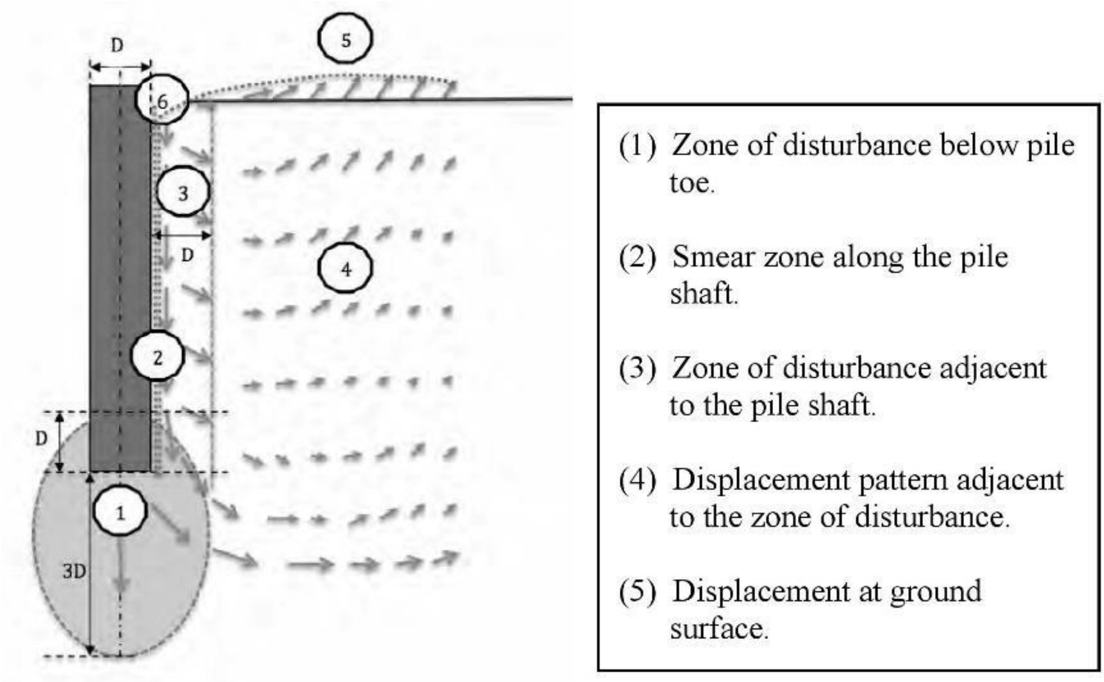



*Figure 2.1* *Overview of the six areas of soil disturbance and the displacement field adjacent to a pile (Massarsch and Wersäll, 2013).*

### 2.1.2 Physical processes during pile history

The physical processes of the soil during the pile history could be divided into the three phases installation, equalization and loading according to Ottolini, Dijkstra and Van Tol (2014). The schematic figure of how the displacements of the soil occurs could be seen in Figure 2.2. The authors describe the phases accordingly: In the initial installation phase the pile is driven into the ground causing the soil to displace away from the pile. This leads to an increased mean total stress in the soil and also an increase in excess pore pressure since the installation process is considered to be undrained and no volume change is occurring during undrained loading. The driving causes remoulding and disturbance of the soil around the tip of the pile (Randolph et al., 1979). The equalization phase is governed by consolidation around the pile, due to a dissipation of the excess pore pressure and the change of mean effective stress. In the loading phase the load of the pile head is transmitted to the soil.

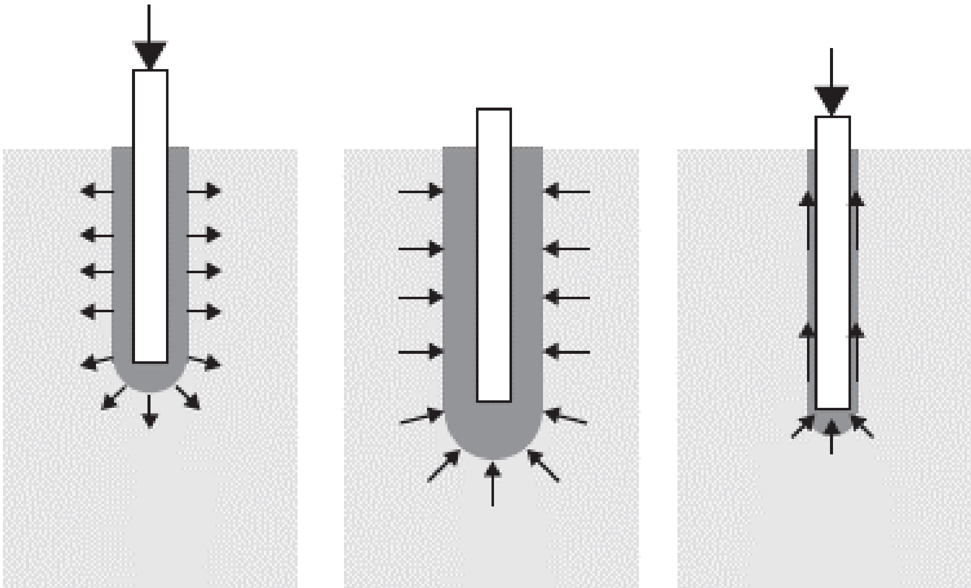


*Figure 2.2 A schematic figure of the physical processes during pile driving history (Ottolini et al., 2014).*

### 2.1.3 Effects on adjacent piles

Pre-existing piles in the ground affect the magnitude of the displacements for a driven pile. The existing pile acts as a reinforcement and counteracts the displacements, especially the heave. The heave is instead concentrated closer to the driven pile. The existing pile itself is also effected and heaves but is also moved with the displacement in lateral direction (Massarsch, 1976). The behaviour could be seen in Figure 2.3.

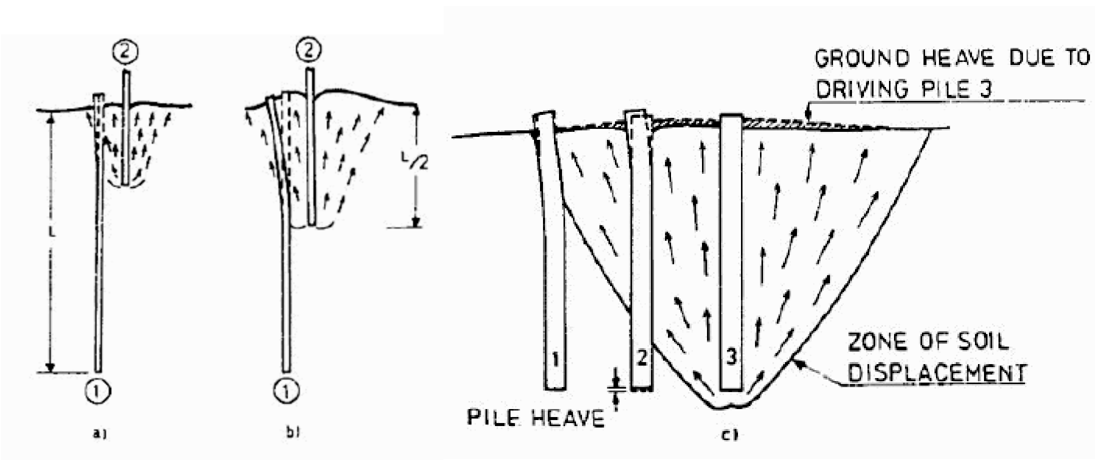



*Figure 2.3 Effects of the adjacent pre-installed pile when driving a pile into soil (Wersäll and Massarsch, 2013).*

## 2.2 Piling techniques

In the year of 2014, sixty per cent of the piles installed in Sweden were pre-cast concrete piles, while thirteen per cent were bored steel pipe piles (Commision on pile research, 2015). The function of piles is to transfer load from superstructures either to bedrock or through weak compressible soils such as clay (Tomlinson and Woodward, 2014a). Installation techniques and differences between hammered concrete piles and bored steel pipe piles are presented in this chapter.

### 2.2.1 Hammered pre-cast concrete piles

Concrete piles are commonly pre-casted, in a factory, before moved to the construction site. The pre-cast concrete piles exist in various cross sections, such as square and hexagonal. In Europe and Sweden, regular reinforced pre-cast concrete piles are common while in the U.S.A. pre-stressed concrete piles, for avoiding cracks, are more extensively used (Hussein, 1993). The piles can be used as either friction or end bearing piles depending on several different factors such as soil type and depth to bedrock.

The most commonly used dimensions are between six to thirteen metres long with square sections ranging from 235x235 to 450x450 millimetres. When driving pile to larger depths, joints are used and connected to each other e.g. using a steel bayonet joint as seen in Figure 2.4 (Tomlinson and Woodward, 2014c).

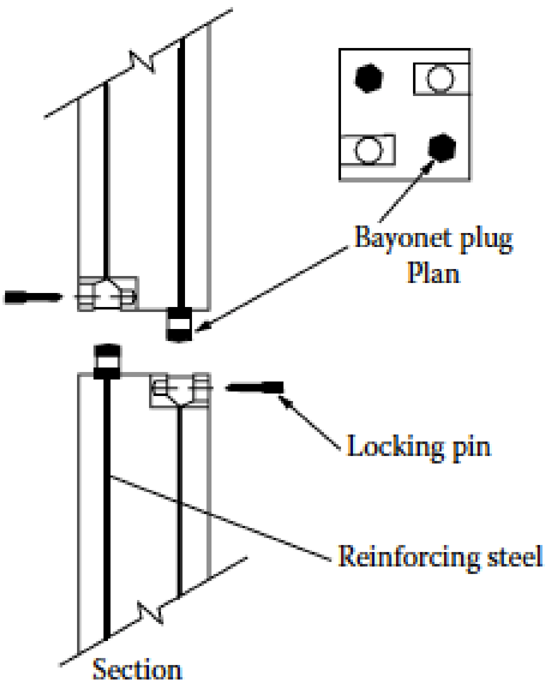



*Figure 2.4 Joint of two pre-cast concrete piles locked together with a bayonet plug (Tomlinson and Woodward, 2014c).*

The common installation technique of concrete piles is performed in a few simple steps (Dept of the Army, 1985). The machine is first of all brought into the right position with the hammer and cap in its top position. Secondly the pile is aligned to the leads with lashes and centred under the pile cap and hammer. Lastly the hammer is raised and dropped on the pile in a rapid but controlled pace causing the pile to move downwards into the soil.

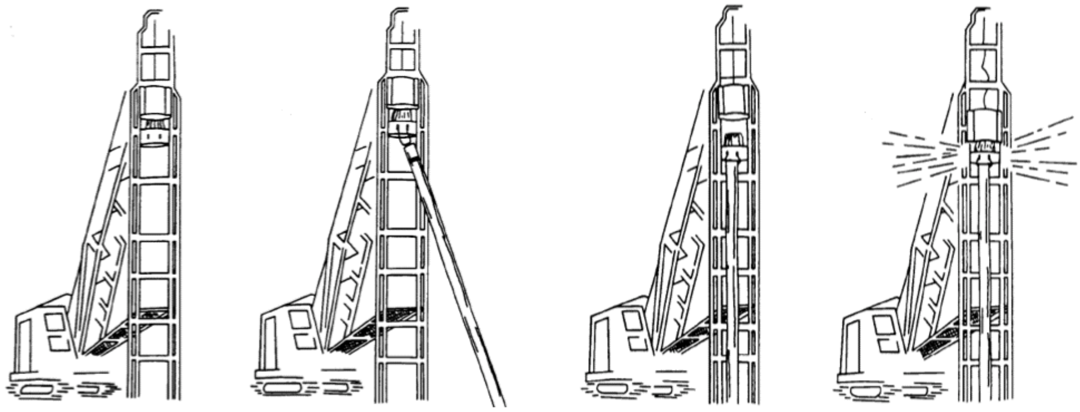


*Figure 2.5 Modified figure from Department of the army (1985) visualizing how the driving of the concrete piles are conducted.*

**Pre-augering**

The effects of pile driving in cohesive soils, such as deformations, can be reduced by pre-boring in the layers closest to the surface (Sagaseta and Whittle, 2001). An auger screw is commonly used in the pre-boring procedure. The screw is constructed to remove a soil volume equal to that of the screw itself, without compacting the soil wall. When utilising the auger screw, the procedure is to first drill to the desirable depth, usually eight to ten metres, and then reversing it to the surface. The reversing movement leads to removal of the soil. Pre-augering is also beneficial in increasing the speed of pile driving and reducing the surface heave. Studies conducted by Sagaseta and Whittle (2001) has shown that pre-augering can have a substantial effect on reducing surface heave, at half a pile length distance it can be reduced by 50-90 per cent.

**Advantages and disadvantages**

Some of the advantages with concrete piles, according to Tomlinson and Woodward (2014c), are; the ease of verifying the quality of the pile material, that the groundwater is not affecting the installation operation, that the pile can be driven in long lengths and be re-driven if problems with heave is noticed. Some disadvantages are; that the pile could break during driving, that adjacent structures and adjacent piles could be affected by displacements and that vibrations and sound caused by driving may reach unacceptable levels.

## 2.2.2 Bored steel pipe piles

Bored pipes, especially steel core piles, were introduced in the Nordic countries in the beginning of the sixties and has successively developed since then. The bored pipe pile has similar characteristics to its ancestor, having a steel pipe bored down to bedrock, but differs in the aspect of being filled with concrete instead of having a steel core. The most common type of steel pipe piles, RD-piles, have dimensions varying between 100 to 813 millimetres in diameter and width dimensions varying between five to sixteen millimetres. (Commision on pile research, 2010).

**Installation methods**

The piles are installed by boring down to bedrock of adequate bearing resistance and simultaneously removing the soil to the surface. The boring could be performed in two ways, by eccentric drilling or by centric drilling. The centric drilling, see Figure 2.6, are in general expected to give straighter bore holes and have a better ability of penetration than eccentric drilling. The difference between the two methods are mainly whether the drill bit is either welded at the pile tip and left in the ground, centric drilling, or withdrawn after reaching desired depth, eccentric drilling (Commision on pile research, 2010).

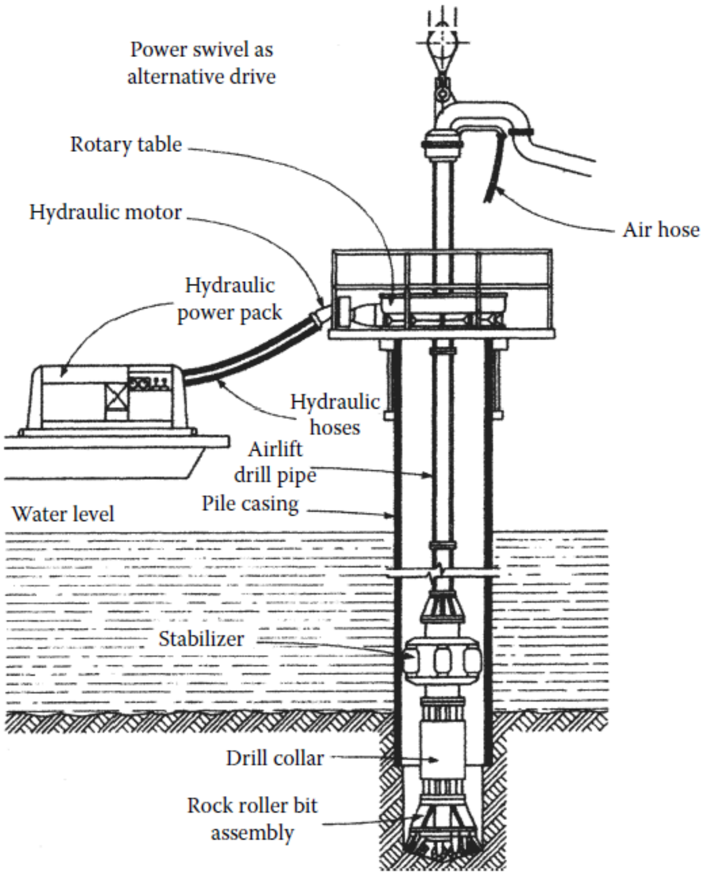



*Figure 2.6 Schematic figure of a drill with centric drilling and reversed circulation (Tomlinson and Woodward, 2014b).*

Different types of hammers are used in the installation to support the boring. The use of Down-The-Hole, DTH, hammers or top hammers is to create a downward movement of the pile. The DTH hammer is working at the pile tip, which reduces the noise and effectively creating a movement downwards. One disadvantage with the DTH hammer is that flushing of soil volumes larger than the pile volume could arise in fine grained soils beneath the ground water level (Commision on pile research, 2010). The air released from the hammer, at the pile tip, can cause disturbance to the surrounding soil as seen in Figure 2.7, but by mainly flush with water instead of air, the impact could be lowered (Langford et al., 2016). The top hammer do not have the same risk associated to flushing as the DTH hammer has, since the pressure surge is applied on the top. However, the pressure surge is attenuated on the way down to the pile tip, where the mobilised force creates a downward movement. The top hammer is therefore limited to smaller dimensions, usually less than 168 millimetres and to depths not greater than 15 metres (Commision on pile research, 2010).

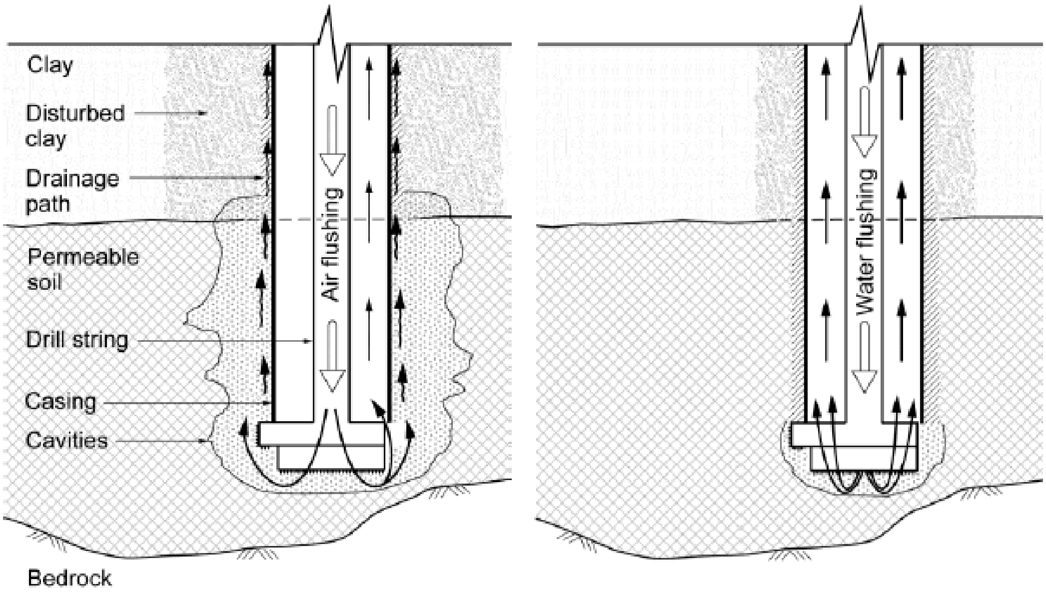



*Figure 2.7 Left: drilling causing extensive soil disturbance when flushing with air and high pressure. Right: ideal drilling using water flushing and low pressure (Langford et al., 2016).*

**Advantages and disadvantages**

There are several advantages associated to using bored piles according to Tomlinson and Woodward (2014c). The method does not cause ground heave, the installation could be done without excessive noise and vibrations and the removed soil or rock could be compared to data from site investigations. The two main disadvantages are local sinkage due to seepage, which can cause settlement of adjacent structures, and loss of ground caused by drilling a number of piles in a group.

## 2.3 Calculation of soil displacement

There are several different methods proposed for the calculation of soil displacement and heave. Below the empirical method proposed by Rehnman, commonly used in Sweden, is explained as well as the more advance semi-analytical method proposed by Sagaseta, used by Skanska for predictions on heave and lateral movement in the project Gamlestadstorget.

### 2.3.1 Rehnman's method

The method was proposed by Sven-Erik Rehnman (Olsson and Holm, 1993) as a development of earlier empirical studies. It was, according to Edstam (2011), considered to be the normal practice used in Sweden when calculating soil displacement caused by piling. The basis are the assumption that heave will occur in an area around the piles of a pile group limited by an angle of 45˚ from the pile bottom, as seen to the right in Figure 2.8.

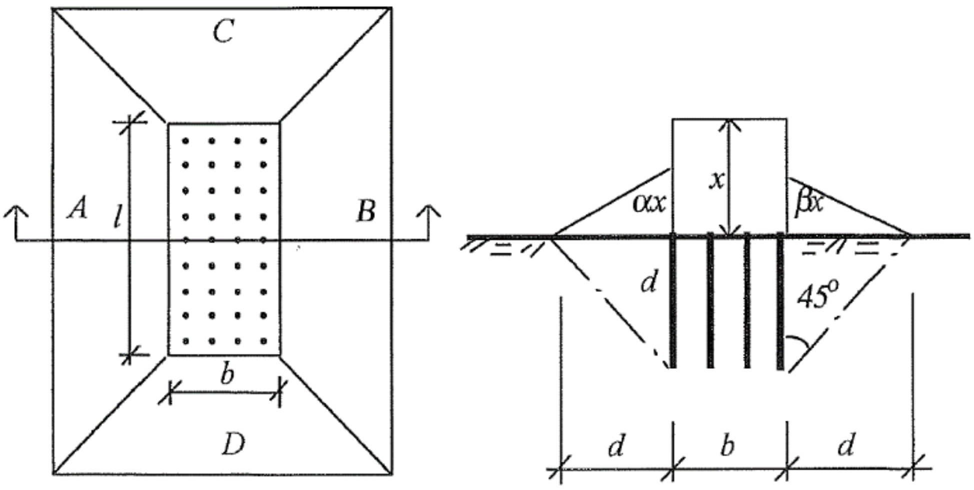



*Figure 2.8 Left: Piling area denoted b*l within the four areas A, B, C and D. Right: Heave, x, caused by pile driving in clay with horizontal ground surface (Hintze et al., 1997).*

The maximum amount of heave, *x*, that will occur inside the piling area, *b*l*, as seen to the left in Figure 2.8, is calculated according to equation 2.1. Due to clays being compressible to some extent, not all the volume from the installed piles will result in heaving, which is why a heave factor η is used (Olsson and Holm, 1993). The factor is normally assumed to be 0.75 but can range from 0.5 to 1.0. The relative load from buildings in the four areas *A*, *B*, *C* and *D*, seen in Figure 2.8, is also considered to estimate the heave distribution. This is in the equation performed using the factors α, β, γ, δ where the relative weight of the buildings in each area is described by a range from 0, for heavy buildings, to 1, for light buildings.

$$x = \frac{\eta(V_{piles} - \Delta V_{pb})}{d\left[(\alpha+\beta)\left(\frac{l}{2}+\frac{d}{3}\right)+(\gamma+\delta)\left(\frac{b}{2}+\frac{d}{3}\right)+\frac{bl}{d}\right]} \quad (2.1)$$

where:
x = heave within piling area
η = heave factor, normally 0.75
$V_{piles}$ = volume of driven piles
$V_{pb}$ = volume of clay removed with pre-boring
α, β, γ, δ = relative load of buildings
d = depth of piles below ground surface
b = width of piling area
l = length of piling area

The horizontal displacement resulting from the pile installation is according to Rehnman (Olsson and Holm, 1993) considered to be correlating to the amount of heave. Thus can accordingly be calculated from the above equation combined with relationship in Figure 2.9.

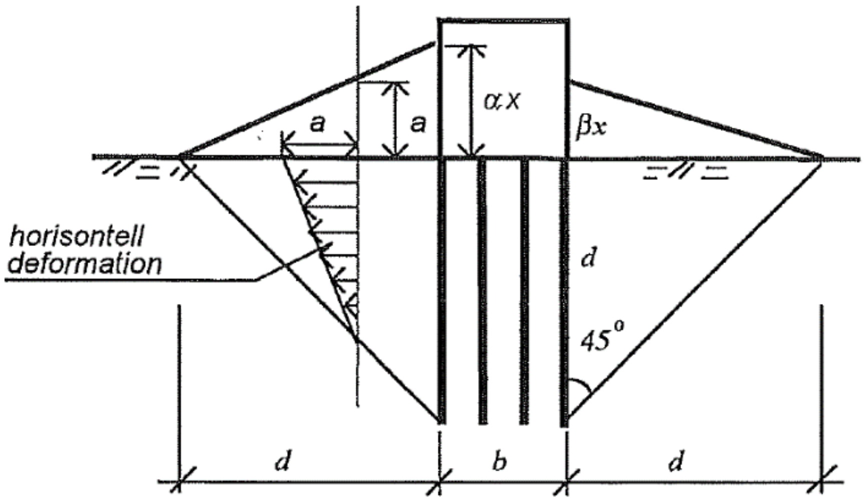



*Figure 2.9 Horizontal soil displacement caused by pile driving in clay (Hintze et al., 1997).*

### 2.3.2 Sagaseta's method

This method is also referred to as the shallow strain path method and is a semi-analytical solution based on a previously developed framework, the strain path method. It was introduced by Sagaseta in 1987 but further developed and described by Sagaseta, Whittle and Santagata (1997) as well as Sagaseta and Whittle (2001). It is used for calculating predictions of ground movements caused by driven or jacked pile installation in clay. By utilizing a set of assumptions, i.e. the clay being undrained and having a stress-free ground surface, both horizontal and vertical deformations can be calculated (Sagaseta and Whittle, 2001).

The installation of a pile is simulated in three steps, see Figure 2.10, which are added together for a final soil displacement analysis (Sagaseta and Whittle, 2001). The first step is executed by assuming a point source penetrating the soil for the full pile length, i.e. the length from the ground surface to the toe of the pile. The point source is assumed to discharge a volume of an ideal fluid causing the soil deformations to occur. The presence of a ground surface is ignored in this step. A mirror image sink moving in the opposite direction, as seen in step 2, is introduced to cancel out the normal stresses. This, however, will cause the shear stresses to double. With the last step, step 3, a set of corrective radial shear forces are added to enable for a stress free ground surface to be modelled.

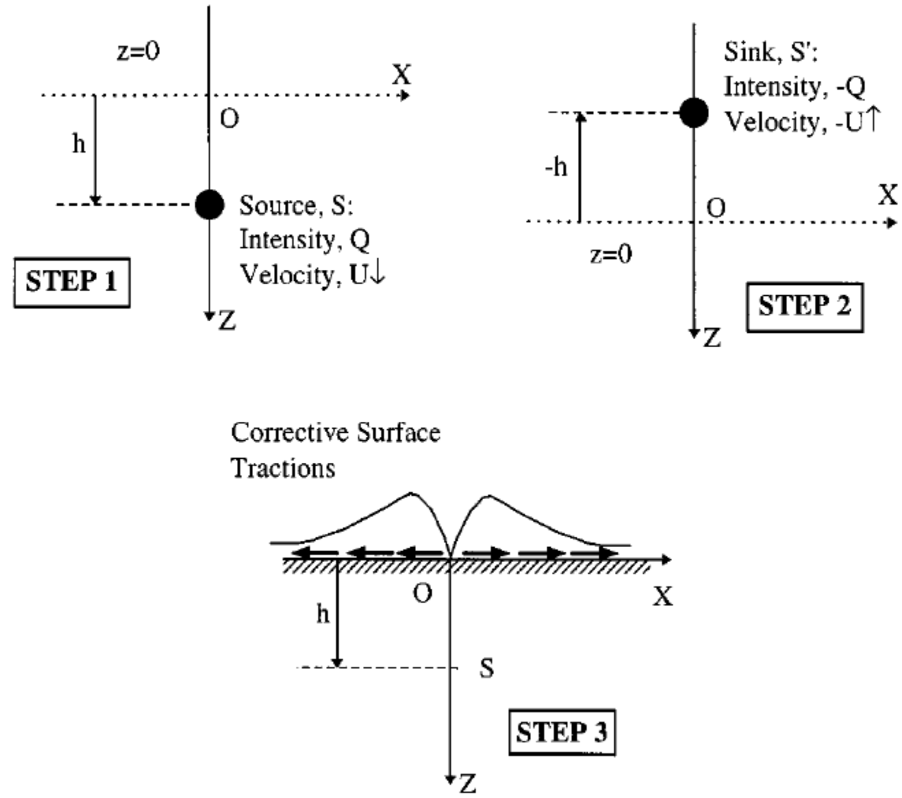



*Figure 2.10 Modified conceptual model of the three steps used in the Sagaseta method (Sagaseta et al., 1997).*

If the displacements of interest are close to the pile, large strain theory is required to be used for the calculations which can then only be solved numerically (Sagaseta and Whittle, 2001). At a certain distance from the pile however, small strain solutions can be used as a good approximation. Thus allowing for closed-form expressions for three geometries; planar wall, cylindrical pile and simple tube.

For the installation of a single cylindrical pile, the horizontal and vertical displacements at ground surface can be calculated through equation 2.2 and equation 2.3 (Edstam and Kullingsjö, 2010).

$$\delta_h(r,0) = \frac{R^2}{2} * \frac{L}{r * \sqrt{r^2 + L^2}} \tag{2.2}$$

$$\delta_v(r,0) = -\frac{R^2}{2} * \left(\frac{1}{r} - \frac{1}{\sqrt{r^2 + L^2}}\right) \tag{2.3}$$

where R = pile radius, L = pile length and r = radial distance from pile.

The effects from pre-boring can be taken into account in the original integral equations simply by changing the length of the pile. The pile length considered is then the length from the pre-boring depth to the final pile depth (Sagaseta and Whittle, 2001). This is however not possible to integrate into the analytical equations, 2.2 and 2.3, above.

The Sagaseta method is designed for accurate predictions in large depth clay deposits, when installing friction piles. But in the case of end-bearing piles or when friction piles are installed with the tip close to bedrock, the soil displacements behave differently. To incorporate this into the analysis, Sagaseta and Whittle (2001) propose a solution using the relative settlement between the ground surface and the bedrock to calculate the resulting displacements.

To achieve the total soil displacement caused by a pile group, the effect from each pile can be superpositioned together. A pile group can also be simplified by forming a super pile, equal to a defined set of piles, to achieve a reasonable accurate approximation as shown by Edstam and Kullingsjö (2010). The super pile is assumed to have the same volume and cross sectional area as the cluster of piles it consists of, with its centre positioned in the pile clusters' centre.

## 2.4 FE-modelling

Finite element method, FEM, was introduced in the 1950s and is a numerical approach that solves differential equations in an approximate way. Complex problems are divided into smaller parts, finite elements, where each element, in relation to the others, are approximated separately. The elements are then assembled, using specific rules, to a complete system and a solution for the whole unity can hence be found (Ottosen and Petersson, 1992). According to Zienkiewicz et al., (2005), FEM applied for civil engineering, can be used to calculate force-displacement relationships.

One of the advantages with FEM is that it can handle very complex geometry and problems, which would be too complicated for analytical methods (De Weck and Kim, 2004). Another advantage is that it can handle complex loadings, such as time or frequency dependant loadings. Some disadvantages with FEM is that the method only obtains approximated solutions and mistakes by users are common and could cause serious problems (De Weck and Kim, 2004). It is important, as a user, to understand the underlying theories behind the model and methods of FEM.

### 2.4.1 Constitutive models

To capture the behaviour of soil, several variations of constitutive models and theories has been formulated. None of these however, capture the behaviour fully in all aspects, since soil is such a complex and varying material (Brinkgreve, 2005). An understanding of constitutive models is essential when interpreting a problem and to be able to choose the right model and their parameters. Hence, follows a description of the linear elastic and the Mohr-Coulomb soil models.

**Linear Elastic model**

The linear elastic model is based on Hooke's law of linear isotropic elasticity, which is the simplest stress-strain relationship (Brinkgreve, 2005). The model contains only two input parameters, Poisson's ratio, $\upsilon$, and Young's modulus, $E$. The advantage with this model is the simplicity of it. However, the model is incapable to capture some important properties of soil, as the behaviour is non-linear and only elastic to some extent, and therefore not a suitable model for soil (PLAXIS, 2016). The model can be used to model massive structures in the soil, however, it is important to keep in mind that stress states in this model are not limited meaning that the model shows infinite strength.

The assumptions about linearity is that the response is proportional to loads, i.e. a doubling in load would lead to a doubling in displacement. The assumption for elasticity is that no deformation occurs and the effected solid returns to its original shape after unloading (Bower, 2012). The stress-strain path for the model is shown in Figure 2.11.

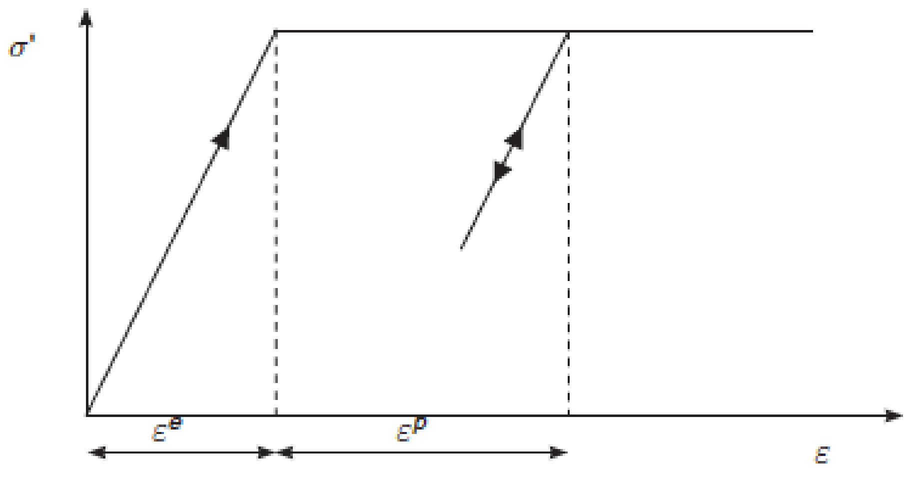



*Figure 2.11 Stress-strain behaviour for a linear-elastic solid (PLAXIS, 2016).*

**Mohr-Coulomb model**

The Mohr-Coulomb model is a combination of Hooke's law, described for the linear elastic model, and the Mohr-Coulomb's failure criterion (Brinkgreve, 2005), see Figure 2.12. The model is an elastic perfectly-plastic model and contains five input parameters.

These parameters are; Young's modulus, $E$, Poisson's ratio, $\upsilon$, the friction angle, $\varphi$, cohesion, $c$, and the dilatancy angle, $\psi$. While the parameters $\varphi$ and $c$ are extracted from the Mohr-Coulomb failure criterion, the dilatancy angle is decided based on the permanent alteration of volume due to shearing (Brinkgreve, 2005).

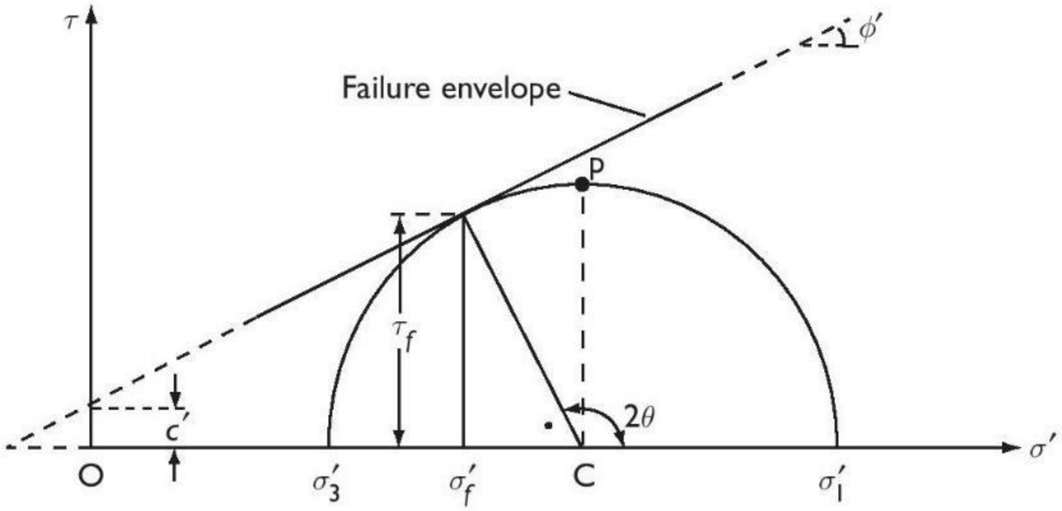



*Figure 2.12 Mohr-Coulomb failure criterion in stress-strain relationship (Craig and Knappett, 2012)*

The advantage with the model lies in its simplicity which makes it suitable to use as a first order model. The model also captures the failure behaviour fairly well while the behaviour before failure is less accurately captured since the stiffness behaviour before failure is assumed to be linear elastic (Brinkgreve, 2005). For example, if many different stress paths are followed or if the stress levels are changing. The Mohr-Coulomb model is appropriate to analyse stability for geotechnical structures, such as dams, slopes and embankments. A disadvantage is that the model lacks the ability to model the softening behaviour of the soil.

### 2.4.2 Pile modelling

There is no fully established and verified method for modelling the installation effects of piles in PLAXIS 2D. There are however different ways of modelling piles and the modelling can either be performed in a plane strain or axisymmetric model, both with their respective disadvantages.

**Plane strain and axisymmetric modelling**

The difference between the plane strain model and axisymmetric model can be seen in Figure 2.13. Which model to use, is mainly depending on the geometry that is to be modelled and stress state to capture.

The plane strain model is used for when the geometry has a relatively uniform cross section with an equivalent stress state and loading scheme in the z-direction, i.e. perpendicular to the cross section. In the z-direction the displacement and strains are assumed to be zero, while the normal stresses are taken into account. With this model, the piles will achieve the behaviour equivalent to that of a wall, extending indefinitely in the z-direction with no spacing in between.

The axisymmetric model however, is used for modelling circular structures, where the cross section, radially, is relatively uniform and has a loading scheme around a central axis. It is assumed in this model that the stress state and deformations are the same in any radial direction. Consequently, this is a way of modelling a three dimensional problem by means of a two dimensional model. For a piling problem however, this model can only capture the behaviour equivalent to a single pile or smaller pile group.

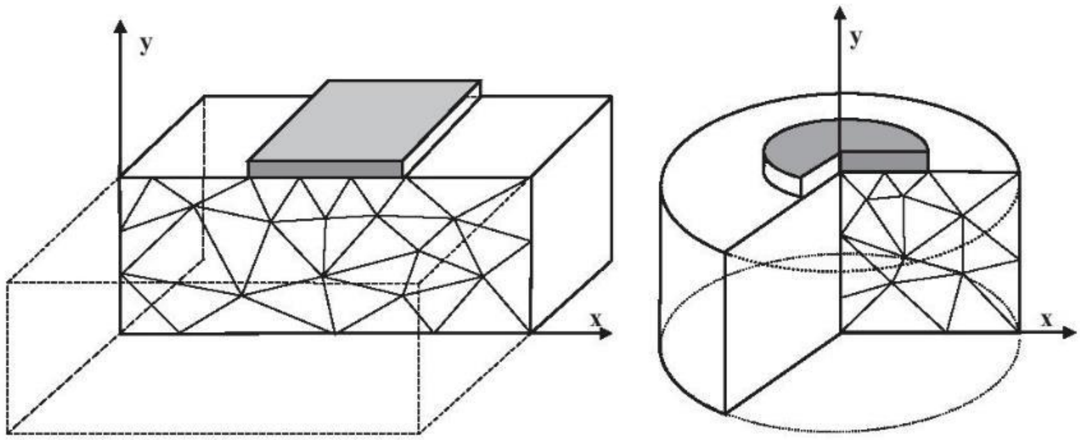



*Figure 2.13 Principe of plane strain (left) and axisymmetric (right) modelling (PLAXIS, 2016).*

**Modelling using volumetric strain**

The volumetric strain function could be used to simulate the volume increase of the clay caused by pile installation. PLAXIS allows for input of strains in x-, y- and z-direction. By applying a volumetric strain to a soil cluster, the program adjusts stresses and forces in the surrounding soil and achieve an output where a stress equilibrium occurs between the boundary conditions and the relevant cluster (PLAXIS, 2016). This means that the final stress level and deformations obtained is determined by the ratio of stiffness between the cluster with volumetric strain and the surrounding clusters. Which is why a larger volumetric strain is sometimes needed to be prescribed for a certain amount of final expansion, assuming different properties are used in the clusters.

Modelling piles by using volumetric strain, especially lime-cement columns, has been done by various authors with good result. Klasson and Kristensson (2012) did for example use the method to model lime-cement columns in PLAXIS 2D. The displacements of the model were compared to the analytical calculation method for a planar wall by Sagaseta and the measured displacement. It was shown that these methods complied well with measured displacements. In the masters' thesis by Nenonen and Ruul (2011), the authors are modelling concrete piles in PLAXIS 3D using the concept of super piles combined with the volumetric strain function. By using this approach, the authors were able to get a good correspondence of the soil behaviour with regard to the measured displacements.

### Global behaviour of piles

The global behaviour of the piles, being a wall, pile row or a single pile, are dependent on the ratio between the centre to centre distance, c/c, and the diameter of the piles (PLAXIS, 2016). A ratio, c/c-distance divided by diameter, of one is equal to the piles standing next to each other, thus acting like a wall. Increasing that ratio would, at a certain point, lead to a change of global behaviour from a wall to a pile row, meaning that the soil is able to move in between the individual piles. Indications of when this happens is depending on the soil, however the PLAXIS manual (2016) mention a ratio between 1.5 to 5 as typical for when the global behaviour of a pile row occurs. Increasing the c/c-distance further than that will in the end give a global behaviour of a single pile, where the piles are no longer influened by each other.

### The concept of super piles

In the analytical method by Sagaseta, both the horizontal and vertical displacements are calculated for when a single pile is installed. The total effect of a pile group is given by superpositioning the effect of each individual pile. In the SBUF-report *Massundanträngning i samband med pålslagning i lera* by Edstam (2011), the author investigated if a simplification of the Sagaseta method was possible by using so-called super piles. The concept of a super pile is to replace a group of piles with one super pile placed in the centre of the group, giving it the same cross sectional area and volume as the sum of the piles replaced. The investigation showed that the concept of super piles was acceptable for ground movements outside of the piling area. An extension of the investigation was made to see if the simplification could be applied to FE-modelling as well. The results in PLAXIS 3D exhibited similar ground movements as the analytical method as long as the focus is on an area outside of the piling area.

# 3 Verification of Pile Modelling

Since there is no verified and established process of modelling soil displacements due to piling in PLAXIS 2D, there was a need to find an approach that captures this behaviour. To do this, a reference project was used where different approaches could be tried out with the goal to get as good agreement as possible with the field measurements.

## 3.1 Reference project

To find an accurate way of modelling the soil displacements caused by piling in PLAXIS 2D, another case study was used for validation. The case used is bridge pier foundation A11 of the Partihallen highway bridge, located close to Gamlestadstorget. The project has been studied in several different publications, e.g. in the SBUF report *Massundanträngning i samband med pålslagning i lera* by Edstam (2011) and in the article *Ground displacement due to pile driving in Gothenburg clay* by Edstam and Kullingsjö (2010). The earlier publications of this case have modelled the soil displacement caused by piling in PLAXIS 3D but never in a 2D model.

The case study has a simple soil geometry and was extensively measured when piling occurred. The foundation consists of 60 pre-cast concrete friction piles installed with a c/c-distance of 1.3 metres, each with a cross sectional area of 275 x 275 square millimetres and a length of 52 metres (Edstam and Kullingsjö, 2010). Vertical and horizontal displacements have been measured at different distances, at the ground surface with settlement gauges and towards depth with inclinometers. The model parameters used can be seen in Table 3.1.

*Table 3.1 Input parameters used in the reference project (Nenonen and Ruul, 2011)*

| **Parameter** | **Linear elastic** | **Mohr-Coulomb** |
|---|---|---|
| Material type | Drained | Undrained |
| $\gamma$ [kN/m$^3$] | 16.5 | 16.5 |
| $G_{ref}$ [kN/m$^2$] | 1100 | 1100 |
| $E_{inc}$ [kN/m$^2$/m] | 358.8 | 324 |
| c [kN/m$^2$] | - | 11 |
| $c_{inc}$ [kN/m2] | - | 1.5 |
| $y_{ref}$ [m] | 4.5 | 4.5 |
| $\nu$ [-] | 0.495 | 0.35 |
| $k_{x,y,z}$ [m/day] | $8.64*10^{-5}$ | $8.64*10^{-5}$ |
| $\phi$' [°] | - | 0 |
| $\psi$ [°] | - | 0 |

The model geometry was configured and set up to replicate the 3D model used in the research performed by Edstam and Kullingsjö (2010). Thus, a depth of 100 metres and 52 metres pile length has been assumed. The inclinometers used for comparison, termed six and three, was located at a distance of 17 and 42 metres from the piling area centre respectively. Detailed calculations and model geometry are found in Appendix 1.

### 3.1.1 Axisymmetric modelling

The axisymmetric case was executed by assuming that all 60 piles act as one large super pile, in the same way as was shown to be most accurate in 3D by Nenonen and Ruul (2011). The pile modelling was performed by assuming a volumetric expansion of a soil cluster with the original size equal to that of the piling areas soil volume, in this case corresponding to a circular radius of 5.745 metres. The size of the volumetric expansion was calculated by adding the total pile volume to the piling areas soil volume, achieving an expansion equal to 4.38 per cent. This approach was performed using both the linear elastic and the Mohr-Coulomb soil model respectively. In this initial investigation, no pre-auger was assumed to be used.

The resulting movement in PLAXIS can be seen compared to the in-situ measured inclinometer movement in Figure 3.1. There is a good agreement with inclinometer three, 42 metres away from the centre line. However, at inclinometer six, 17 metre from the centre line, an overestimation of the movements is attained, especially at the top and the bottom. This could be due to the practice of using a super pile, which can be expected to overestimate the displacements close to the pile.

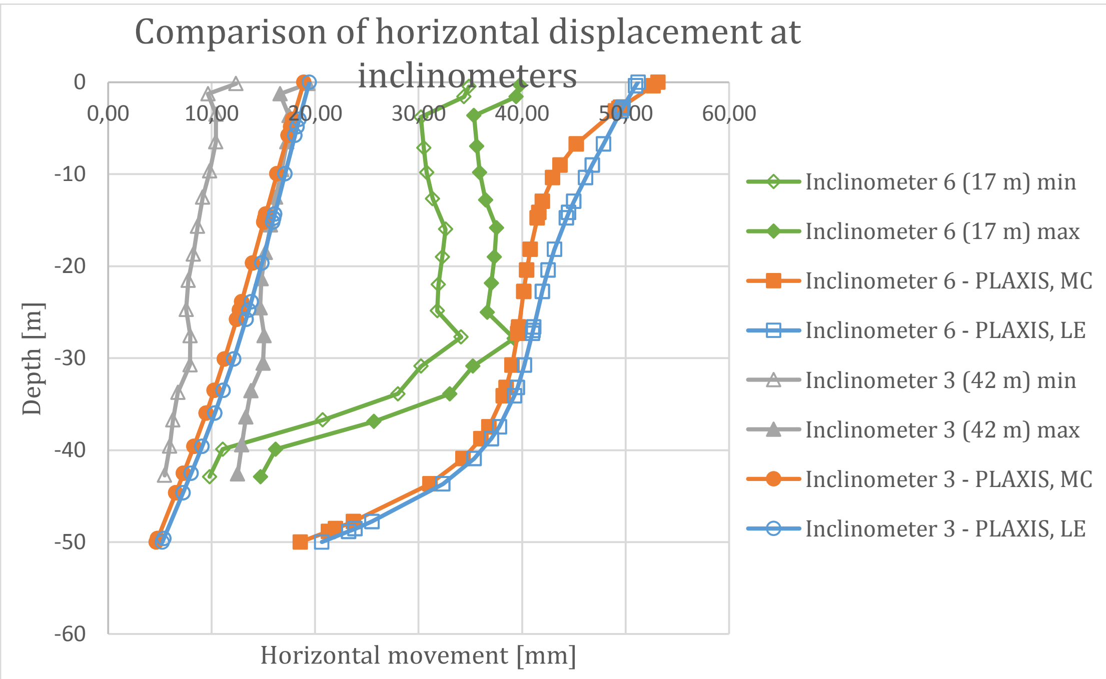



*Figure 3.1 Horizontal displacements at the inclinometers and the difference between the linear elastic and Mohr-Coulomb soil model.*

In Figure 3.2 is a comparison of the vertical displacement at ground surface, measured with ground settlements gauges, compared to the resulting movement attained in PLAXIS. Both soil models give a good agreement to measured data even though the MC model have a slightly more accurate shape.

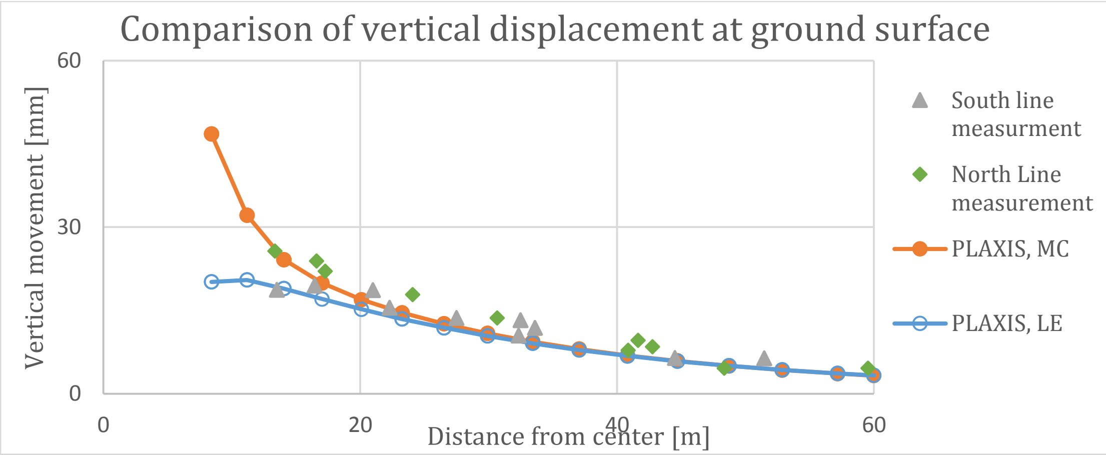



*Figure 3.2* *Vertical displacements at ground level and the difference between the linear elastic and Mohr-Coulomb soil model.*

### 3.1.2 Pre-auger

The efficiency of pre-augering was taken into account by altering the volumetric expansion for the first ten metres of the soil cluster. When an auger screw is used to remove the soil, the efficiency cannot be fully assured. Because of that, different degrees of efficiency have been simulated in the model to find an agreement with the measurements. A 100 per cent efficient pre-auger was taken into account by prescribing zero expansion. While a 50 per cent efficient pre-auger was performed by prescribing 2.19 per cent expansion, equal to half of the calculated expansion. The effect of piling without pre-augering was simulated by using the same expansion for all of the soil cluster. Detailed calculations of this can be found in Appendix 1.

It can be seen in Figure 3.3 that the ten metre pre-augering have an effect on the displacements down to a depth of about 25 metres, but no effect further down. It can also be seen that the case with 50 per cent efficiency give the best agreement when compared to measured data. All simulations for the pre-auger efficiency was performed using the Mohr-Coulomb soil model.

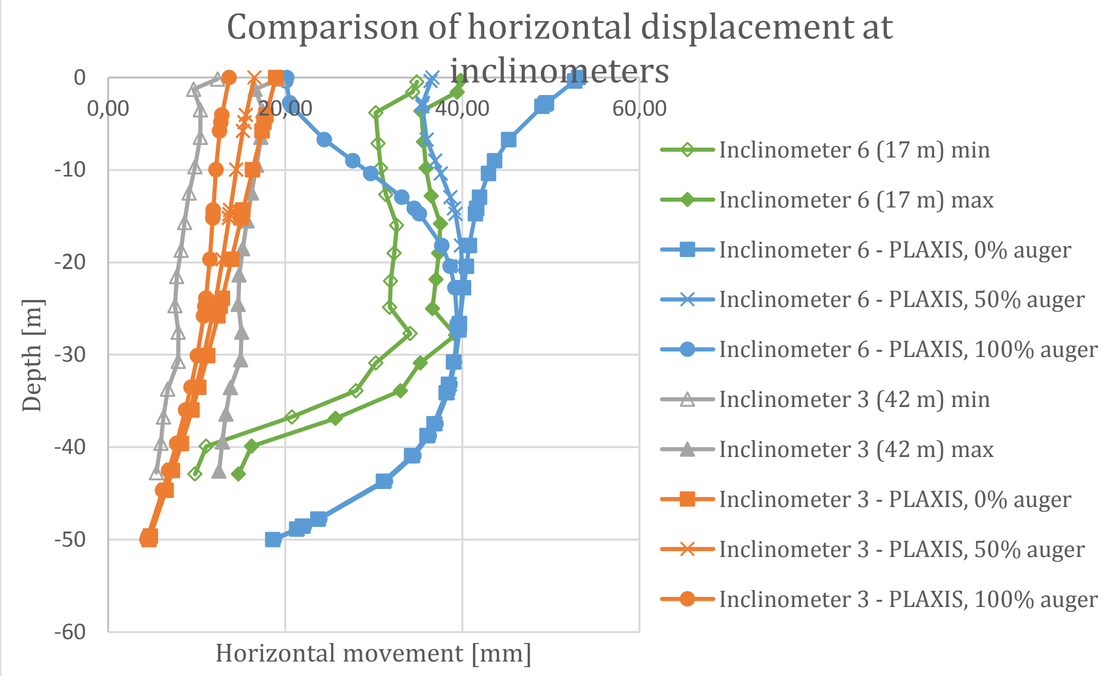



*Figure 3.3* *The effect of pre-augering on the horizontal displacements at the inclinometers. Performed using the MC soil model.*

The resulting vertical movements at ground surface using different pre-auger efficiency can be seen in Figure 3.4. Here it can be seen that the pre-augering affect the displacements up to a distance of about 25 metres from the piling area centre. The difference, between the variations of pre-augering efficiency, only becomes significant closer to the super pile. All variations do however have a good agreement compared to measured data. The 50 per cent effective auger could be assumed to give the most realistic response with a slight increase in displacement at a closer distance to the pile.

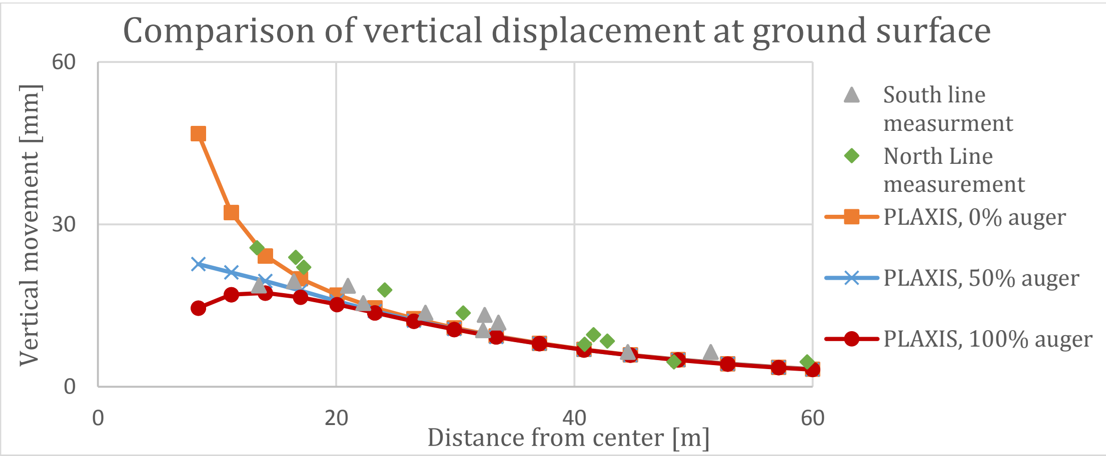



*Figure 3.4* *The effect of pre-augering on the vertical displacements at ground surface. Performed using the MC soil model.*

# 4 Case Study at Gamlestadstorget

The city of Gothenburg is under expansive development and within twenty years, by 2035, the population of the municipality could reach around 700,000 people (Göteborgs Stad, 2014). The city of Gothenburg has chosen five, strategically located, public transportation hubs to develop, to be able to meet the demands from an increased population. One of those public transportation hubs is Gamlestadstorget (Swedish Transport Administration, 2013).

Skanska is one of the contractors, assigned by Trafikkontoret, which is to develop Gamlestadstorget in the district of Gamlestaden. The project, Spårskede 4, involves replacing the old tramway with a new one, constructing two new bridges crossing Säveån and building public transportation stops. The project also involves building a new square and quayside. The construction area at Gamlestadstorget is visualised in Figure 4.1. The surrounding area consists mainly of streets and tracks north and south of the river, Säveån, which itself is running in an east-west direction. To the west of the area the boundaries consist of railway tracks that is crossing Säveån on bridges. (Vectura, 2015b).


*Figure 4.1 The area of construction assigned to Skanska is represented by the red circumscribed area (Vectura, 2015a).*

**History about Gamlestaden**

Between the years 1473 and 1624, the precursor to the city of Gothenburg, Nya Lödöse, was situated where the existing Gamlestadstorget is today (Staden Nya Lödöse, 2016). The town consisted of simple houses made of wood and straw. Due to this, large archaeological excavations have been carried out before and parallel to construction project.

In the 19th and 20th century, the industrialism developed and around Gamlestaden factories, such as SKF, was built. As the factories developed and grew, the district expanded and as a result more houses were built to quarter workers. A solution, for those who did not live in the district, was the construction of the railway that went through the whole district (Bjur, 2015). The existing railway tracks in Gamlestadstorget as seen in Figure 4.1, was at this time a tributary to Göta älv, see Figure 4.2. The flow through this tributary was stopped some time before 1863, while the riverbed was fully filled before 1930 (Sweco, 2011).

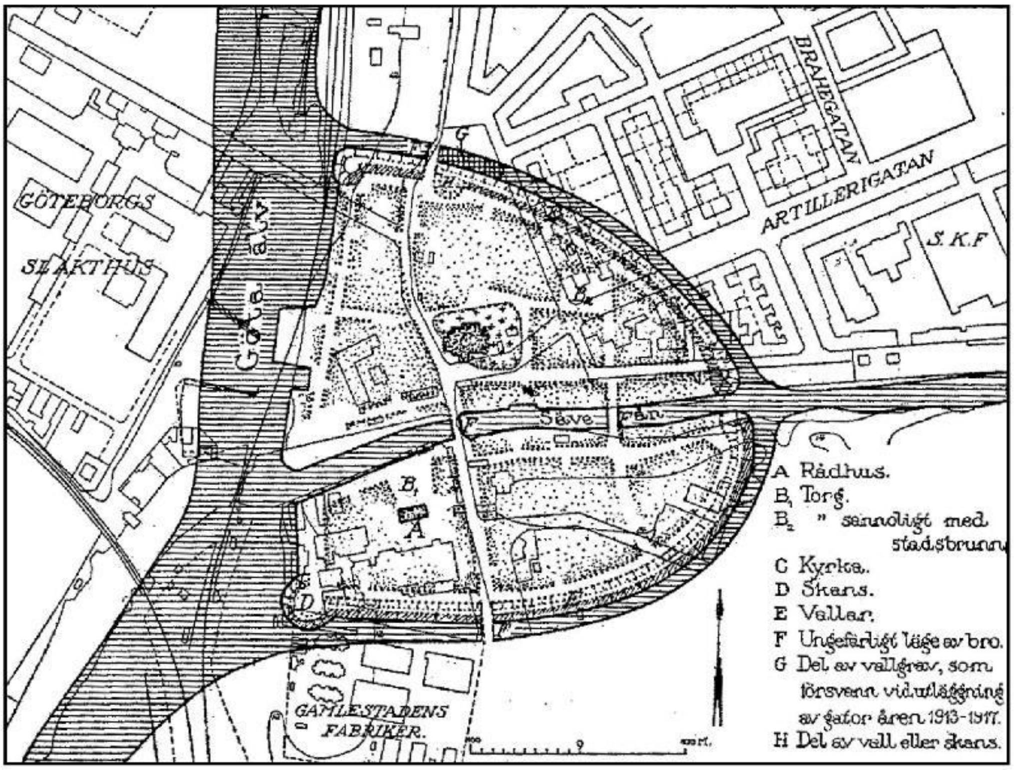



*Figure 4.2 The layout of Göta älv at Nya Lödöse before it was stopped and filled, the existing railway can also be seen (Lilienberg, 1928).*

## 4.1 General ground conditions

The ground surface level in the area varies in level between +1.5 and +4.5. The soil is known to be very prone to settlements and thus sensitive to loading of the ground, i.e. if increasing the fill on top of the soil or lowering the groundwater (Sweco, 2011).

Earth-rock probing, cone penetration test (CPT), pressure probing and un-disturbed sampling have been carried out in order to get a representation of the soil layers. The layers in the area consists generally of fill, fluvial deposits and clay down to a layer of friction soil, which is resting on bedrock. The soil have varying layer depths depending on investigated sections of the area (Vectura, 2015b). South of Säveån the fill is around 1 to 2.5 metres in depth, while the fluvial deposit is around ten metres closest to the river. The fluvial deposit layer decreases in the south-east direction. The clay layer is varying between 60 to 70 metres in the sections around the river and increasing with depth in the southern direction. North of Säveån the fill layer is varying between 0.5 to 5 metres, while the clay layer is varying between 20 to 65 metres. The clay layer depth is lowest in the east and increases in the west-south direction.

The mean water level, MWL, in the river is +0,147 but with a variation of water level as seen in Table 4.1. The river bottom level varies between -3 to -3,5 (Vectura, 2015a).

*Table 4.1 Variations of water level in the river, Säveån (Vectura, 2015a).*

| **Water levels (RH2000)** | |
|---|---|
| $HHW^{100}$ | +1,897 |
| $HHW^{50}$ | +1,747 |
| MHW | +1,067 |
| MWL | +0,147 |
| MLW | -0,453 |
| LLW | -1,053 |

## 4.2 Description of section 100/130

The focus of the thesis and the area to be modelled is section 100/130 which is a cross section through pile group -13 close to pile cap foundation -6616, see Figure 4.3. This section was chosen because of it crossing through pile group -13 and the inclinometer as well as being relatively complex with sloping ground surface and the proximity of the quayside.

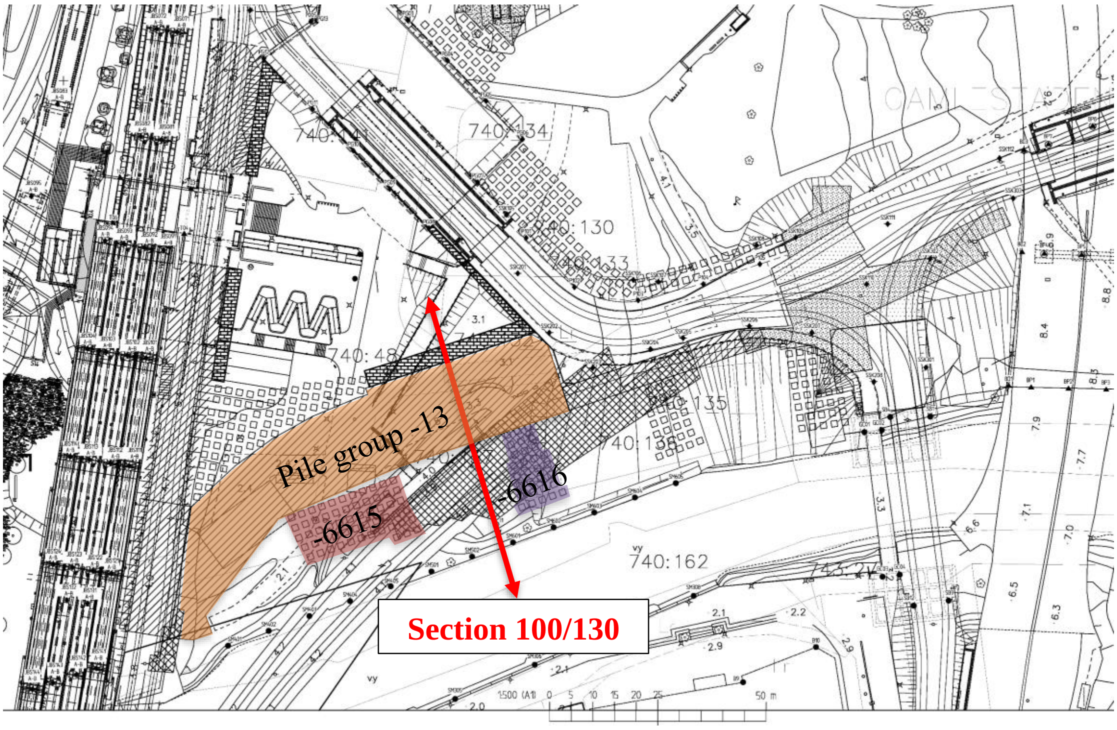



*Figure 4.3 Map of the construction area, where section 100/130 is marked out by the red line through pile group -13, between -6615 and -6616.*

### 4.2.1 Geotechnical conditions

The pre-investigation of the soil at the construction site was, in the construction documentation, divided into five different areas. Section 100/130 is mainly located in area three but does also extend into area one and is bordering to area two. A map of the areas as well as all the testing points can be found in Appendix 2.

The assumed soil layering at section 100/130 can be seen in Figure 4.4. It consists of a top layer of varying thickness, between 0.5 to 3 metres, containing mostly fill material and fluvial deposits. Below the top layer is a silty clay layer with a thickness of about ten metres. The bottom layers consist of clay down to bedrock, reaching a thickness of about 10 to 15 metres. The ground water level has been assumed to be located at a level of +0.5, approximately one to two metres below ground surface.

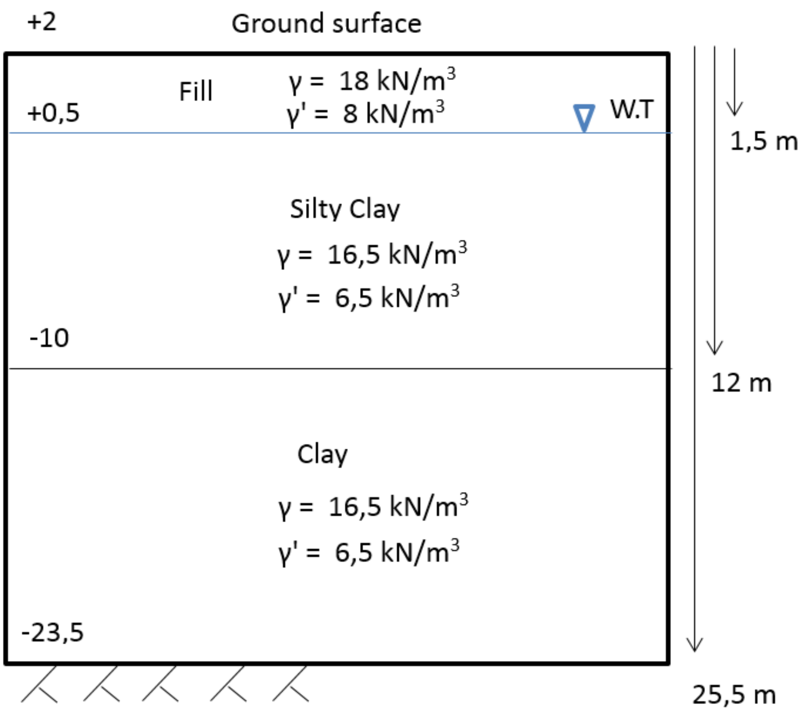



*Figure 4.4 Assumed soil layering, with saturated and unsaturated soil weight, for section 100/130.*

**Unit weight, γ**

The unit weight of the soil is approximated from both earlier construction projects and more recent investigations made in the area of Gamlestadstorget. The results are presented as plots and can be seen in Appendix 3. In general, the clay is quite homogenous, even though small variations are present at the site. Both the clay and silty clay are assumed to have a saturated unit weight, γ, of 16.5 kN/m$^3$ and an effective unit weight, γ', of 6.5 kN/m$^3$, as seen in Table 4.2. The fill material in the area is considered to have a saturated unit weight, γ, of 18 kN/m$^3$ and an effective unit weight, γ', of 10 kN/m$^3$.

*Table 4.2 Unit weights for the different soil types.*

| **Material** | **Saturated soil weight γ [kN/m$^3$]** | **Unsaturated soil weight γ' [kN/m$^3$]** |
|---|---|---|
| Fill/sand | 18 | 10 |
| Silty Clay | 16.5 | 6.5 |
| Clay | 16.5 | 6.5 |

**Liquid limit, $w_L$**

The liquid limit, $w_L$, of the soil is relatively constant, both at different locations in the area as well as with depth, and varies only slightly around 70 per cent. Plots of the liquid limit can be found in Appendix 3.

**Natural water content, $w_N$**

The natural water content, $w_N$, of the soil is also relatively constant, both in the area as well as with depth, and varies around 65 per cent, about 5 points lower than the liquid limit. Plots of the natural water content can be found in Appendix 3.

**Undrained shear strength, $c_u$**

From the evaluation of the investigations, the undrained shear strength, $c_u$, of the soil at the area for pile group -13 has in the construction documentation been interpreted as 25 kPa down to level -20 and then increasing with 1.1 kPa/m, as seen in Table 4.3. A plot of the interpreted undrained shear strength and the data from nearby investigations can be found in Appendix 3.

*Table 4.3 Evaluated undrained shear strength depending on type of material and depth extracted from the construction documentation.*

| **Material** | **Level** | **Soil strength parameters** |
|---|---|---|
| Fill/Sand | Varying | - |
| Clay (Muddy) | to -20 | $c_u$=25 kPa |
| Clay | -20 and below | $c_u$=25+1.1*z kPa (z from -20) |

**Pore water pressure, u**

The pore water pressure in the area is behaving quite linearly and has in general a pore over pressure with depth compared to a hydrostatical distribution. The plot and interpretation of the investigations made in the area could be seen in Appendix 3. There is an average pore over pressure of around 11.5 kPa/m, which is in agreement with the interpretation in the construction documentation seen in Table 4.4.

*Table 4.4 Variation of pore pressure with depth*

| **Level** | **Increase [kPa/m]** |
|---|---|
| +0.5 | Ground water level |
| +0.5 to -3 | 10 |
| -3 to -30 | 11.5 |
| -30 to | 10 |

**Pre-consolidation pressure, $\sigma_c$**

For the construction site of Gamlestadstorget, the pre-consolidation pressure varies slightly depending on investigated location. In Figure 4.5 the derived pre-consolidation pressures can be seen plotted together with the calculated vertical effective stress. It can be seen that the clay is slightly over consolidated all the way down to bedrock with an OCR ranging from slightly above one to about seven in the upper fill layer. The derived pre-consolidation pressure is based on the evaluation of several different locations nearby section 100/130, a graph of the evaluation can be seen in Appendix 3.

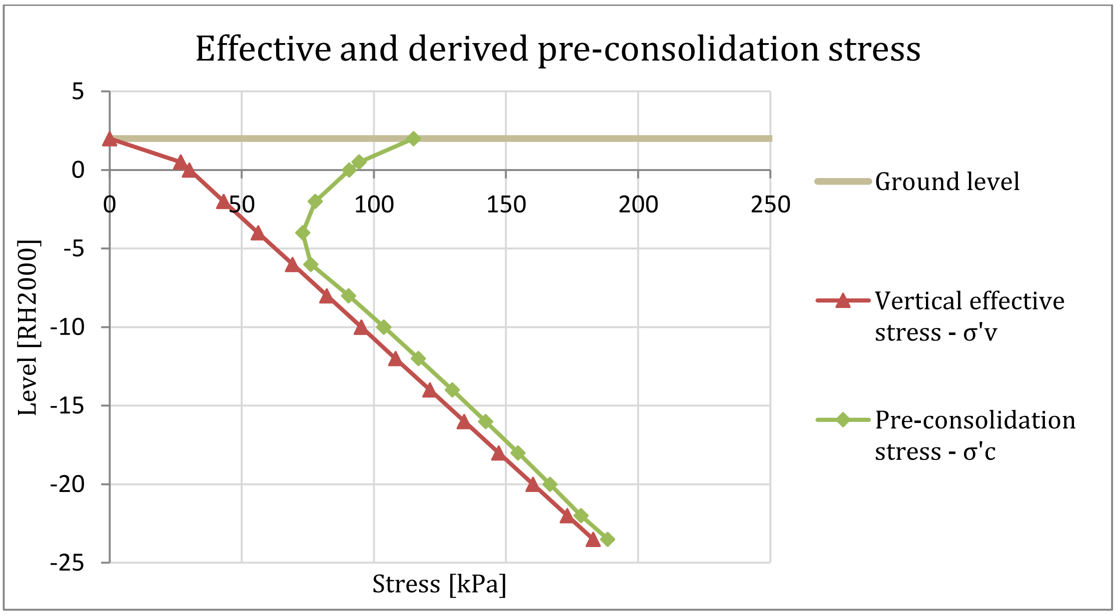



*Figure 4.5 Calculated effective stresses of section 100/130 and pre-consolidation stress derived from ground investigations.*

### 4.2.2 Pre-existing piles and constructions

Earlier activities and constructions in the area have generated a significant amount of piles and other constructions that have been left in the ground. The foundation of the old tramway, which was constructed as a pile cap foundation on a combination of wooden and concrete piles, are located just south of pile group -13. It extends straight through pile cap foundation -6616 and partly through -6615 as well as pile group -13. There are also some larger concrete constructions left in the ground, mainly the abutment of the old tram bridge as well as the existing quaysides. Appendix 4 gives an overview of the pre-existing constructions.

### 4.2.3 Ground improvements

The ground improvement technique mainly used in the area around section 100/130 are hammered concrete piles with a majority of these installed in pile group -13, seen as green areas in *Figure 4.6*. There are however also bored steel pipe piles, seen as blue areas, used in this foundation, mostly in the western end but also the two rows furthest to the east. For the pile cap foundations -6615 and -6616, there are mainly bored steel pipe piles used but with a significant amount of concrete piles as well. An overview of all the pile foundations in the area can be found in Appendix 1.

All the different foundations use the same type of concrete and steel pipe piles. The concrete piles used are pre-cast piles of class SP2 with a cross sectional area of 275x275 millimetres. The steel pipe piles are RD170 piles with a dimension of Ø168.3x12.5 millimetres. Both types are end-bearing and hence driven into the ground down to firm bedrock, the concrete piles being hammered and the steel pipe piles bored down.


*Figure 4.6 Overview of the different ground improvements used, the green areas represents concrete piles and the blue areas represents steel pipe piles.*

**Pile group -13**

The piles in this foundation are installed in a squared pattern with a c/c-distance of 3 to 3.5 metres, see Figure 4.7. In the east, the depth to bedrock are the lowest with pile lengths of 20 metres required while it increases considerably towards the west where piles as long as 72 metres have been installed. Each concrete pile is pre-bored with an auger screw down to a depth of ten metres. A minority of the piles are installed with an angle, mainly to avoid conflict with pre-existent piles in the ground. The full layout of pile group -13 is available in Appendix 1.

A total of 105 concrete piles have been installed, adding up to a total length of 3,292 metres, equal to a total installed volume of 249 cubic metres. The installation of the concrete piles for pile group -13 was roughly carried out by starting from the west and then moving on towards the east. The total installed length of the concrete piles in the west part was 2,173 metres, giving a total installed volume of 164.3 cubic metres. In the east part, the total installed length was 1,119 metres, giving a total installed volume of 84.6 cubic metres.

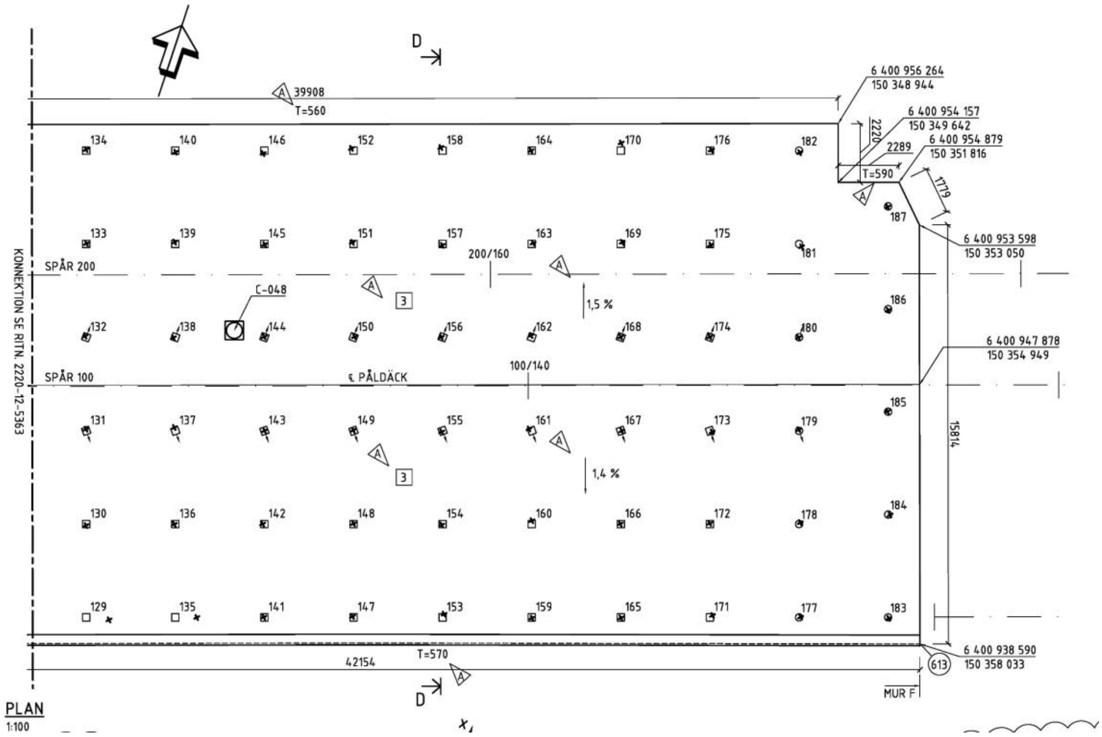
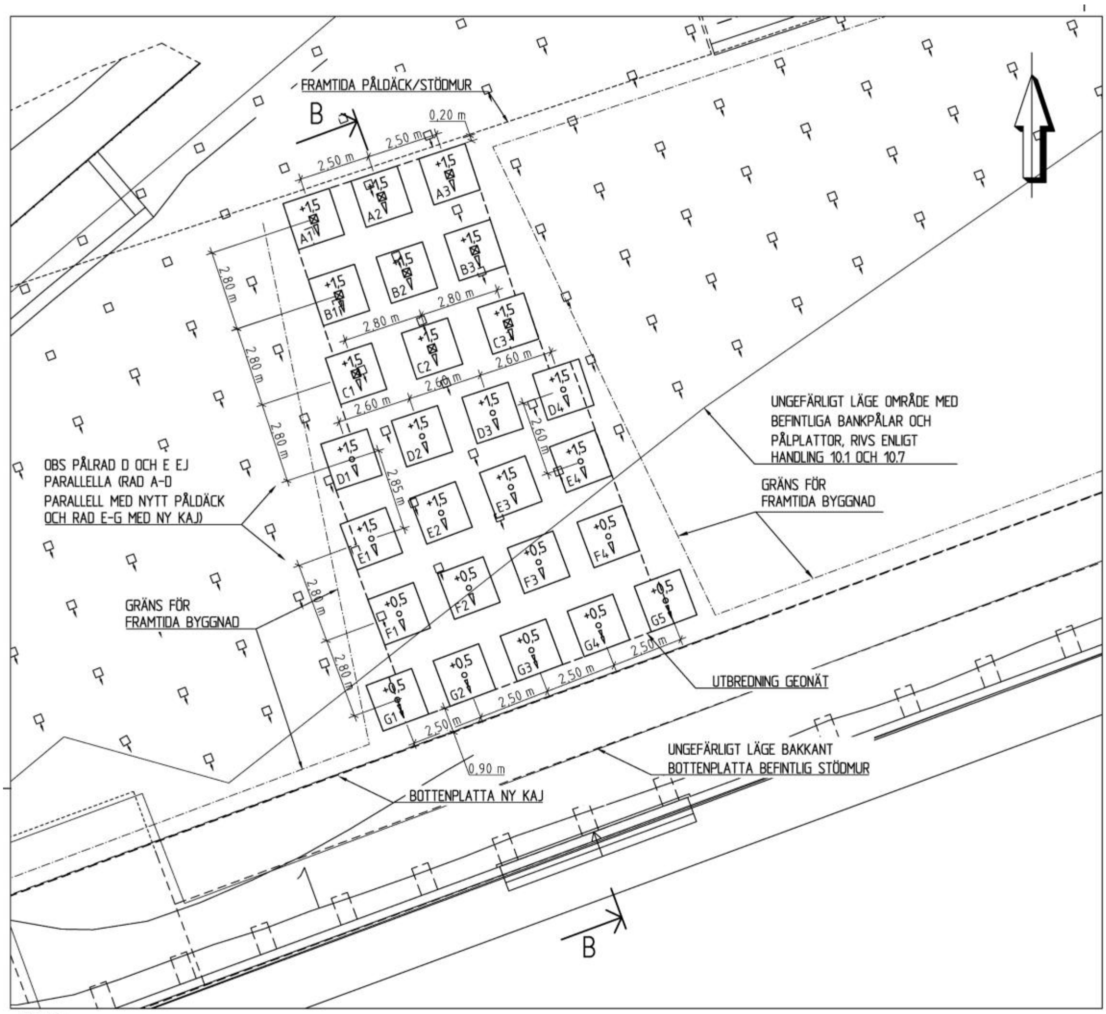



*Figure 4.7* *Layout of the eastern part of pile group -13. The two rows of circles to the right represents steel pipe piles while the squares are concrete piles.*

**Pile cap foundation -6615 and -6616**

There are mainly bored steel pipe piles used in pile cap foundations -6615 and -6616 but also a significant amount of concrete piles. About one third of the piles in -6616 and forty per cent of the piles used in -6615 are concrete piles. Figure 4.8 presents the layout for pile cap foundation -6616 where the different pile types are shown. The figure also clearly shows the pre-existing piles from the old tramway, seen as unnumbered rectangles. The length of the piles varies between 21 metres in -6616 up to 53 metres furthest to the west in -6615. The piles are installed in a pattern with a c/c-distance of between 2.5 to 2.8 metres in both foundations. The full layout of both foundations are available in Appendix 1.




PLAN
1:100 (A1)

*Figure 4.8* *Pile cap foundation -6616, where rows A to C consists of concrete piles and rows D to G of steel pipe piles. The unnumbered rectangles represent the pre-existing piles of the old tramway foundation.*

### 4.2.4 Piling order

Concerning the main area of investigation, around section 100/130, the concrete piles of pile group -13 are of most interest due to those being expected to give the largest displacements. However, the influence of the steel pipe piles in pile group -13 and pile cap foundations -6615 and -6616 has also been considered. A summary of the pile installation order around section 100/130 can be seen in Table 4.5 and a map overview can be seen in Figure 4.9.

*Table 4.5 Summary of installation order of the piled areas located around section 100/130, sorted after date of installation.*

| **Area of installation:** | **Type of piling:** | **Starting date:** | **Finishing date:** | **Piling order #:** |
|---|---|---|---|---|
| West part of pile group -13 | Bored steel pipe piling | 20th of November 2015 | 20th of December 2015 | 1 |
| Pile cap foundation -6615 | Hammered concrete piles | 15th of December 2015 | 22nd of December 2015 | 2 |
| Pile group -13 | Hammered concrete piling | 7th of January 2016 | 3rd of February 2016 | 3 |
| Pile cap foundation -6616 | Hammered concrete piling | 3rd of February 2016 | 5th of February 2016 | 4 |
| Pile cap foundation -6615 | Bored steel pipe piling | 8th of February 2016 | 18th of February 2016 | 5 |
| East part of pile group -13 | Bored steel pipe piling | 24th of February 2016 | 25th of February 2016 | 6 |
| Pile cap foundation -6616 | Bored steel pipe piling | 26th of February 2016 | 29th of February 2016 | 7 |


*Figure 4.9 Overview of the different ground improvements, numbered in the order of installation.*

### 4.2.5 Inclinometer

The inclinometer is used to monitor the lateral displacement (Transportation Research Board, 2008) in the horizontal direction due to soil movement. This is used to alert the engineers if the critical displacement limits are exceeded. With the help of this information necessary measures can be taken, commonly to stop the pile driving, for a brief or longer period. The inclinometer is installed in a PVC casing and anchored at the bedrock, by drilling, or where firm strata is occurring.

The location of the inclinometer is seen in Figure 4.10 and is located approximately 20 metres from the southern edge and 30 metres from the centre line of pile group -13. The inclinometer registers the horizontal displacement perpendicular to Säveån, with a positive displacement defined as a movement towards the river.

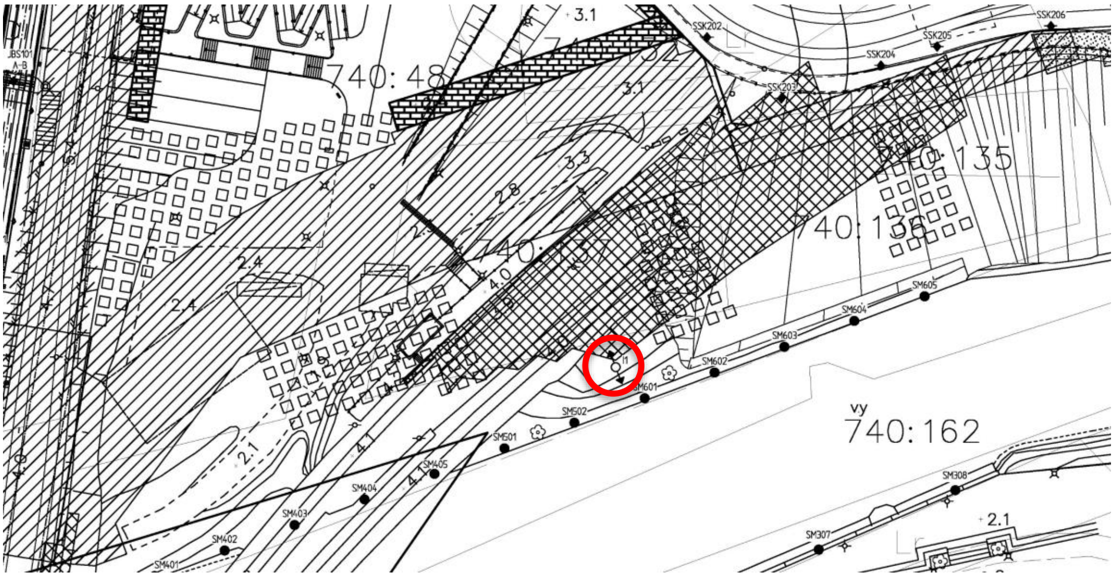



*Figure 4.10 The location of the inclinometer west of pile cap foundation -6616, distinguished by a red circle. The horizontal displacement is measured perpendicular to Säveån, positive movement in south-east direction.*

**Measurements**

The inclinometer readings from start and end of all installations of the piling areas around section 100/130 are presented in Figure 4.11. It can be seen that the inclinometer has moved towards the north before the pile installations started, i.e. the measurements show negative values. This could be due to the large archeologic excavation north of pile group -13 which would lower the load in the area and allow for soil movement. When the pile installation started, it can be seen that there are small movements from the steel pipe piling of the west part of pile group -13, #1, and the concrete pile installation in pile cap foundation -6615, #2. However, the largest movements can, as expected, be seen during the concrete pile driving of pile group -13, #3. There is also significant movement during the concrete pile installation of pile cap foundation -6616, #4, which probably have a large effect due to the proximity to the inclinometer. The last measurements are from the installation of bored steel pipe piles in pile cap foundation -6615, #5, east part of pile group -13, #6, and pile cap foundation -6616, #7. These do however only indicate small movements and are hard to distinguish from each other.

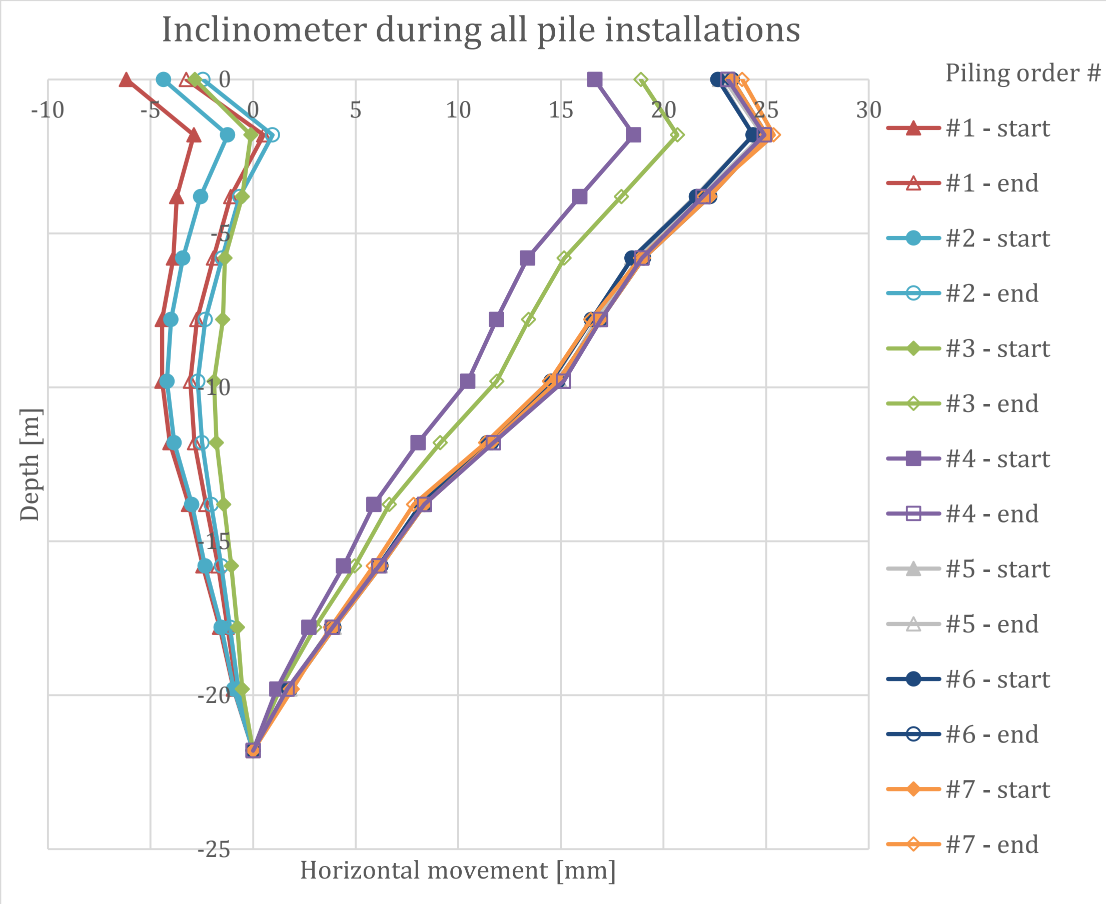



*Figure 4.11 Inclinometer reading at start and end of all pile installations in the different piling areas.*

### Measurements during concrete pile installation

The resulting movements of the inclinometer from the period when the installation of the concrete piles in pile group -13 was performed is seen in Figure 4.12. The graph shows the daily mean average. The data has been corrected so that the measurement readings on the day before the piling started, in the west part of pile group -13, is set to zero. It can be seen that the total movement for the installation of pile group-13 is around 25 millimetres at the end of installation. The reason it is slightly increasing one day after is because the concrete pile installation continues at pile cap foundation -6616. The original uncorrected graph can be seen in Appendix 5.

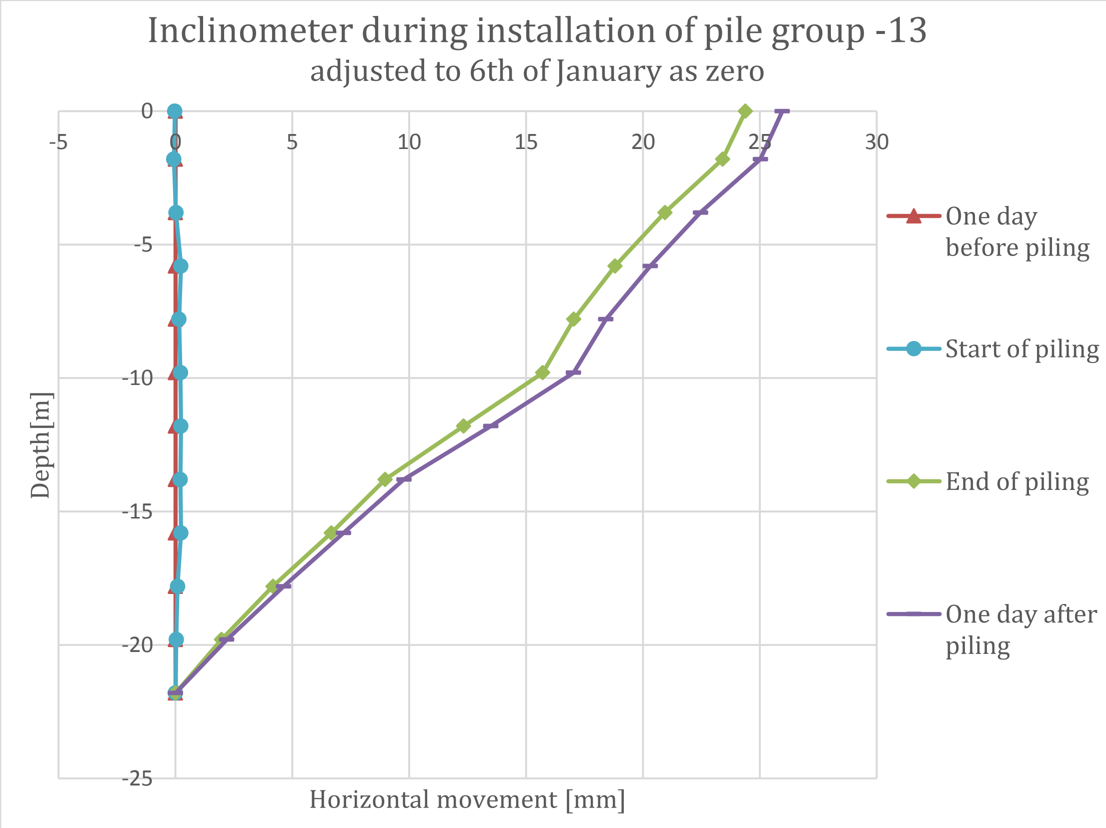



*Figure 4.12 Adjusted horizontal movements registered by the inclinometer during the installation of concrete piles in pile group -13.*

**Measurements during bored steel pipe pile installation**

The steel pipe piles were installed in the west part of pile group -13, pile cap foundation -6615, east part of pile group -13 and pile cap foundation -6616. The installations occurred at different dates and the resulting horizontal inclinometer movements can be seen in Figure 4.13. The piles installed in the west part of pile group -13 have not been taken into consideration since those were installed before the concrete pile installation.

The data in the graph is corrected so that the movement, after the installation start of pile cap foundation -6615 which has been adjusted to zero, can be seen. The original uncorrected graph can be seen in Appendix 5 It is important to notice that the scale of the x-axis is changed from the previous graph, Figure 4.12, and that the movements from the steel pipe installation is considerably smaller than those from the concrete piles. Since the maximum movements only are around two millimetres, the effect from surrounding activities and measurement errors can have a substantial impact on the results.

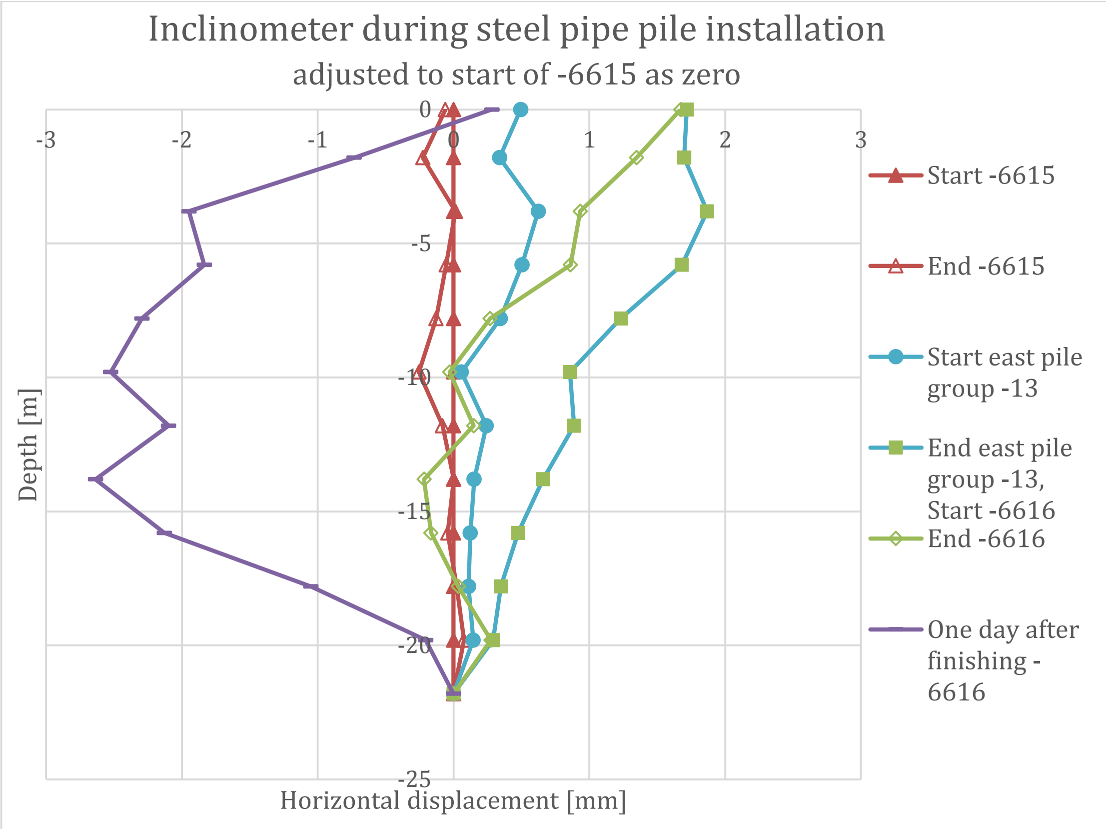



*Figure 4.13 Adjusted horizontal inclinometer movements during the installation of steel pipe piles for -6615, east part of pile group -13 and -6616.*

### 4.2.6 Ground movement gauges

The ground movement gauges are installed on top of the existing retaining wall of the quay side, south of the pile cap foundations -6615 to -6617, see Figure 4.14. Some of the piling had commenced when the movement gauges were measured for the first time. The measurement of some gauges started on the 17$^{th}$ of January while some have only been measured since the 16$^{th}$ of February. The measurement of the gauges' movements is divided into different directions, i.e. north-south, east-west and vertical.

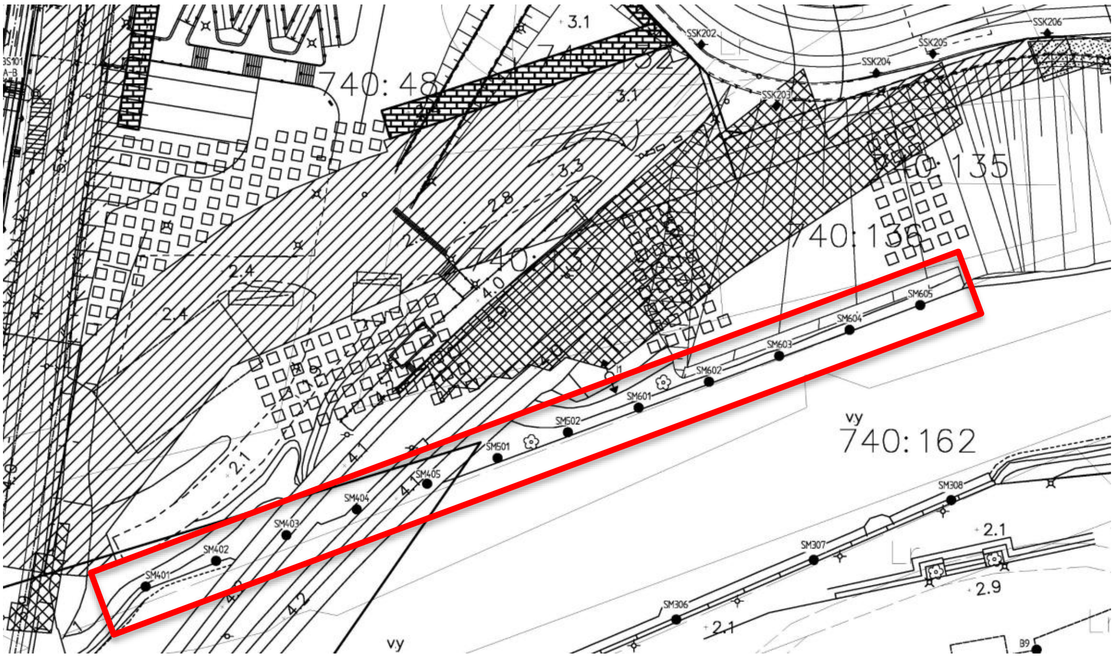



*Figure 4.14 The location of the twelve ground movement gauges, visualised as black dots inside the red rectangle.*

**Measurements**

The movements from the ground settlement gauges are somewhat hard to interpret due to several directions of measurement, different starting dates and the numerous, spread out, locations of the gauges. However, the gauges SM502 and SM601 have been deemed more important, since those are located closest in line to section 100/130, and are presented below. Mainly, it is the time period during the installation of concrete piles that show the largest movements which is why those graphs are presented. The steel pipe pile installation shows a much more diffuse movement pattern between the gauges and it is hard to draw any certain conclusions from the data. All graphs containing the full uncorrected measurements in all directions are found in Appendix 5.

**Measurements during concrete pile installation**

The horizontal movement in north-south direction during part of the concrete piling can be seen in Figure 4.15. The piling of pile group -13 had already started ten days before the measurements commenced. However, the two gauges also have a significant movement towards the river during the remaining pile installation of pile group -13 and pile cap foundation -6616. To be able to compare the values from the gauges, a recalculation was performed to correct for the missing movement of the piles installed before measurement was initiated. The re-calculated values can be seen in Figure 4.15 and the method of recalculation is found in Appendix 5.

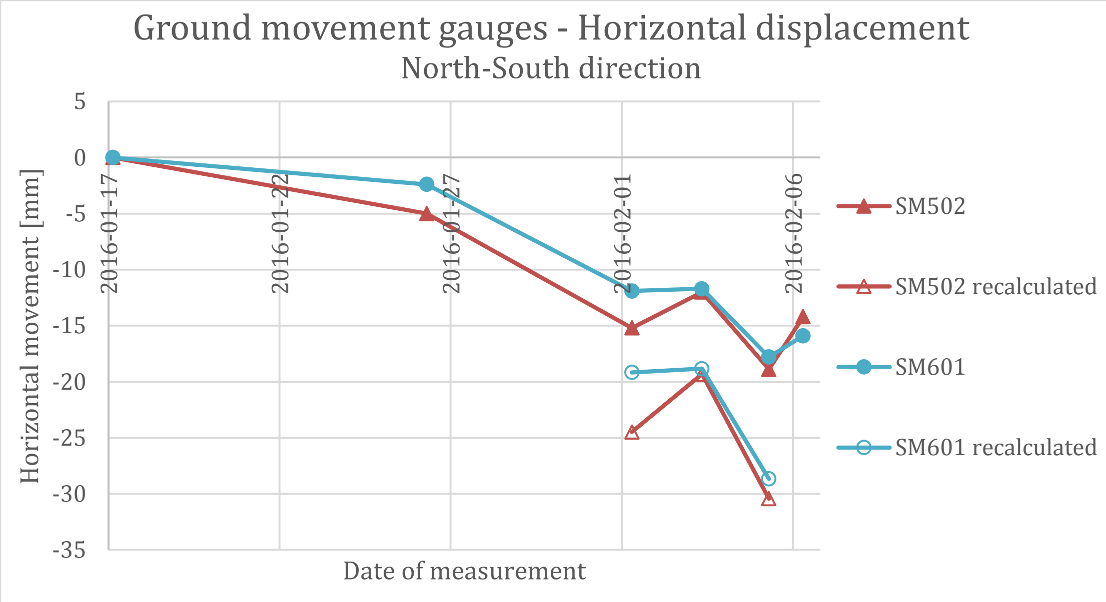



*Figure 4.15 Movement in north-south direction for the measured ground movement gauges during the installation of concrete piles.*

The vertical movements, during the same time period as shown in Figure 4.15, can be seen in Figure 4.16. In this graph, it can be seen that the gauges, SM502 and SM601, have a maximum vertical movement of around 5.5 millimetres. The same technique used previously, for recalculating the horizontal displacements, is also utilised for the vertical displacements, in order adjust for the missing movements of the pre-installed piles.

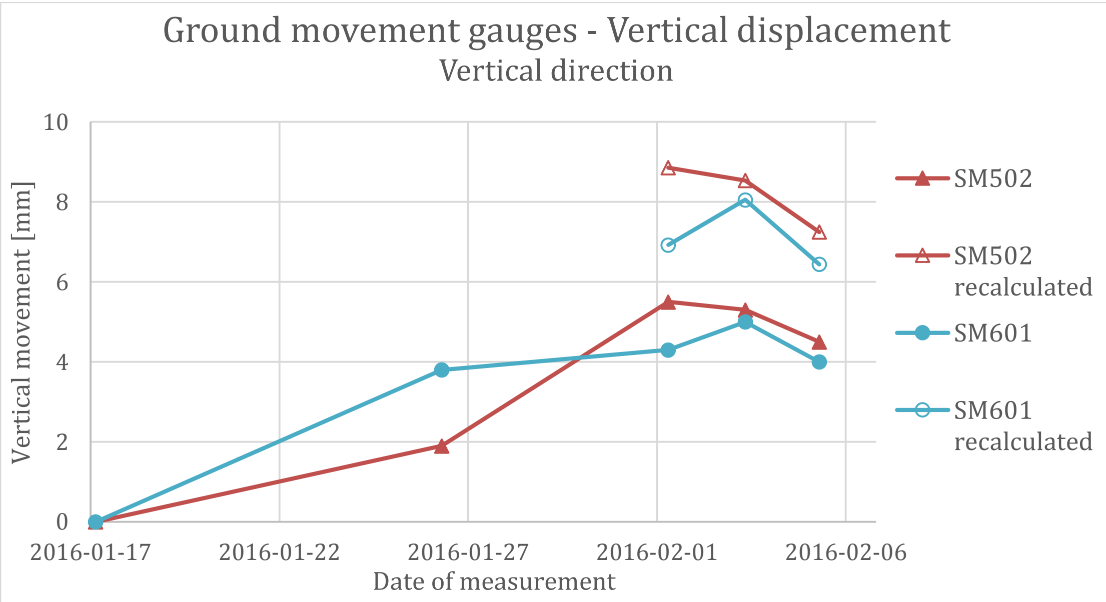



*Figure 4.16 Movement in vertical direction for the measured ground movement gauges during the installation of concrete piles.*

## 4.3 Modelling of section 100/130

In this chapter the modelling of pile group -13 at section 100/130 in PLAXIS will be explained. The chapter involves; which input parameters that have been used, how the geometry is modelled, how the mesh is distributed, the calculation phases and what parameters that is used in the sensitivity analysis.

### 4.3.1 Input parameters

The input parameters to be used in a model depends upon the collected data. The input parameters could be chosen through field measurements, empirical relations and values, soil tests in laboratory or tabulated values. However, all data needs to be interpreted, using an engineering judgment, to decide what values that are reasonable to be used.

When considering a problem of piling in clay, an undrained behaviour is usually assumed due to the low permeability of clay and short time spans. However, it is still common to use the effective, i.e. drained, parameters as input in PLAXIS.

**Young's modulus, E**

The Young's modulus, sometimes also referred to as the elastic modulus, used in soil mechanics is a measure of soil stiffness. The parameter describes the relationship between stress and strain in the soil.

In this case study, the Young's modulus has been derived indirect from the CRS-tests by matching the curve and calculating the oedometer modulus, $E_{oed}$. This value can then be inserted into PLAXIS giving the effective stiffness, E', through the relation in equation 4.1 (PLAXIS, 2016).

$$E_{oed} = \frac{(1-\nu)E}{(1-2\nu)(1+\nu)} \tag{4.1}$$

The effective stiffness, E', is constant around 1,500 kPa in the silty clay layer down to level -10. It then increases in the lower clay layer, from 1,500 to 2,668 kPa at level -23.5. The fill layer has been assumed to have a constant effective stiffness of 10,000 kPa.

**Poisson's ratio, ν**

The Poisson's ratio is a measure of the Poisson effect, the effect when a material tends to expand in the two directions that is not being compressed. The same value of Poisson's ratio is obtained when observing if a material is stretched instead of compressed. In this case the material tends to contract in the transverse direction of the direction of stretching, e.g. a rubber band becomes thinner when stretching. The ratio could be seen as the fraction of expansion divided by the fraction of compression.

The Poisson's ratio, $v$, is depending on the specific case that is to be modelled. When only considering a short duration in clay, the behaviour is undrained and therefore close to 0.5, in other words acting as an incompressible material (Craig and Knappett, 2012). However, the effective Poisson's ratio $v'$, that is used in the modelling has been derived from empirical values and matching the curves of the CRS-tests. Hence achieving a value of $v'$ equal to 0.25 for the entire soil profile beneath the fill layer. The fill itself is assigned a value of 0.35 (Gercek, 2006).

**Friction angle, ϕ**

The definition of the friction angle is derived from the Mohr-Coulomb failure criterion. The friction angle, ϕ, is a measure of the soils shear strength and is different for different soils. If the inclination for a slope, when excavating, exceeds the friction angle the state of the soil particles changes, from rest to rolling, and a landslide will occur. (Swedish Geotechnical Institution, 2016).

The effective friction angle, ϕ', were extracted from Larsson et al (2007). The authors state that slightly over consolidated to normally consolidated clays in western Sweden have an effective friction angle of 30 degrees. Further on, when calculating bearing capacity and strength in the fill the author refers to the guidelines and a friction angle of 32 degrees.

**Cohesion, c**

In cohesive soils, such as clay, the shear strength is combined by the frictional forces between soil particles and the adhesion between the finer particles in the soil (Swedish Geotechnical Institution, 2016)

The effective cohesion c' were extracted from Larsson et al (2007). The authors state that slightly over consolidated to normally consolidated clays in western Sweden have an effective cohesion, $c'$, of either ten per cent of the undrained shear strength, $c_u$, or three per cent of the in-situ effective stress, $\sigma'_0$. Giving an effective cohesion of around three for both of the clay layers. The fill however is assigned zero cohesion since it is mostly containing sand and gravel, thus being cohesionless (PLAXIS, 2016).

**Earth pressure coefficient, $K_0$**

The earth pressure coefficient, $K_0$, is the relationship between the horizontal and vertical stresses in the soil. Which for normally consolidated Swedish clays can be approximated by the empirical relationship seen in equation 4.2 (Larsson et al., 2007). If the clay has a high content of silt, the $K_{0NC}$ can be assumed to be approximated by the value of 0.5.

$$K_{0NC} \approx 0.31 + 0.71(w_L - 0.2) \tag{4.2}$$

Which means that a clay with a liquid limit of 70 per cent, as the clay at Gamlestadstorget, would equal a $K_{0NC}$ of around 0.665.

If a clay is over consolidated, i.e. an over consolidation ratio (OCR = $\sigma'_c$ / $\sigma'_v$) above one, the relationship in equation 4.3 should be used instead (Swedish Transport Administration, 2014). This is since the vertical stresses becomes lower if the soil has previously experienced a loading higher than present conditions. This changes the ratio between horizontal and vertical stresses and thus also changes the earth pressure coefficient $K_0$.

$$K_0 \approx K_{0NC} * OCR^{0.55} \tag{4.3}$$

For the clay at Gamlestadstorget, which has a varying OCR of up to about seven, this means that the $K_0$ is also varying from the value of $K_{0NC}$ up to 1.94.

**Dilatancy angle, ψ**

When exposed to shearing, soils either expands or contracts. This behaviour is captured with the value of the dilatancy angle. The angle controls how much the volume changes during plastic shearing, meaning that if the dilatancy angle is zero degrees then the deformation of volume is unchanged during shear.

The dilatancy angle for clays can usually be assumed to be zero degrees in undrained behaviour, if the clay is not heavily over consolidated i.e. clays with an OCR greater than four (PLAXIS, 2016). The dilatancy for the fill material, consisting of some parts sand, is assumed to 15 degrees according to typical values presented by Barlett (2016)

**Summary of input parameters**

In Table 4.6, a summary of the input parameters used for the different layers, along with the input parameters for the quaysides, in the model of section 100/130 can be seen. The soil was interpreted and simplified as consisting of three main layers with different soil properties. The parameters used for the quaysides are a concrete pile material extracted from the PLAXIS tutorial manual (PLAXIS, 2016).

To verify the soil properties of the model, CRS-tests in PLAXIS 2D have been performed and compared to the data from the real CRS-tests. These evaluations can be found in Appendix 6 and was implemented to find a corresponding Young's modulus and Poisson's ratio. It was also done to verify that the soil model realistically captures the real behaviour of the soil.

*Table 4.6 Input parameters for the different soil layers at section 100/130.*

| **Layer** | **Fill** | **Silty Clay** | **Clay** | **Quaysides** |
|---|---|---|---|---|
| Material type | Drained | Undrained | Undrained | Non-porous |
| $\gamma_{unsat}$ [kN/m$^3$] | 10 | 6.5 | 6.5 | 24 |
| $\gamma_{sat}$ [kN/m$^3$] | 18 | 16.5 | 16.5 | 24 |
| E' [kN/m$^2$] | 10,000 | 1,500 | 1,500 | 30E6 |
| $E'_{inc}$ [kN/m$^2$/m] | 0 | 0 | 83.5 | 0 |
| $y_{ref}$ [m] | - | - | -10 | - |
| $\nu'$ [-] | 0.35 | 0.25 | 0.25 | 0.10 |
| c' [kPa] | 0 | 3 | 3 | - |
| $\phi'$ [°] | 32 | 30 | 30 | - |
| $\psi$ [°] | 15 | 0 | 0 | - |
| $K_{0,x}$ [-] | 1.88 | 1.10 | 0.70 | 1 |

### 4.3.2 Calculation of volumetric expansion

The calculation of the volumetric expansion was based on the ratio between the volume of the installed piles and the total soil volume of pile group -13, seen in detail in Appendix 7. The total volume of piles was calculated and divided with the total soil volume, thus achieving a volume increase of 0.73 per cent. This calculated volume increase was then used as the volumetric expansion in the PLAXIS modelling. Different degrees of this volume increase was used to find a best agreement to measurements, i.e. 0.73 per cent volume expansion representing a 100 per cent volume increase and 0.365 per cent volume expansion representing a 50 per cent volume increase.

The same method was used for modelling the pre-auger efficiency of the top ten metres, see Appendix 7. Thus a 100 per cent pre auger efficiency is represented as a 0 per cent volume expansion while a 50 per cent efficiency is represented as half of the modelled volume expansion.

### 4.3.3 Geometry

The geometry of the model could be seen in Figure 4.17. The boundary to the left is representing the centre line of pile group -13 with the soil clusters next to it being pile group -13. The vertical line near the river is representing the location of the inclinometer. The model is extending in the same direction as the inclinometer measurements i.e. north-south direction. The quaysides are coloured in pink, where the geometries have been extracted from the drawings found in Appendix 4. All the boundaries around the geometry is closed in one direction, except for the bottom boundary outside of the piling area. The left and right boundary are closed in x-direction, while the bottom boundary, outside the piling area, is closed in both x- and y-direction. The global ground water level is set to level +0.5.

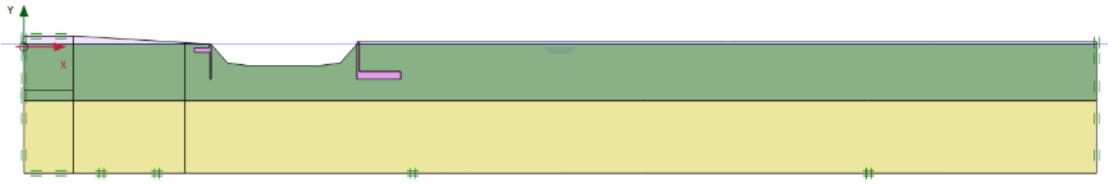


*Figure 4.17* *The geometry of the PLAXIS model, with the straight vertical line closest to the river is representing the inclinometer and the pink structures are representing the quaysides.*

Figure 4.18 is an enhancement of the left boundary showing the soil clusters that represents the soil volume of the piling area. These are the clusters that are subject to a volumetric expansion to simulate the increase of volume that the pile installation causes. The horizontal line in the silty clay layer is the bottom level for pre-augering. The top and bottom boundaries of the piling area a closed in x-direction.

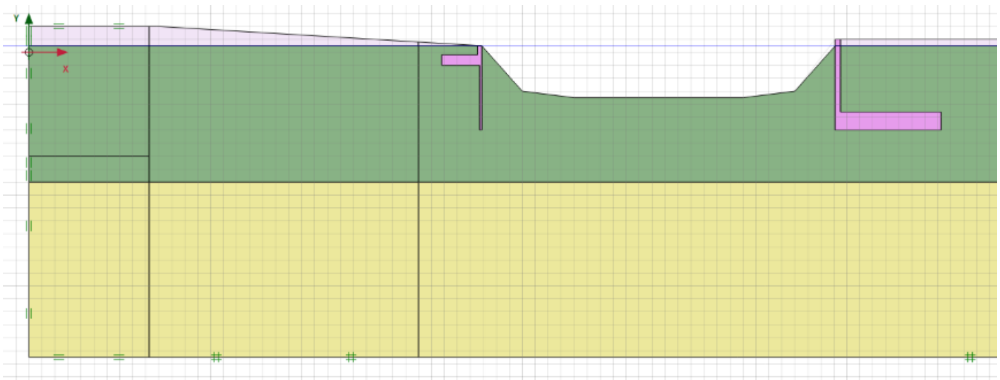


*Figure 4.18* *Enhancement of the PLAXIS model showing the soil clusters at the left boundary representing the piling area.*

### 4.3.4 Mesh

Figure 4.19 shows the mesh used in the model. It can be seen that the mesh is refined close to the piling area and the quaysides. The mesh is also refined manually on the inclinometer line, to get a more evenly distributed mesh from the centreline to the river. The mesh is made by medium element distribution, 0.06, generating in 1626 triangular 15-noded elements and a total of 13,685 nodes.

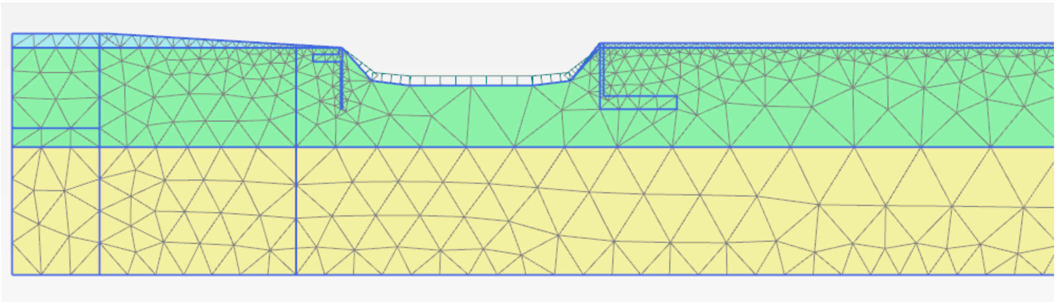


*Figure 4.19* *Enhancement of the mesh of the PLAXIS model containing 1626 elements and 13685 nodes.*

### 4.3.5 Calculation phases

**Phase 1, Gravity loading**

The first phase is performed as gravity loading, which is used to generate the initial stresses in the model. Gravity loading is a type of plastic calculation which is used instead of the K0 procedure when a non-horizontal ground level is to be modelled. The effective horizontal stresses generated are dependent on the value of Poisson's ratio. It is however not possible to assign a $K_0$ value higher than one when the gravity loading phase is used. Thus, in order to achieve the representative horizontal effective stresses, the Poisson's ratio is temporarily changed in this phase as well as in phase 2. The ratio is calculated through Equation 4.4 (PLAXIS, 2016), giving a ratio of 0.42 in the bottom clay layer and higher than 0.5 for the fill and silty clay layer. Since it is not possible to assign a value above 0.5 either, the layers are assigned 0.495 instead.

$$K_0 = \frac{\nu}{1-\nu} \tag{4.4}$$

**Phase 2, Plastic nil-slip**

This calculation phase is implemented to take care of the out-of-balance forces and re-establish equilibrium in the model. When applying gravity loading, large loadings are activated causing these problems. The plastic nil-slip is a plastic calculation where no further loads or changes are applied.

**Phase 3, Change of soil material**

In this calculation phase, only the soil materials used in the model are changed. The soil assigned a higher Poisson's ratio, used in phases 1 and 2, is replaced with the soil material assigned the actual input parameters.

**Phase 4, Volumetric expansion**

In the last phase, all the boundaries around the model are activated. The soil clusters representing the piling area is then subjected to the volumetric expansion used to simulate the pile installation. The pre-augering is accounted for by varying the volumetric expansion in the upper soil clusters. In this phase, the options reset displacements to zero, updated mesh, ignore suction and reset small strains are activated. A gradual error reduction is also activated.

### 4.3.6 Sensitivity analysis

A sensitivity analysis is performed by varying certain input parameters for the two clay layers in the PLAXIS model. The variations of the parameters used can be seen in Table 4.7. The stiffness parameters are varied because they have the largest impact of the resulting displacements and the mesh are varied to ensure that it does not affect the result. The fill layer was assumed to only have a small impact on the displacement and are thus not changed.

*Table 4.7 Variation of input parameters in the PLAXIS model.*

| **Parameter** | **Min** | **Assumed** | **Max** |
|---|---|---|---|
| Young's modulus, E' | 500 | 1500 | 2500 |
| Poisson's ratio, *v'* | 0.10 | 0.25 | 0.35 |
| Mesh | Very fine | Medium | Coarse |

# 5 Results

The results from the PLAXIS modelling, together with a sensitivity analysis, and the comparison to the other calculations methods are presented in this chapter.

## 5.1 PLAXIS modelling

The PLAXIS model is a representation of the cross section at section 100/130 at the Gamlestadstorget construction project. The horizontal displacements are evaluated from at 30 metres from the centreline of pile group -13, equal to the location of the inclinometer.

### 5.1.1 Volume expansion variation

In Figure 5.1 is the horizontal displacement, with alternation in the volume expansion effect of the installed pile volume, compared with the measured data from the inclinometer. The comparison is based on and utilises a 50 per cent pre-auger efficiency, as established in the reference project. It can be seen that the displacements, as expected, become lower with decreased volume expansion. The difference between using the LE or MC soil model are small for each of the volume expansion effects modelled. It can be concluded that a 50 per cent of the installed pile volume as volume expansion complies well with the measurement from the inclinometer.

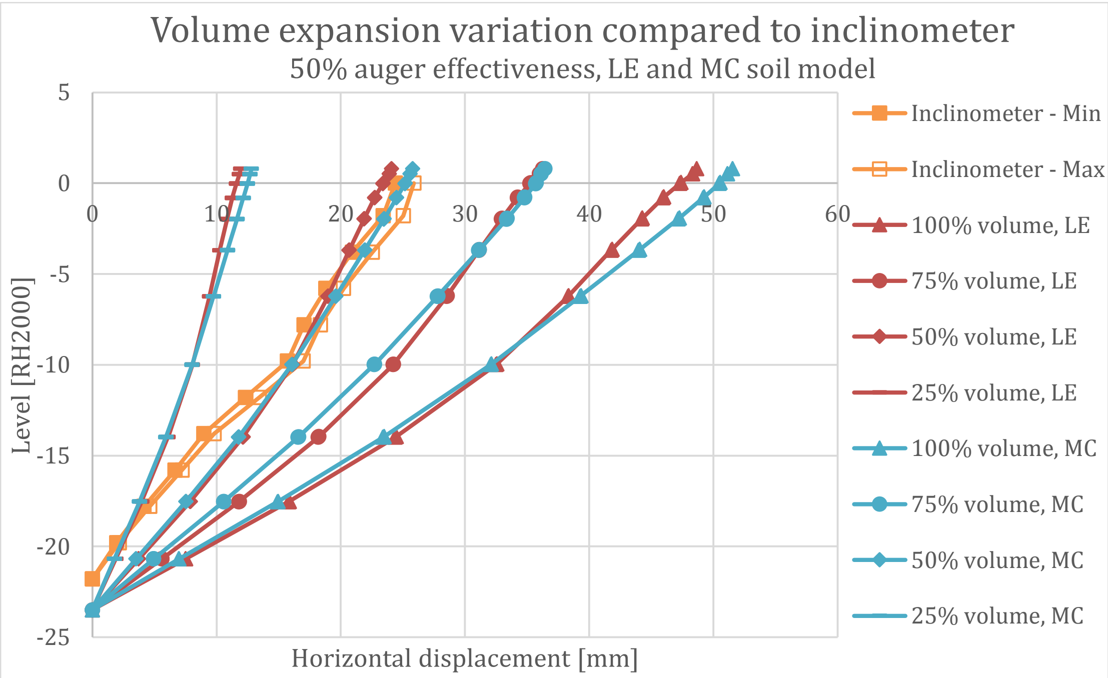



*Figure 5.1* *Horizontal displacements for varying volume expansion when using 50 per cent pre-auger efficiency, both with LE and MC soil model, compared to the measured data from the inclinometer.*

The reason 50 per cent of the calculated volume expansion have the best fit in the model could be due to the shape of the piling area. First of all, when tackling a 3D case in a 2D model there will consequently be some loss of 3D effects. The 3D effects disappear since the model assumes the pile group to act as a wall, infinitely long, preventing the soil to move in between the piles. Pile group -13 can, with its c/c-distance and pile diameter having a ratio of around ten, be assumed to have the behaviour of a pile row. In that case, some of the displacements would occur inside of the piling area, thus lowering the amount of displacements outside of it.

### 5.1.2 Pre-augering efficiency

The effect of pre-augering is compared to the measured data of the inclinometer in Figure 5.2. The case with 50 per cent of the calculated volume expansion was chosen since it agreed well to the measured data. The case using 75 per cent of the calculated volume expansion was also added to see if it would achieve a good fit with higher pre-auger efficiency. When altering the pre-auger, it can be seen that the horizontal displacements varies a lot. In the case using 50 per cent of the calculated volume expansion, the difference between 0 to 100 per cent pre-auger is about 20 millimetres.

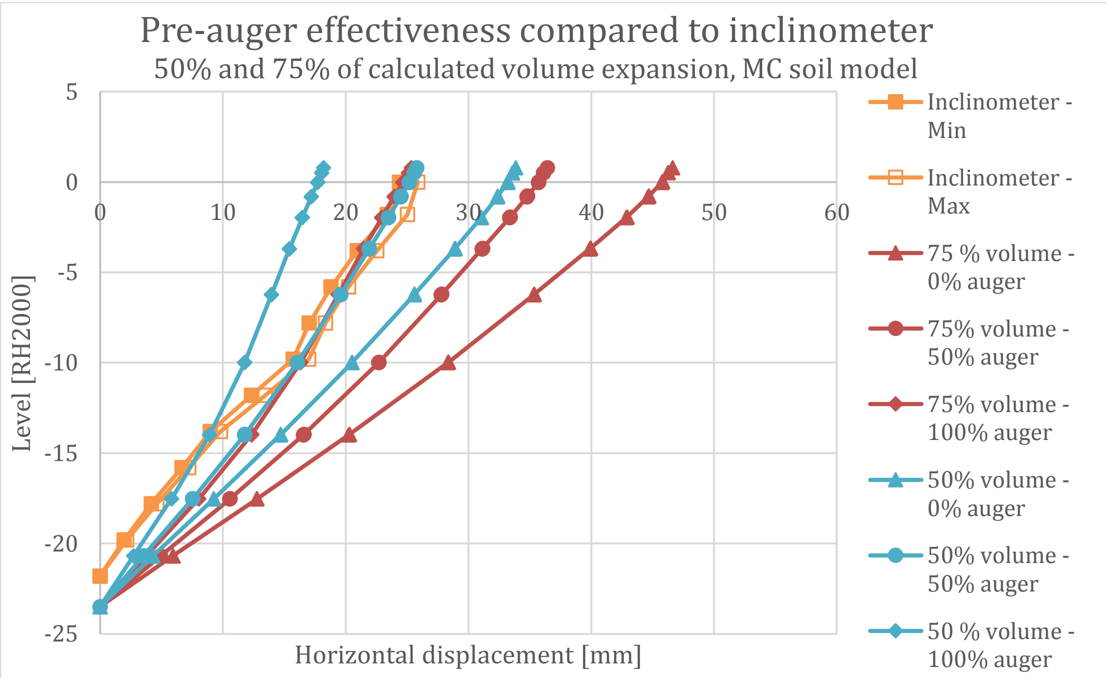



*Figure 5.2 Horizontal displacements for 50 and 75 per cent of calculated volume expansion each with 0, 50 and 100 per cent pre-augering and Mohr-Coulomb soil model, compared to measured data from the inclinometer.*

The result for the case using 50 per cent of the calculated volume expansion complies with the results from the reference project, were it also was concluded that 50 per cent pre-auger efficiency gave the best fit. However, in the modelled case, using 75 per cent of the calculated volume expansion with 100 per cent efficient pre-auger gave a good agreement as well, which contradicts the result from the reference project.

### 5.1.3 Displacement at different distances

In Figure 5.3, an overview of the lateral displacements is distinguished by a shading plot. The scale of the displacements could be seen on the right side of the figure, where red is representing the largest amount of displacements and dark blue the smallest. The plot shows how the larger displacements are focused close to the pile clusters and is decreasing with distance from the centreline of pile group -13. The model is based on the previous modelling, where it was found that 50 per cent of calculated installed pile volume with 50 per cent pre-auger complies well with the measured data.

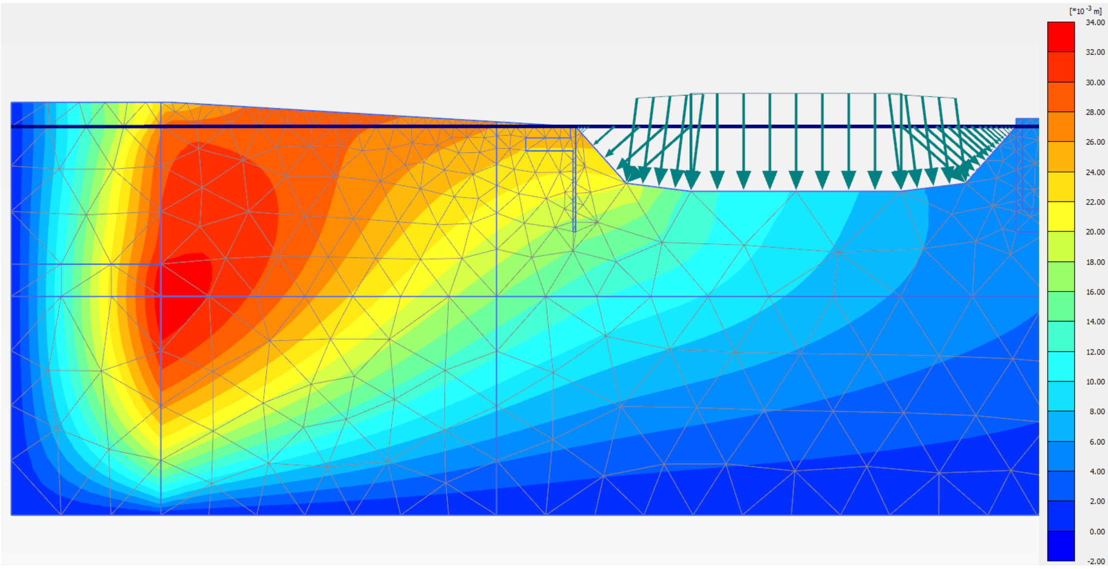



*Figure 5.3 Overview of the horizontal displacements distinguished by a shading plot, where the scale is red being the largest amount of displacements and dark blue the smallest.*

In Figure 5.4, the displacements at different distances from the centre line of pile group -13 is plotted. All data is from the same model and it gives a clear indication about how the displacements varies with distance. The displacements are also compared with the measured data from the inclinometer which is located at a distance of 30 metres from the centre line. It can be seen that the largest displacements are found close to the pile clusters and is decreasing at cross sections farer away.

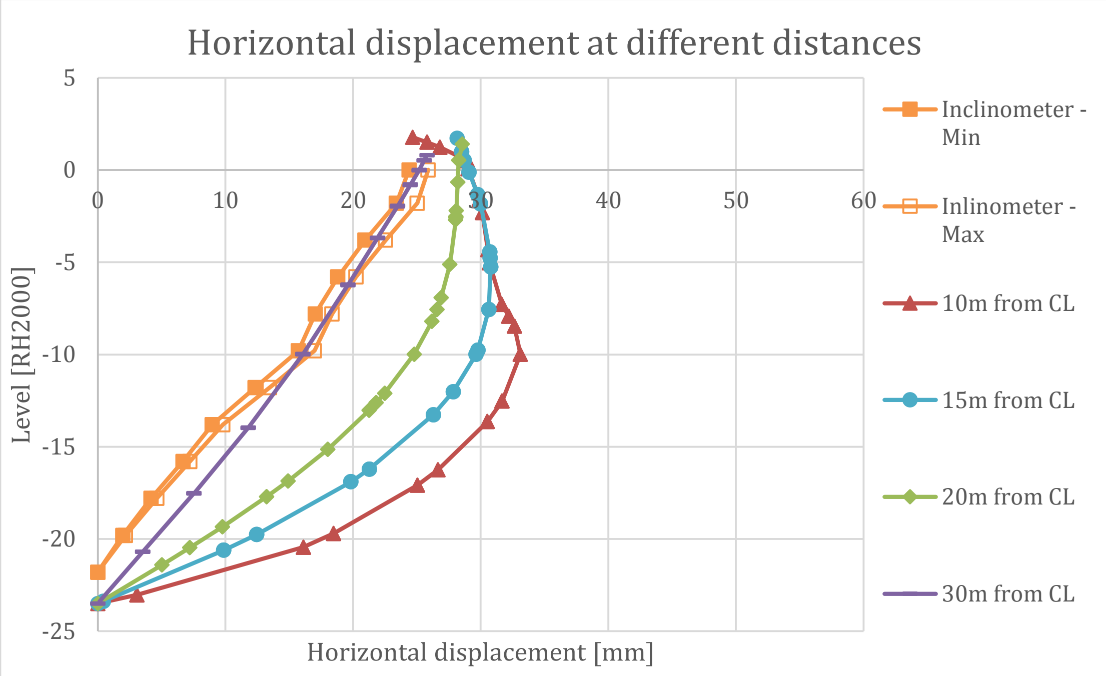



*Figure 5.4 Horizontal displacements at different distances from the centreline of pile group -13.*

The displacements at ten metre from centre line are at the edge of pile group -13, where the pile clusters are subjected to volumetric expansion. Thus, it displays the highest amount of displacement. For the same distance but above level -10, the displacement curve is not as smooth as for the other distances. This is because of the pre-auger effect of the upper cluster, where the expansion is 50 per cent less than for the lower clusters. For the other distances, the displacements of the first ten metres is likely affected by the movement of the soil below, as seen in Chapter 2.1.1, giving a smoother curve. For the different distances, the displacement at ground surface only varies marginally.

### 5.1.4 Sensitivity analysis

In this chapter the result of the sensitivity analysis is presented. The model uses the volume expansion equal to 50 per cent of the installed pile volume and 50 per cent pre-auger efficiency.

**Young's modulus**

The main difference in horizontal displacements was achieved when decreasing Young's modulus to 500 kPa, as seen in Figure 5.5. This causes the displacements to become higher above level +6 but lower below +6 when compared to the original model. Increasing Young's modulus to 2500 kPa gives a similar curve as using the original value of 1500 kPa.

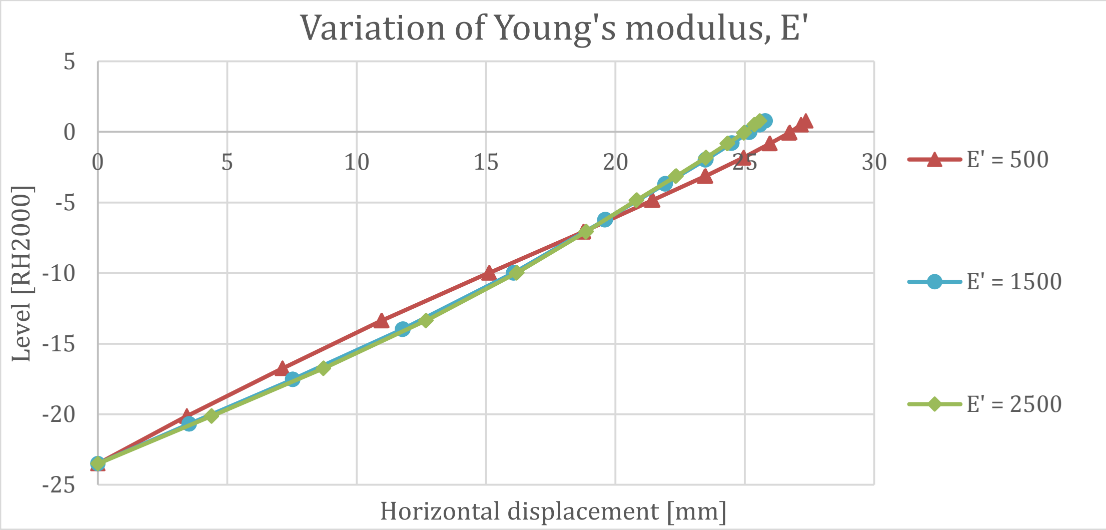



*Figure 5.5 Horizontal displacements with alternation of Young's modulus.*

**Poisson's ratio**

Figure 5.6 shows the variation of the horizontal displacements with altered Poisson's ratio. No significant differences could be seen, however, increased Poisson's ratio gives a slightly higher displacement closer to the surface.

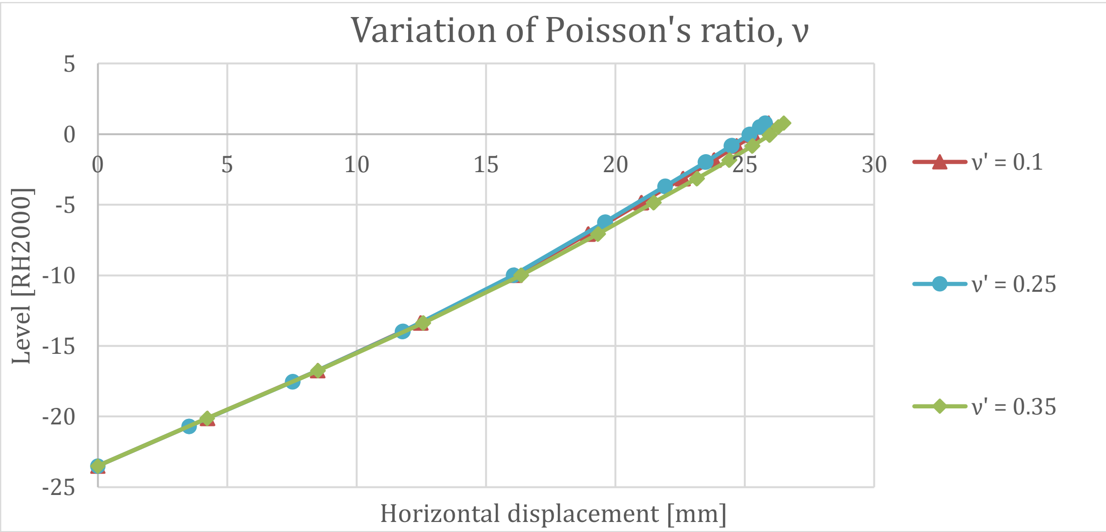



*Figure 5.6 Horizontal displacements with altered Poisson's ratio.*

**Mesh**

Neither the mesh distribution has a significant impact on the horizontal displacements. The model could have been calculated using a coarse distribution, instead of medium, to save time. However, the model was not complex enough to give a large significant computational time difference.

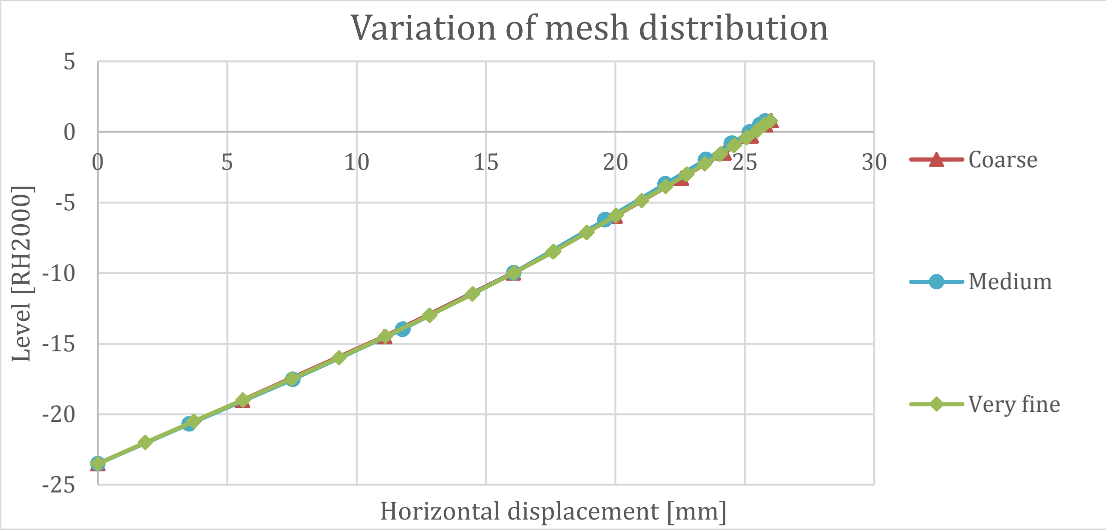



*Figure 5.7 Horizontal displacements with altered mesh distributions.*

## 5.2 Comparison of different methods

The resulting movements from PLAXIS are compared to measured data and two other methods used to calculate the displacements. Rehnman's method are empirical and have traditionally been used in Sweden. Sagaseta is a newer and more advanced semi-analytical method which is also the method used by Skanska for their prognoses in the case of Gamlestadstorget. For the PLAXIS results, the method of modelling the piles that have been reckoned to be the most accurate in Chapter 5.1 are used in the comparison. The calculations for Rehnman's method and Sagaseta's method are found in Appendix 8 and Appendix 9 respectively.

All the different methods are subject to some degree of difference and insecurity. The PLAXIS model have been analysed in Chapter 5.1, were a large variety of outcomes can be seen depending on how the model is set up. For the ground movements calculated with Sagaseta, the simplified analytical equations are used. These do not take any pre-augering into account, neither does it account for the piles being end-bearing on bed rock or the sloping ground surface. Rehnman's method have many different variables that needs to be assumed causing large differences and insecurity depending on the chosen values.

Measured data used are taken from the inclinometer and the ground settlement gauges closest to section 100/130. The data from the ground settlement gauges have been recalculated to account for the pile installation that took place before the measurements started. This, combined with the pre-existing measurement variation for the dates around the end of the concrete pile installation, is also a cause of insecurity of the data.

### 5.2.1 Horizontal displacement at inclinometer

In Figure 5.8, the resulting horizontal displacements from the different methods are compared to the inclinometer data measured after the pile installation of pile group -13 was finished. The PLAXIS analysis, which have been matched to the inclinometer movements, gives a good approximation, particularly down to level -10. The results of the Sagaseta method used here has been performed by Skanska and is matched to agree with a calculated 25 millimetre horizontal movement at ground surface, for the full calculations see Appendix 9. It has a good agreement down to a level of -8 but greatly overestimates the deformations below this, probably due to the fact that the method is a simplification, have only been matched to the ground surface movement and does not assume any firm bottom. Rehnman's method provide an approximation that is too low compared to the measurements. This is largely because the method uses the average pile length, 31 metres, as depth.

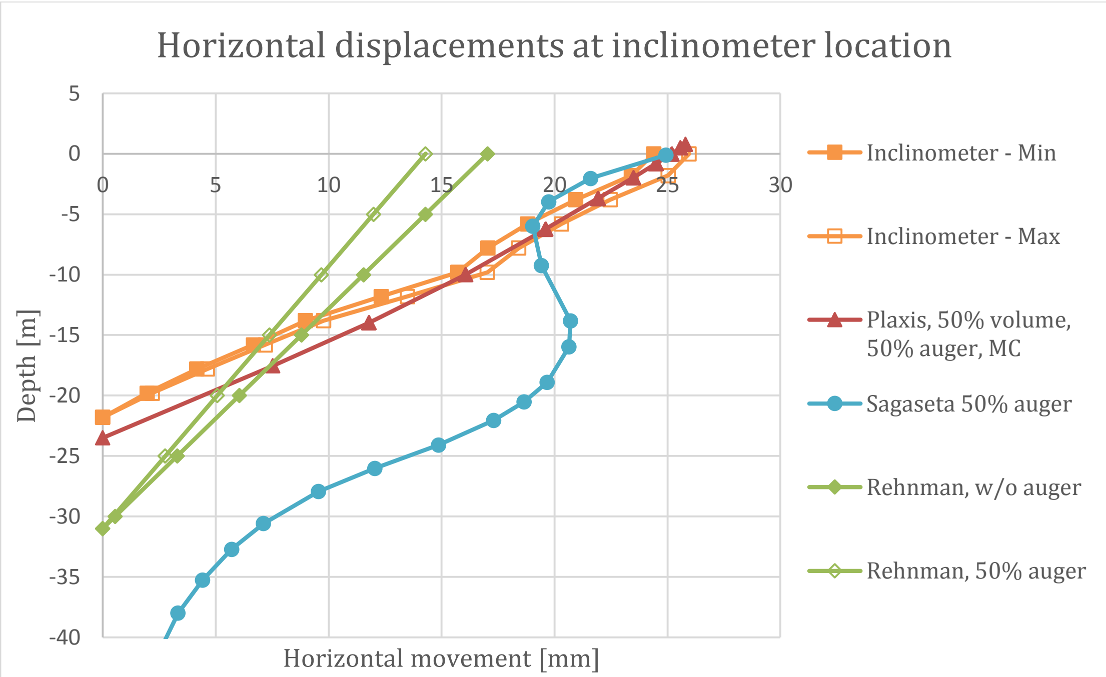



*Figure 5.8 Horizontal displacement at inclinometer location, analysed with different approaches.*

### 5.2.2 Horizontal displacement at ground surface

Both the PLAXIS result and Sagaseta's method give a good estimation of the actual movements at the point of measurement, see Figure 5.9. Rehnman's method however, achieves a results that is considerably lower than the actual measurements, even when assuming zero per cent efficiency of the pre-augering. The largest differences in the results between the methods are found close to the centre line and at a distance. Neither Rehnman's or Sagaseta's method does take the influence of Säveån river, nor the quayside, into account, both these methods also assumes a flat ground surface.

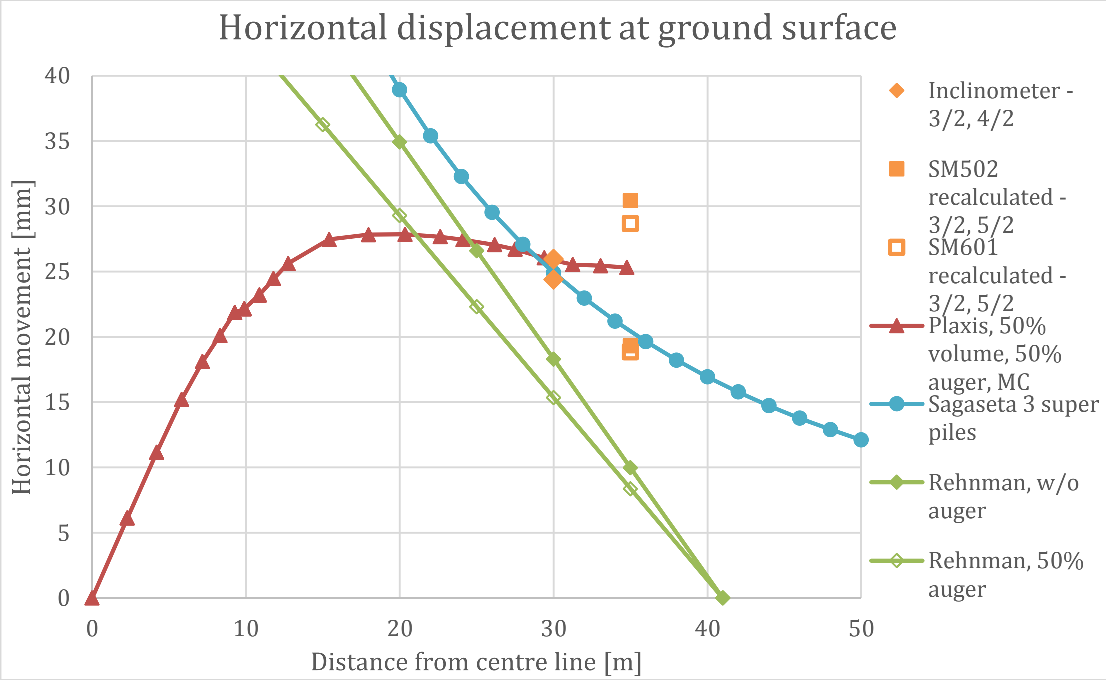



*Figure 5.9* *Horizontal displacement at ground surface, analysed with different methods and compared to measured data from ground settlement gauges at the quayside and the inclinometer.*

### 5.2.3 Vertical displacement at ground surface

For the vertical displacements, all three methods are close to each other and accurate at the point of measurement, see Figure 5.10. The difference between the methods are smaller when comparing heave than for the horizontal displacements. The largest difference can be seen closer to the pile group. The same differences between the methods results as mentioned about the horizontal displacements at ground surface can also be seen to be true for the vertical displacements.

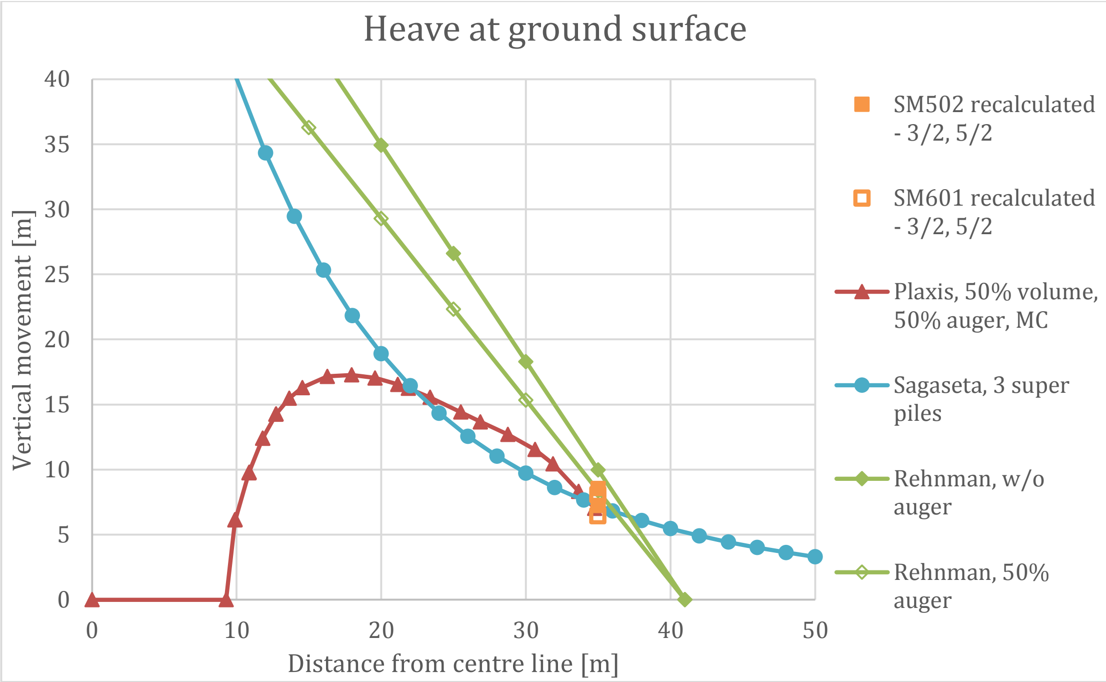



*Figure 5.10 Vertical displacement at ground surface, analysed with different methods and compared to measured data from ground settlement gauges at the quayside.*

## 5.3 Bored steel pipe piles

The measurement data from the inclinometer described in Chapter 4.2.5 and Figure 4.13 showed that the movements, when installing the areas with bored steel pipe piles, were small to almost insignificant. Due to the lack of movements as well as the insecurities surrounding the installation, together with the unclear process of how to model the pile installation, it was judged unfeasible to model the piped pile installation in the case of Gamlestadstorget using PLAXIS.

# 6 Discussion

As seen in the results, it is possible to achieve a good agreement with the measured data when using a plane strain model to model a complex situation as the case of section 100/130. Several insecurities are however limiting the conclusions that can be drawn from the model.

**Pile installation modelling**

The results show that a 0.365 per cent volume expansion equal to 50 per cent of the total installed pile volume, together with a 50 per cent pre-auger efficiency, have the best agreement compared to measurements. The reason for this is probably an effect of several different variables. First of all is the procedure of modelling a 3D problem, as a pile group actually is, using a 2D model. By using a plane strain model, the soil clusters representing the piles and piling area soil volume acts as a wall expanding indefinitely in the z-direction. The loss of 3D effects in the model that this causes can primarily be the reason why a volume expansion equal to 100 per cent installed pile volume overestimates the soil displacements. Pile group -13 have a quite large c/c-distance of three metres, which in reality would allow for soil to move in between the piles. It would also allow for some of the displaced soil to cause heave within the piling area, thus lowering the volume of soil being displaced outside of it. Another reason for overestimation could be that the soil in reality compresses to a certain extent, since the assumption of clay in undrained conditions being incompressible is a simplification. Other effects such as the pile installation causing a disturbed and smeared zone around the pile, could also have some influence but is hard to know the extent of. These effects might also allow for some dissipation of water embedded in the soil, which would lower the pore pressure and thus the displacements.

The process of modelling the pile installation used in the thesis is just one of several different ways of how it could be done. There is also an abundance of different settings that can be changed and tweaked. It is not possible to find one unique and universal correct process; instead different approaches will give somewhat different results. The process chosen in the thesis was used because it had previously been used in articles and theses, showing accurate results compared to measured data. However, it is important to keep in mind that a different modelling technique could have been more accurate or have given a better approximation. The exact settings used and lack of explanation from previously performed models is also an insecurity to take into account.

Both the case study of section 100/130 and the pile modelling reference project displayed a good agreement when assuming a pre-auger efficiency of 50 per cent. In both cases, a standard sized auger was used down to a depth of ten metres. One of the problems when using an auger screw, is that it is not possible to confirm exactly how much soil that is removed. Another is that the screw is circular and not squared as the pre-cast concrete piles used which could have some impact on the effect. Nonetheless, the two cases studied in this report does show that assuming 50 per cent efficiency give a good agreement to measured data while 100 per cent overestimates the pre-auger effect. Additional case studies do however needs to be performed before a general conclusion of the pre-auger efficiency can be established.

**Method comparison**

Compared to the ground surface measurements performed at section 100/130, all methods agree well with the measured vertical displacement. However, when compared to the horizontal displacements, Rehnman's method generally disagrees the most while both PLAXIS and Sagaseta's method demonstrate a good agreement. When it comes to the horizontal movements towards depth at the inclinometer location, Sagaseta's method largely overestimates while Rehnman's method largely underestimates the displacements. It is only PLAXIS that is able to achieve values close to the measured data. This is however not surprising since the PLAXIS model have been matched to the inclinometer movement, while Sagaseta is only matched to the calculated ground surface movement and a simplified calculation method was used.

An overall agreement cannot be generally verified for the PLAXIS model however, due to the lack of additional measurement points to compare with. This is a weakness that have to be taken into account, especially if the PLAXIS modelling is to be used in a different scenario. All methods do have several different insecurities integrated in the models which is important to know about when using them for prognoses. Sagaseta's method could be considered a more general method, but with less accuracy to complexity when compared to PLAXIS. Rehnman's method needs to be seen as a rough estimate of the soil displacements, with less connection to reality than the other two methods.

**Weaknesses and insecurities**

A weakness when it comes to the measured data is the fact that the ground movement gauges was not installed until the $17^{th}$ of January. That is ten days after the concrete pile installation of pile group -13 commenced, which have a large impact on the results since 37 per cent of the pile volume was installed during these ten days. To be able to compare the calculated and modelled values to the measured ones, the measured values were re-calculated. This method further increases the insecurity of the measured data. It should however give a worst-case scenario since the missing data are from piles furthest to the west which should give a lower movement at the measured locations.

The sensitivity analysis performed could be more extensive but nonetheless give a perception of the effect from the most important parameters with regard to the displacements. In this specific case study, none of the parameters have a particularly large effect when compared to the effect from the modelling process and volume expansion chosen. The large insecurities, of how the modelling process should be carried out, results in less importance of the input parameters chosen. In a more controlled experiment with a soil model of greater complexity, a higher level of accuracy would be required when choosing the parameters since they could have a larger effect in such a case.

Another insecurity is the pre-existing piles from the old tramway, located south of pile group -13. These could counteract the soil movement caused by the pile installation and are not accounted for in the Sagaseta and Rehnman methods. Which could be one of the causes to why Sagaseta's method gives an overestimation of the movements towards depth. For Rehnman's method, the movements are already underestimated and the prediction would probably have been even less accurate without the pre-existing piles.

**The modelling of bored steel pipe piles**

Regarding the bored steel pipe piles, there were extensive insecurities concerning both the measurements and installation technique used. The measurements during the steel pipe installation displayed only very small movements, which would be expected since the piles are supposed to be non-displacement. But the measurements were also inconsistent, which could be caused by parallel activity nearby the area of installation. Another insecurity was that not all parameters used during the installation were known, e.g. pressure, air-flow and water flow. This combination of factors caused the idea of modelling the steel pipe piles to be considered unfeasible.

# 7 Conclusions

Regarding the PLAXIS modelling, the result show that it is possible to set up a plane strain model that achieve movements that are in good agreement with the measured data, if the geometry modelled is elongated. It can be concluded that a model using 50 per cent of the calculated volume expansion together with a pre-auger efficiency of 50 per cent give the best fit in this case. There are however many insecurities involved in the process. One being that a majority of the soil movements are related to the way the pile installation is set up, with regard to amount of volume expansion and pre-auger efficiency, rather than the input parameters of the soil material. Even the pile modelling process are insecure due to the fact that it has not yet been tried out in many different settings and cases. Assuming a 50 per cent pre-auger efficiency have however been confirmed to be accurate both in the case study and the reference project. Still it is not possible to conclude a general advice until the method have been verified in several other cases.

The compared methods of Sagaseta and Rehnman show a varying degree of agreement depending on the type of movement calculated. Sagaseta's method are simple to use when calculating the ground surface movements but increasingly complex when used for displacements towards depth. Nevertheless, it does show a very good agreement at the ground surface points of measurement in the case study. It does not deliver a sufficient result towards depth in this case when using a simplified method of calculation, however it would probably have been possible to achieve a better agreement using a more advanced method. Rehnman's method is simple but contain several different factors that have to be assumed, thus resulting in large insecurities. The method also has a tendency to underestimate the actual soil displacements, except for heave, in the case study. It also underestimates the area of influence with the assumption of zero movements at just one pile lengths distance.

One of the objectives for the thesis was to examine bored steel pipe piles and to try to model the movements measured. The modelling however, ended up being infeasible due to the small movements and by only using the given data in the case of Gamlestadstorget. More extensive measurements and a controlled environment would have been needed to ensure that all movements actually are caused by the pile installation. It can nonetheless be concluded that the installation of bored steel pipe piles only caused minor displacements adjacent to the studied section. However, settlements related to the installation of bored steel pipe piles have been measured in other areas, which have not been within the scope of this thesis, of project Gamlestadstorget.

# 8 Future Research

The modelling of the Gamlestadstorget project was rather complex in the sense that there was a large presence of old constructions and pre-existing piles as well as a large variation between different sections. As a consequence of this a lot of simplifications were made. An interesting future research would be to simulate this problem in a 3-dimensional FEM-program to capture and incorporate more of the complexity into the model, but also to capture the behaviour of the piles in a more accurate way. Further studies could also be performed by using the thesis's way of modelling piles in combination with a more advanced constitutive model. This would allow to better capture the behaviour of the soil and analyse the effect from the piles more in detail.

Varying the pre-auger efficiency gave different results, where 50 per cent gave the best fit in both the case study and the reference project. To verify that this is an accurate efficiency to use, further studies could be conducted including both field measurements of actual removed soil volume but also modelling of different cases. This could also include research on different types of pre-boring and different methods of how it is performed and could end up as some form of guidelines.

Another interesting research would be to do a proper, more extensively monitored, test piling of bored steel pipe piles. One where the ground displacements, type of flushing used and downward pressure is accurately documented. This could be done to investigate and evaluate how different parameters affect the displacements in the area around the installation, to find the best solution for installation.

# References


Barlett, S. F. (2016) *Mohr Coloumb model*, University of Utah, See http://www.civil.utah.edu/~bartlett/CVEEN6920/Mohr-Coulomb Model.pdf (accessed 15/5 2016).

Bjur, H. (2015) Gamlestaden - genesis och strukturella förändringar. In *Gamlestaden, Strukturella förändringar och kulturarvsprocesser - en fallstudie*. (Andersson, S., Oisson, K., and Wetterberg, O.) INEKO. (In Swedish).

Bower, A. F. (2012) *Mechanics of Elastic solids*. Brown University School of Engineering, See http://www.brown.edu/Departments/Engineering/Courses/En221/Notes/Elasticity/Elasticity.htm (accessed 26/2 2016).

Brinkgreve, R. B. J. (2005) Selection of soil models and parameters for geotechnical engineering application In *Proceedings of Geo-Frontiers Congress 2005*. American Society of Civil Engineers, pp. 69-98.

Commision on Pile Research (2010) *Borrade stålrörspålar - Anvisningar för projektering, dimensionering, utförande och kontroll*. Report 104, pp. 52. (In Swedish).

Commision on Pile Research (2015) *Pålstatistik för Sverige 2014*. Stockholm. (In Swedish).

Craig, R. F. & Knappett, J. A. (2012) *Craig's soil mechanics.* 8th edn. London, CRC Press.

De Weck, O. & Kim, I. Y. (2004) *Finite Element Method*. Massachusetts Institute of Technology, See http://web.mit.edu/16.810/www/16.810_L4_CAE.pdf.

Dept of the Army (1985) *Pile Construction FM 5-134.* Dept of the Army.

Edstam, T. (2011) *Massundanträngning i samband med pålslagning i lera*. Sbuf, SBUF, Report 12133. (In Swedish).

Edstam, T. & Kullingsjö, A. (2010) *Ground displacements due to pile driving in Gothenburg clay*. CRC Press.

Gercek, H. (2006) *Poisson's ratio values for rocks*, Sciencedirect, See http://www.sciencedirect.com/science/article/pii/S136516090600075X (accessed 15/5 2016).

Göteborgs Stad (2014) *Strategi För Göteborg 2035, Utbyggnadsplanering, Godkänd Av Byggnadsnämnden Februari 2014*. (In Swedish).

Hagerty, J. & Peck, R. (1971) Heave and Lateral Movements Due to Pile Driving. *Journal of the soil mechanics and foundations division - Proceedings of the American Society of Civil Engineers*.

Hintze, S., Liedberg, S., Massarsch, R., Hanson, M., Elvhammar, H., Lundahl, B. & Rehnman, S.-E. (1997) *Omgivningspåverkan vid pål- och spontslagning*. Pålkommissionen. (In Swedish).

Hussein, M. H. (1993) Driving long precast concrete piles In *Proceedings of 14th International Congress*.

Klasson, P. & Kristensson, T. (2012) *Soil displacement due to installation of lime-cement columns - An investigation of methods to estimate horizontal displacements*. Chalmers University of Technology, Gothenburg.

Langford, J., Karlsrud, K., Lande, E. J., Baardvik, G. & Engen, A. (2016) Begrensskade - Limitation of damage caused by foundation and ground works In *Proceedings of Grundläggningsdagen 2016*, pp. 15.

Larsson, R., Sällfors, G., Bengtsson, P.-E., Alén, C., Bergdahl, U. & Eriksson, L. (2007) *Skjuvhållfasthet - Utvärdering i kohesionsjord*. Swedish Geotechnical Institute, Linköping, See http://www.swedgeo.se/globalassets/publikationer/info/pdf/sgi-i3.pdf (accessed 24/3 2016). (In Swedish).

Lilienberg, A. (1928) *Statsbildningar och stadsplaner i Götaälvs mynningsområde från äldsta tider till omkring adertonhundra* Göteborg, Zachrissons Boktryckeri AB. (In Swedish).

Massarsch, R. (1976) *Soil Movements Caused by Pile Driving In Clay*. Stockholm, pp. 261.

Massarsch, R. K. & Wersäll, C. (2013) Cumulative Lateral Soil Displacement Due to Pile Driving in Soft Clay In *Proceedings of Sound Geotechnical Research to Practice* (Stuedlein, A. W. (ed)). Geotechnical Special Publication 230, pp. 463-480.

Nenonen, P. & Ruul, J. (2011) *Environmental impact of pile driving: An FE-analysis of the displacement of the Skäran bridge.* Chalmers University of Technology, Göteborg.

Olsson, C. & Holm, G. (1993) *Pålgrundläggning.* Solna;Linköping;, Svensk byggtjänst. (In Swedish).

Ottolini, M., Dijkstra, J. & Van Tol, F. (2014) Immediate and long-term installation effects adjacent to an open-ended pile in a layered clay. *Canadian Geotechnical Journal* 52(7):982-991.

Ottosen, N. & Petersson, H. (1992) *Introduction to the finite element method.* Prentice Hall.

Plaxis (2016) *PLAXIS 2D 2016 Manual.*

Randolph, M. F., Carter, J. P. & Wroth, C. P. (1979) Driven Piles In Clay - The Effects Of Installation And Subsequent Consolidation. *Geotechnique* 29(4):361-393.

Sagaseta, C. & Whittle, A. J. (2001) Prediction of ground movements due to pile driving in clay. *Journal of Geotechnical and Geoenvironmental Engineering* 127(1):55-66.

Sagaseta, C., Whittle, A. J. & Santagata, M. (1997) Deformation analysis of shallow penetration in clay. *International Journal for Numerical and Analytical Methods in Geomechanics* 21(10):687-719.

Staden Nya Lödöse (2016) *Staden Nya Lödöse*, See http://www.stadennyalodose.se/nya-lodose/ (accessed 22/3 2016). (In Swedish).

Sweco (2011) *Gamlestads torg och Gamlestadens fabriker, Göteborg - Geoteknisk utbredning för detaljplan PM Geoteknik*. Göteborgs Stad, Fastighetskontoret. (In Swedish).

Swedish Geotechnical Institution (2016) *Jords hållfasthet*, See http://www.swedgeo.se/sv/kunskapscentrum/om-geoteknik-och-miljogeoteknik/geoteknik-och-markmiljo/jords-hallfasthet/skjuvhallfasthet/ (accessed 25/4 2016). (In Swedish).

Swedish Transport Administration (2013) *Gamlestaden blir knytpunkt och resecentrum*, See http://www.trafikverket.se/nara-dig/Vastra-gotaland/Vastsvenska-paketet/Aktuellt-om-Vastsvenska-paketet-och-trangselskatter/2013-06/Gamlestaden-blir-knutpunkt-och-resecentrum/ (accessed 31/3 2016). (In Swedish).

Swedish Transport Administration (2014) Trafikverkets tekniska råd för geokonstruktioner TR Geo 13. Trafikverket, Report TDOK 2013:0668. (In Swedish).

Tomlinson, M. & Woodward, J. (2014a) General principles and practices. In *Pile Design and Construction Practice, Sixth Edition*.) CRC Press, pp. 1-10.

Tomlinson, M. & Woodward, J. (2014b) Piling equipment and methods. In *Pile Design and Construction Practice, Sixth Edition*.) CRC Press, pp. 69-129.

Tomlinson, M. & Woodward, J. (2014c) Types of pile. In *Pile Design and Construction Practice, Sixth Edition*.) CRC Press, pp. 11-68.

Transportation Research Board (2008) *Use of inclinometers for geotechnical instrumentation on transportation projects*. Washington DC.

Vectura (2015a) *Knutpunkt Gamlestadstorg Spårskede 4, 2220/12 Markteknisk undersökningsrapport, Geoteknik MUR/Geo*. Report 13:20. (In Swedish).

Vectura (2015b) *Knutpunkt Gamlestadstorg Spårskede 4, 2220/12 Mängdförteckning Markarbeten med beskrivande text 10.1*. (In Swedish).

Wersäll, C. & Massarsch, R. (2013) Soil Heave Due to Pile Driving in Clay In *Proceedings of Sound Geotechnical to practice* (Stuedlein, A. W. (ed)). Geotechnical Special Publication (GSP 230), pp. 481-499.

Zienkiewicz, O. C., Taylor, R. L. & Zhu, J. Z. (2005) The standard discrete system and origins of the finite element method. In *The Finite Element Method Set (Sixth Edition)*.) Butterworth-Heinemann, Oxford, pp. 1-18.

# Appendices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# Appendix 1 – Partihallen bridge modelling reference project

Calculation of the piles influence of the piling areas soil volume and the resulting expansion.

## Input parameters and calculations

**Piles**
Number of piles: 60
Length of piles: 52 m
Cross sectional area per pile: 275 x 275 mm = 0.0756 $m^2$
Total cross-sectional area of piles: 60 x 0.0756 = 4.538 $m^2$
Total volume of piles: 4.5375 x 52 = 236.0 $m^3$

**Piling area**
Length: 6.4 m
Width: 16.2 m
Area: 6.4 x 16.2 = 103.7 $m^2$
Total volume of piling area: 103.7 x 52 = 5391.4 $m^3$

Pre-auger length: 10 m
Total volume of 100% effective pre-auger: 4.5375 x 10 = 45.38 $m^3$

Percentage installed piles of total piling area: by area 4.538 / 103.7 $m^2$ = 4.38% or by volume 235.95 / 5392.4 $m^3$ = 4.38%

## Axisymmetric case

Circle area formula: $A = r^2\pi$
Radius of circle: $r = \text{sqrt}(A/\pi)$

**Without pre-auger:**
Circular radius of piling area: r1 = sqrt(103.7/π) = 5.745 m
Circular radius of piling area + piles: r2 = sqrt(108.24/π) = 5.869 m

Increase in radius: Δr = r2 – r1 = 0.124 m
Percentage increase in radius: Δr / r1 = 0.124 / 5.745 = 2.16%

**With 50% pre-auger effectiveness:**
Assuming 50% increase in radius at first 10 meters of pile length.

Increase in radius: $\Delta r_{pa}$ = (r2 – r1)/2 = 0.0622 m
Percentage increase in radius $\Delta r_{pa}$ / r1 =1.08%

**With 100% pre-auger effectiveness:**
Assuming 0% increase in radius at first 10 meters of pile length.

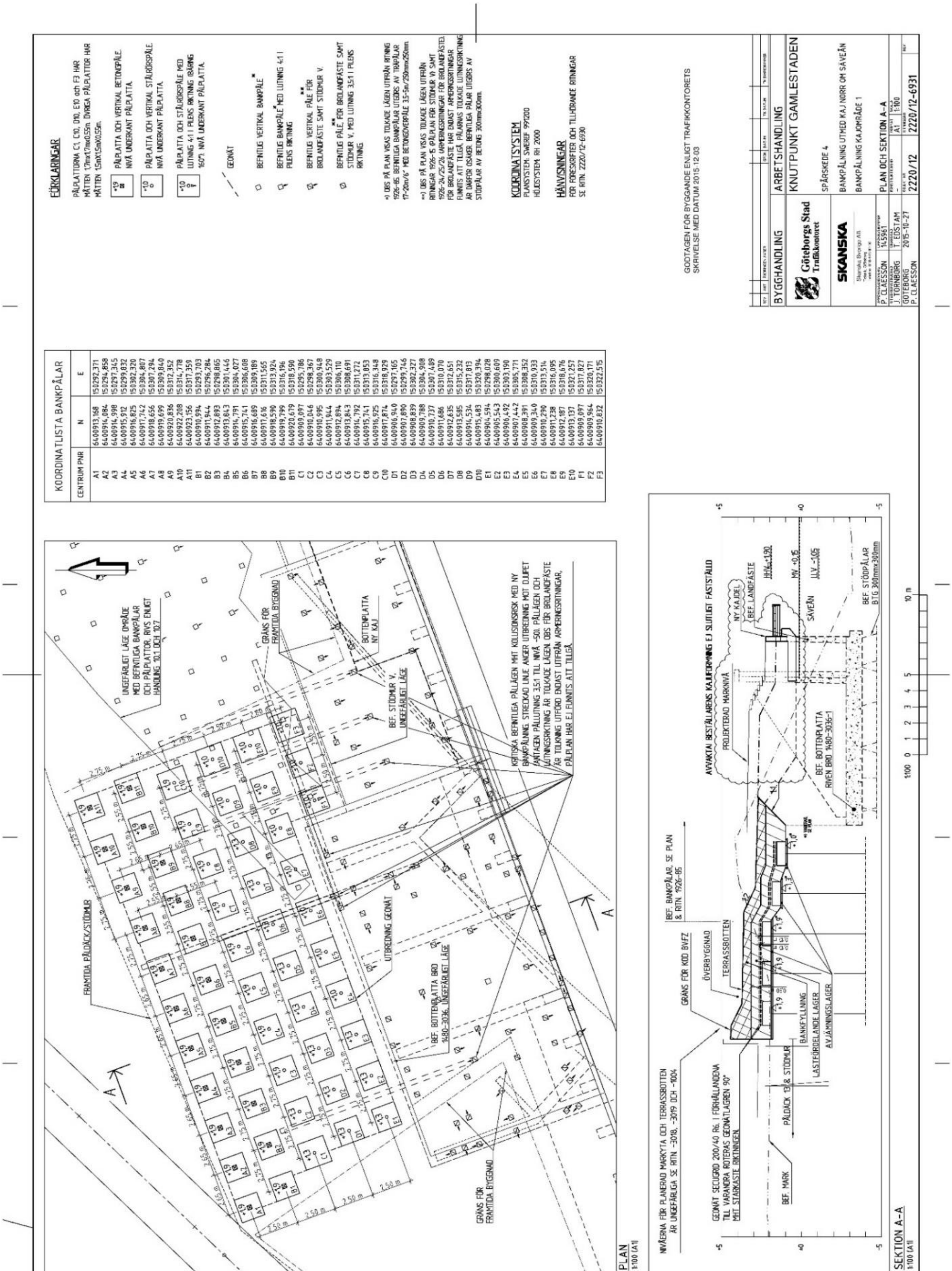
**Model of axisymmetric case**

The model was set up with the boundaries equalling a 250 metre width and 100 metre depth, the right and bottom boundary are fixed in x- and y-direction respectively. The yellow soil cluster representing the piling area soil volume and the piles was modelled as being 52 metres deep, equalling the length of the piles. The soil cluster is fixed in y-direction at the top and bottom. A line at 10 metres depth is used for modelling the pre-augering. The ground water level is set at ground level.

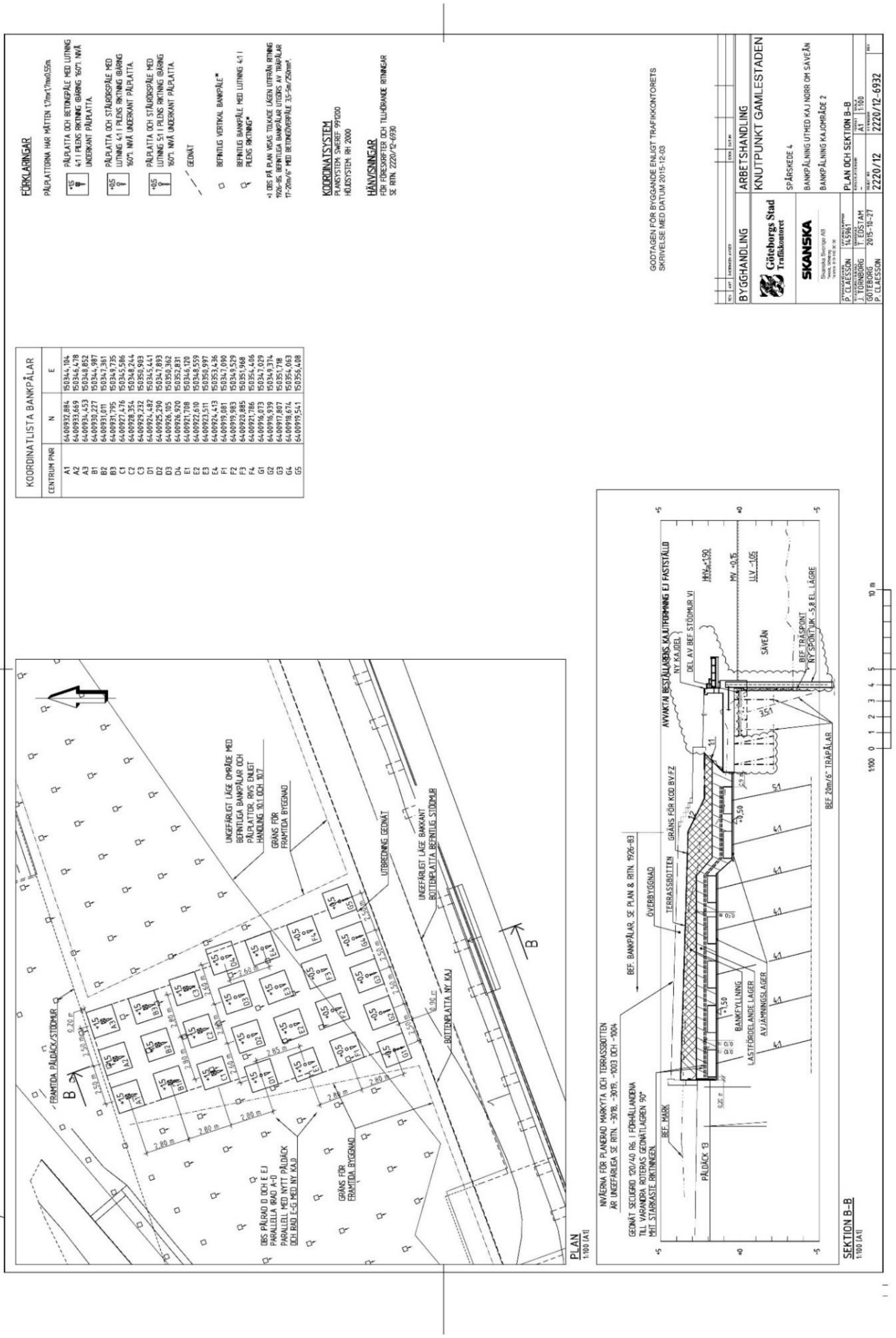
# Appendix 2 – Maps and blueprints of the project

**Partition of the different geotechnical areas and location of geotechnical test points**

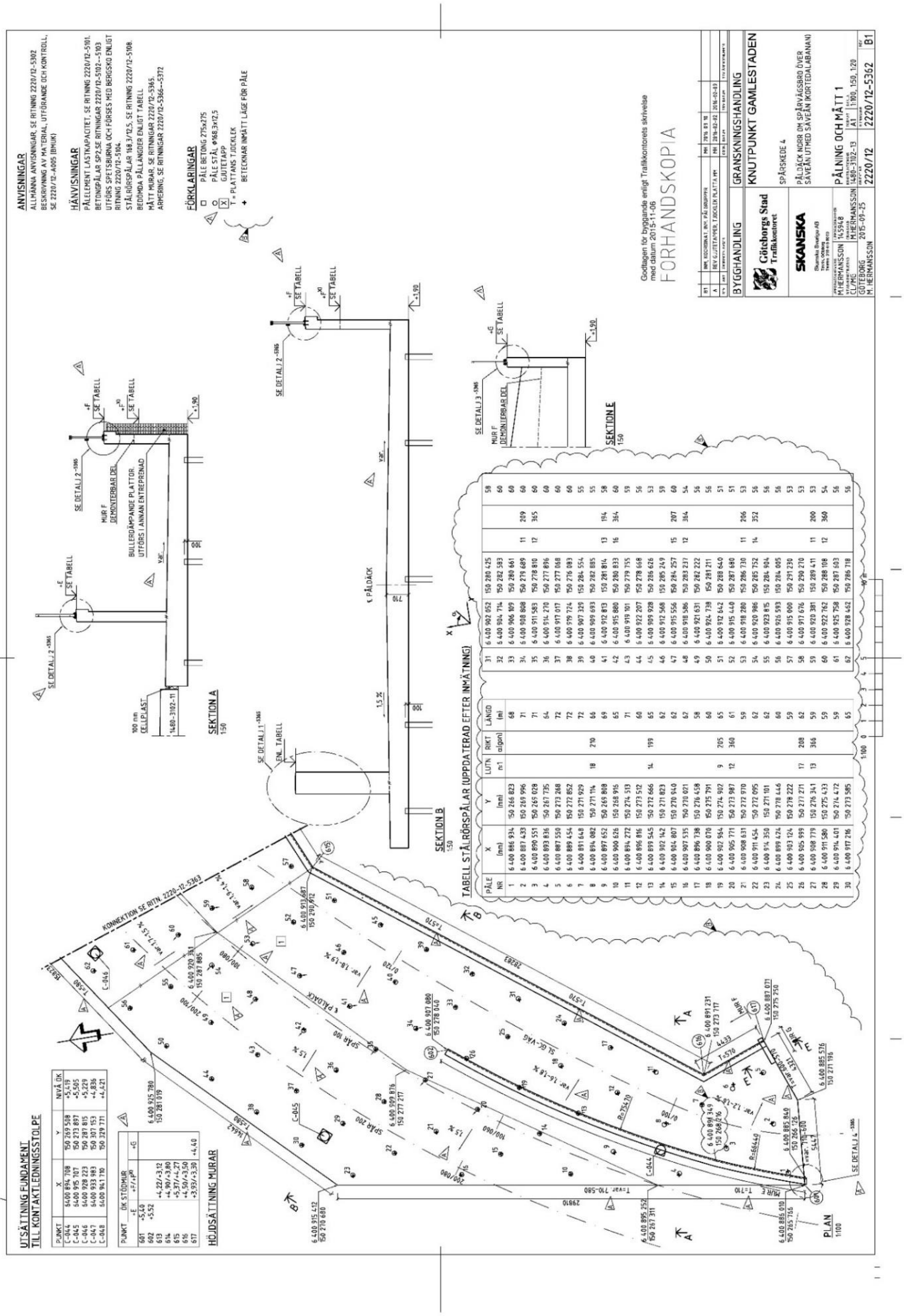
## Plan and section of pile cap foundation -6615

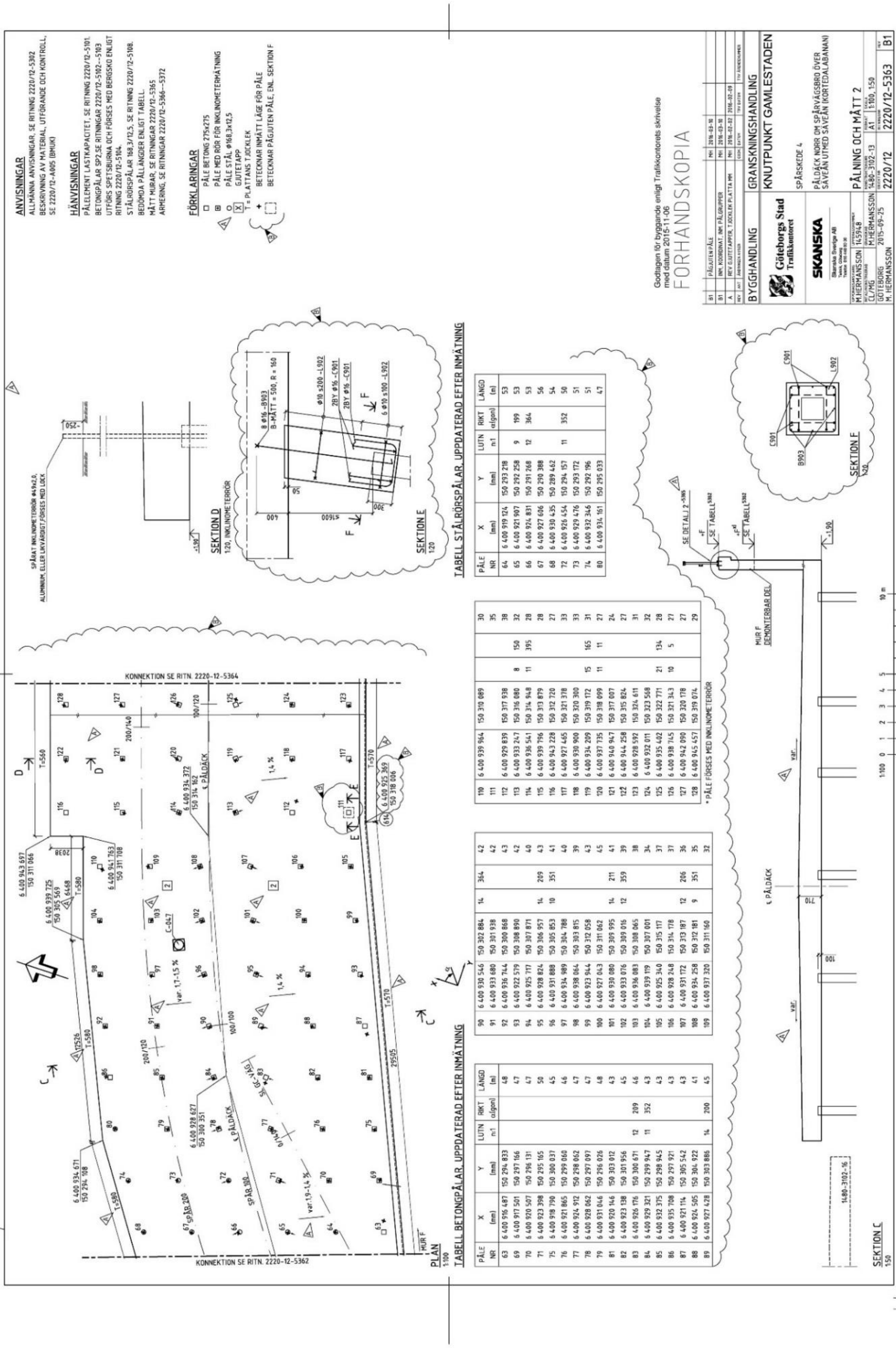


**Plan and section of pile cap foundation -6616**



**Plan and section of pile group -13**





ANVISNINGAR
HÄNVISNINGAR
FÖRKLARINGAR
TABELL BETONGPÅLAR, UPPDATERAD EFTER INMÄTNING
TABELL STÅLRÖRSPÅLAR, UPPDATERAD EFTER INMÄTNING
SEKTION C
SEKTION D
SEKTION E
SEKTION F
PLAN
FÖRHANDSKOPIA
BYGGHANDLING
GRANSKNINGSHANDLING
Göteborgs Stad Trafikkontoret
SKANSKA
KNUTPUNKT GAMLESTADEN
SPÅRSKEDE 4
PÅLNING OCH MÅTT 2
2220/12-5363
B1

ANVISNINGAR

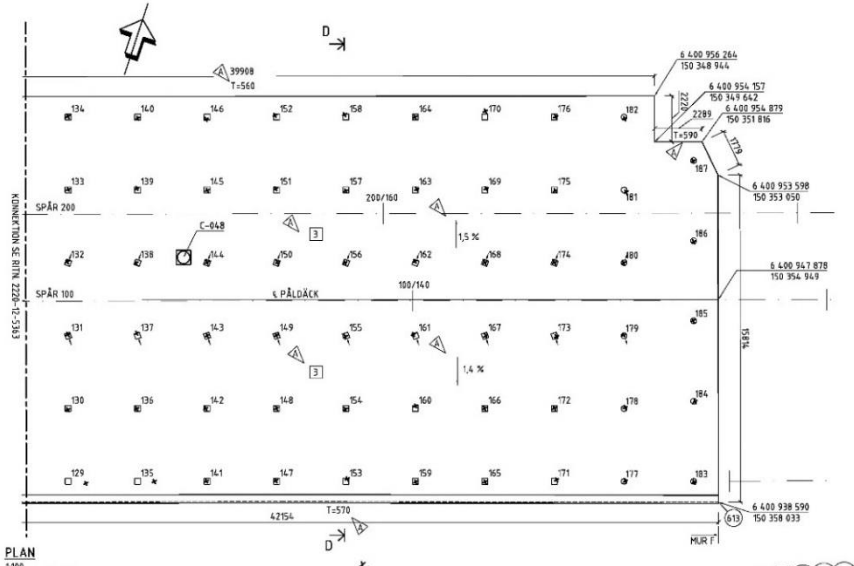
ALLMÄNNA ANVISNINGAR, SE RITNING 2220/12-5302
BESKRIVNING AV MATERIAL, UTFÖRANDE OCH KONTROLL, SE 2220/12-A005 (BMUK)

HÄNVISNINGAR

PÅLELEMENT LASTKAPACITET, SE RITNING 2220/12-5101.
BETONGPÅLAR SP2,SE RITNINGAR 2220/12-5102--5103 UTFÖRS SPETSBURNA OCH FÖRSES MED BERGSKO ENLIGT RITNING 2220/12-5104.
STÅLRÖRSPÅLAR 168,3/12,5, SE RITNING 2220/12-5108.
BEDÖMDA PÅLLÄNGDER ENLIGT TABELL.
MÅTT MURAR, SE RITNINGAR 2220/12-5365
ARMERING, SE RITNINGAR 2220/12-5366--5372

FÖRKLARINGAR

□ PÅLE BETONG 275x275
○ PÅLE STÅL ⌀168,3x12,5
[X] GJUTETAPP
+ BETECKNAR INMÄTT PÅLE

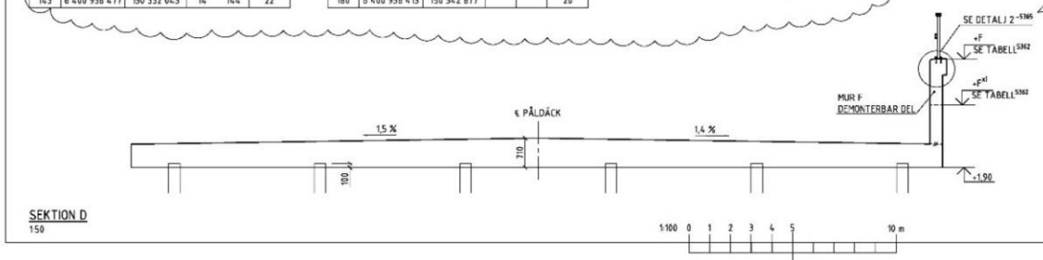



TABELL BETONGPÅLAR, UPPDATERAD EFTER INMÄTNING

| PÅLE NR | X (mm) | Y (mm) | LUTN n:1 | RIKT α(gon) | LÄNGD (m) |
|---|---|---|---|---|---|
| 129 | 6 400 929 861 | 150 328 709 | | | 31 |
| 130 | 6 400 932 966 | 150 326 766 | | | 31 |
| 131 | 6 400 936 421 | 150 325 632 | 13 | 161 | 27 |
| 132 | 6 400 939 742 | 150 324 496 | 14 | 14 | 26 |
| 133 | 6 400 943 080 | 150 323 450 | | | 27 |
| 134 | 6 400 946 436 | 150 322 325 | | | 27 |
| 135 | 6 400 930 983 | 150 331 822 | | | 23 |
| 136 | 6 400 934 057 | 150 329 918 | | | 24 |
| 137 | 6 400 937 523 | 150 328 771 | 15 | 134 | 23 |
| 138 | 6 400 940 732 | 150 327 674 | 13 | 399 | 25 |
| 139 | 6 400 944 170 | 150 326 590 | | | 26 |
| 140 | 6 400 947 406 | 150 325 560 | | | 26 |
| 141 | 6 400 931 791 | 150 334 218 | | | 21 |
| 142 | 6 400 935 113 | 150 333 076 | | | 22 |
| 143 | 6 400 938 477 | 150 332 043 | 14 | 144 | 22 |
| 144 | 6 400 941 835 | 150 330 919 | 14 | 14 | 24 |
| 145 | 6 400 945 151 | 150 329 840 | | | 24 |
| 146 | 6 400 948 390 | 150 328 707 | | | 24 |
| 147 | 6 400 932 861 | 150 337 391 | | | 21 |
| 148 | 6 400 936 202 | 150 336 328 | | | 21 |
| 149 | 6 400 939 544 | 150 335 211 | 9 | 142 | 22 |
| 150 | 6 400 942 906 | 150 334 150 | 18 | 1 | 23 |
| 151 | 6 400 946 261 | 150 332 962 | | | 23 |
| 152 | 6 400 949 617 | 150 331 812 | | | 24 |
| 153 | 6 400 934 025 | 150 340 640 | | | 20 |
| 154 | 6 400 937 196 | 150 339 498 | | | 21 |
| 155 | 6 400 940 528 | 150 338 396 | 16 | 149 | 22 |
| 156 | 6 400 943 922 | 150 337 263 | 16 | 5 | 23 |
| 157 | 6 400 947 220 | 150 336 207 | | | 24 |
| 158 | 6 400 950 707 | 150 335 000 | | | 25 |
| 159 | 6 400 934 928 | 150 343 834 | | | 20 |
| 160 | 6 400 938 413 | 150 342 677 | | | 20 |
| 161 | 6 400 941 665 | 150 341 488 | 13 | 165 | 21 |
| 162 | 6 400 945 042 | 150 340 458 | 14 | 399 | 23 |
| 163 | 6 400 948 373 | 150 339 436 | | | 24 |
| 164 | 6 400 951 660 | 150 338 266 | | | 25 |
| 165 | 6 400 935 984 | 150 347 014 | | | 20 |
| 166 | 6 400 939 347 | 150 345 876 | | | 20 |
| 167 | 6 400 942 692 | 150 344 834 | 14 | 160 | 21 |
| 168 | 6 400 946 046 | 150 343 682 | 14 | 22 | 23 |
| 169 | 6 400 949 455 | 150 342 618 | | | 24 |
| 170 | 6 400 953 004 | 150 341 403 | | | 24 |
| 171 | 6 400 937 155 | 150 350 266 | 8 | 295 | 20 |
| 172 | 6 400 940 423 | 150 349 074 | | | 21 |
| 173 | 6 400 943 726 | 150 348 080 | 10 | 153 | 21 |
| 174 | 6 400 947 102 | 150 346 870 | 8 | 6 | 23 |
| 175 | 6 400 950 404 | 150 345 814 | | | 24 |
| 176 | 6 400 953 815 | 150 344 715 | | | 24 |

TABELL STÅLRÖRSPÅLAR, UPPDATERAD EFTER INMÄTNING

| PÅLE NR | X (mm) | Y (mm) | LUTN n:1 | RIKT α(gon) | LÄNGD (m) |
|---|---|---|---|---|---|
| 177 | 6 400 938 137 | 150 353 441 | | | 23 |
| 178 | 6 400 941 518 | 150 352 308 | | | 24 |
| 179 | 6 400 944 833 | 150 351 157 | 9 | 150 | 24 |
| 180 | 6 400 948 128 | 150 350 051 | 13 | 394 | 24 |
| 181 | 6 400 951 416 | 150 349 088 | | | 23 |
| 182 | 6 400 954 768 | 150 347 909 | | | 26 |
| 183 | 6 400 939 131 | 150 356 610 | | | 23 |
| 184 | 6 400 942 854 | 150 355 434 | | | 24 |
| 185 | 6 400 946 534 | 150 354 114 | | | 22 |
| 186 | 6 400 950 162 | 150 352 944 | | | 23 |
| 187 | 6 400 953 862 | 150 351 698 | | | 25 |




Godtagen för byggande enligt Trafikkontorets skrivelse med datum 2015-11-06

| | | | | | | |
|---|---|---|---|---|---|---|
| B | | INMÄTT PÅLGRUPPER | MH | 2016-03-10 | | |
| A | | REV GJUTETAPPER, TEXT | MH | 2016-02-02 | 2016-02-09 | |

BYGGHANDLING | ARBETSHANDLING

Göteborgs Stad Trafikkontoret

SKANSKA
Skanska Sverige AB

KNUTPUNKT GAMLESTADEN
SPÅRSKEDE 4
PÅLDÄCK NORR OM SPÅRVÄGSBRO ÖVER SÄVEÅN UTMED SÄVEÅN (KORTEDALABANAN)

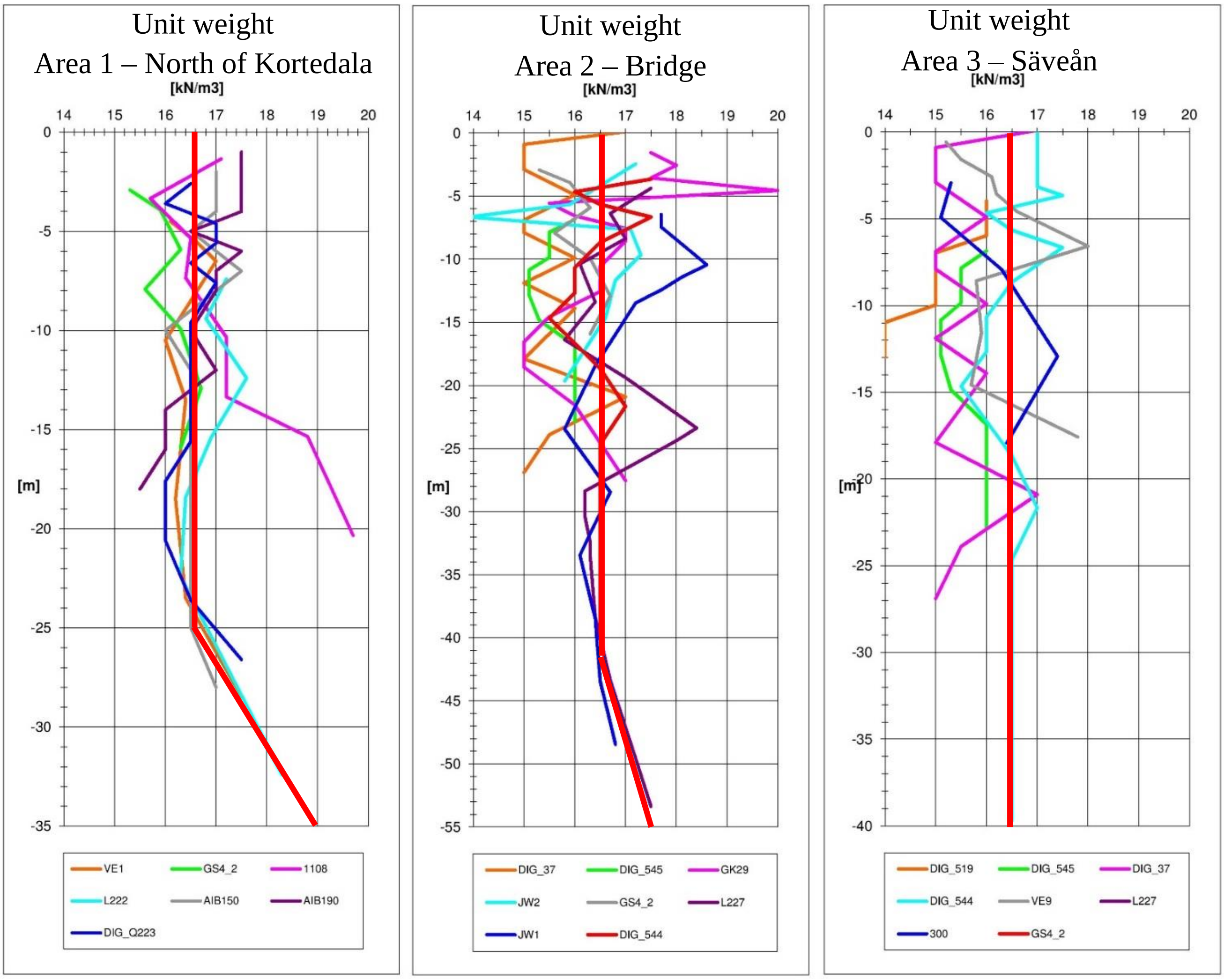
PÅLNING OCH MÅTT 3

M.HERMANSSON | 145948
CL/MG | M.HERMANSSON | 1480-3102-13 | A1 | 1:100, 1:50
GÖTEBORG 2015-09-25 M. HERMANSSON | 2220/12 | 2220/12-5364 | B

# Appendix 3 – Evaluation of soil parameters

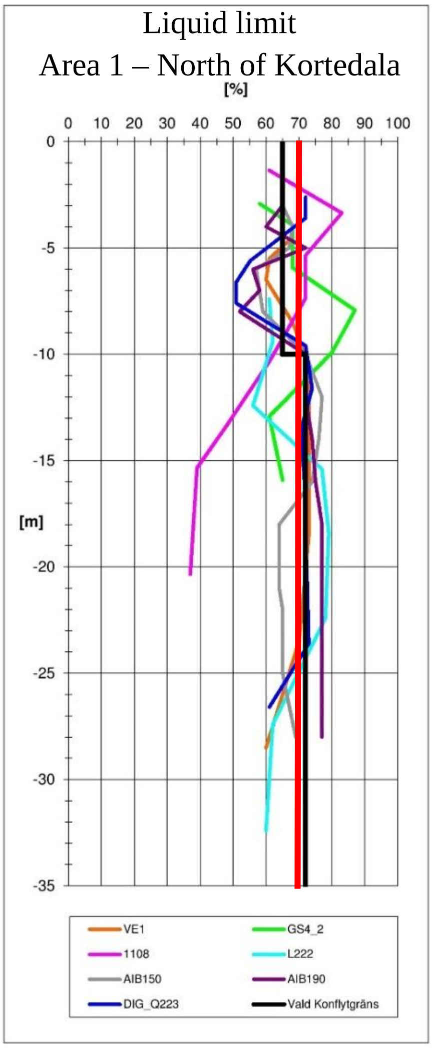
## Unit weight

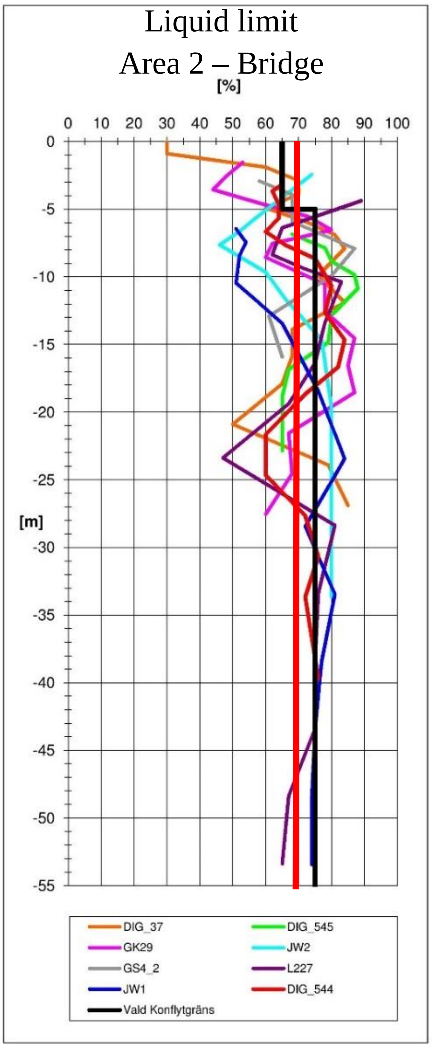
Investigated unit weights from test sites in the three areas closest to the section of choice. The red line equals the chosen soil weight of 16.5 kN/m$^3$.

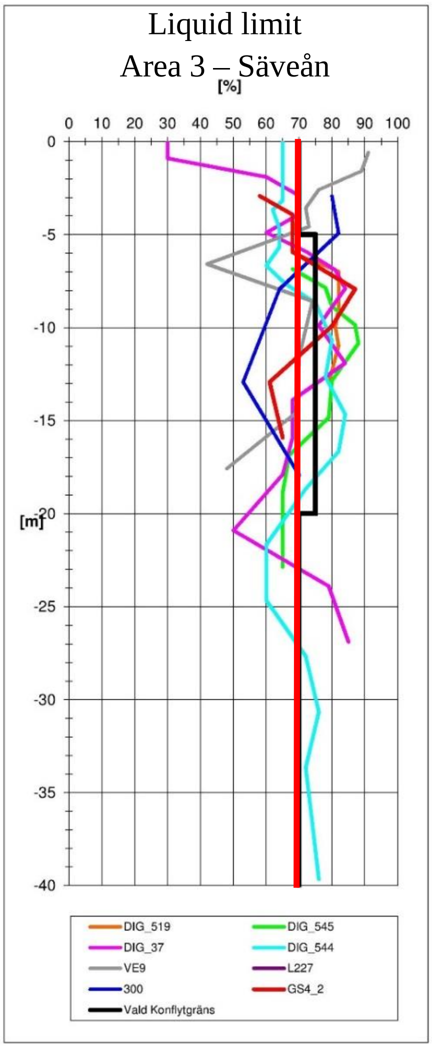


## Liquid limit

Investigated liquid limit from test sites in the three areas closest to the section of choice. The red line equals the chosen liquid limit of 70%.

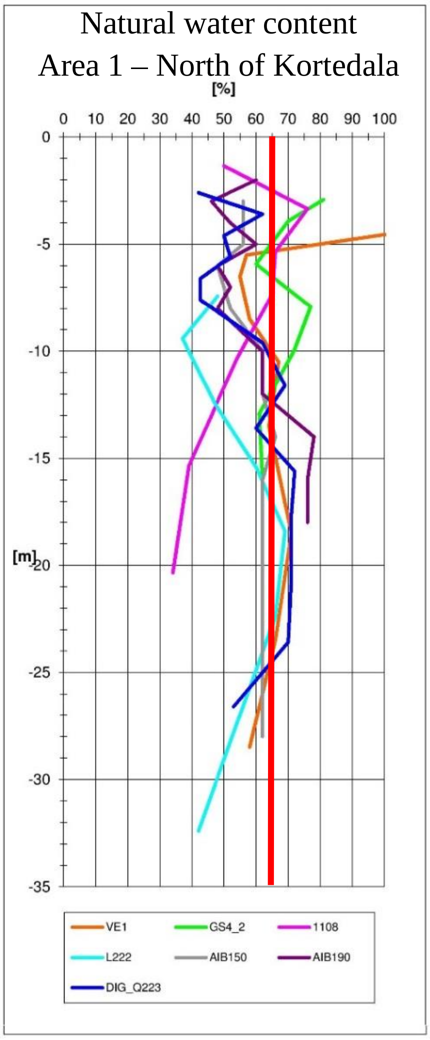


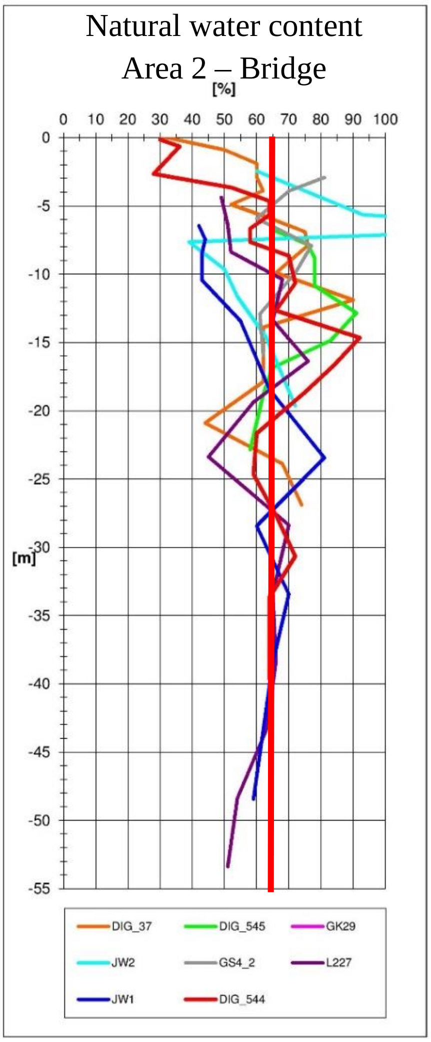



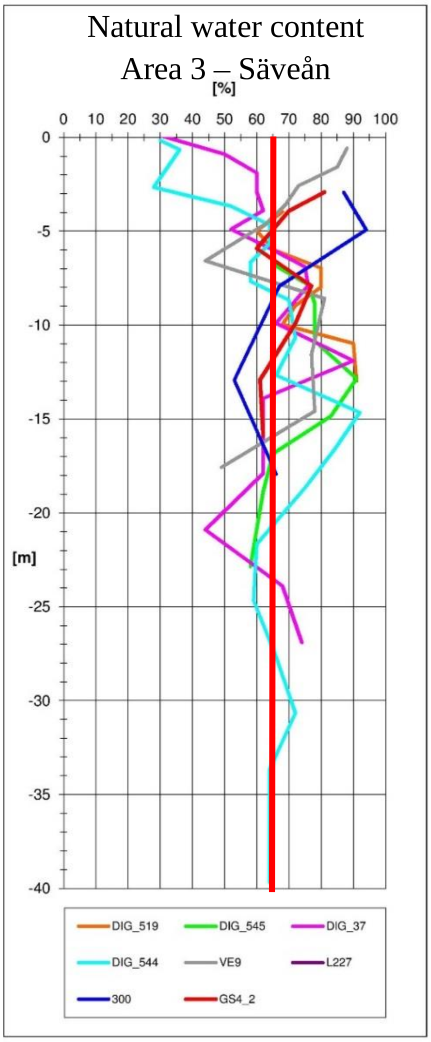



**Natural water content**

Investigated natural water content from test sites in the three areas closest to the section of choice. The red line equals the chosen natural water content of 65%.

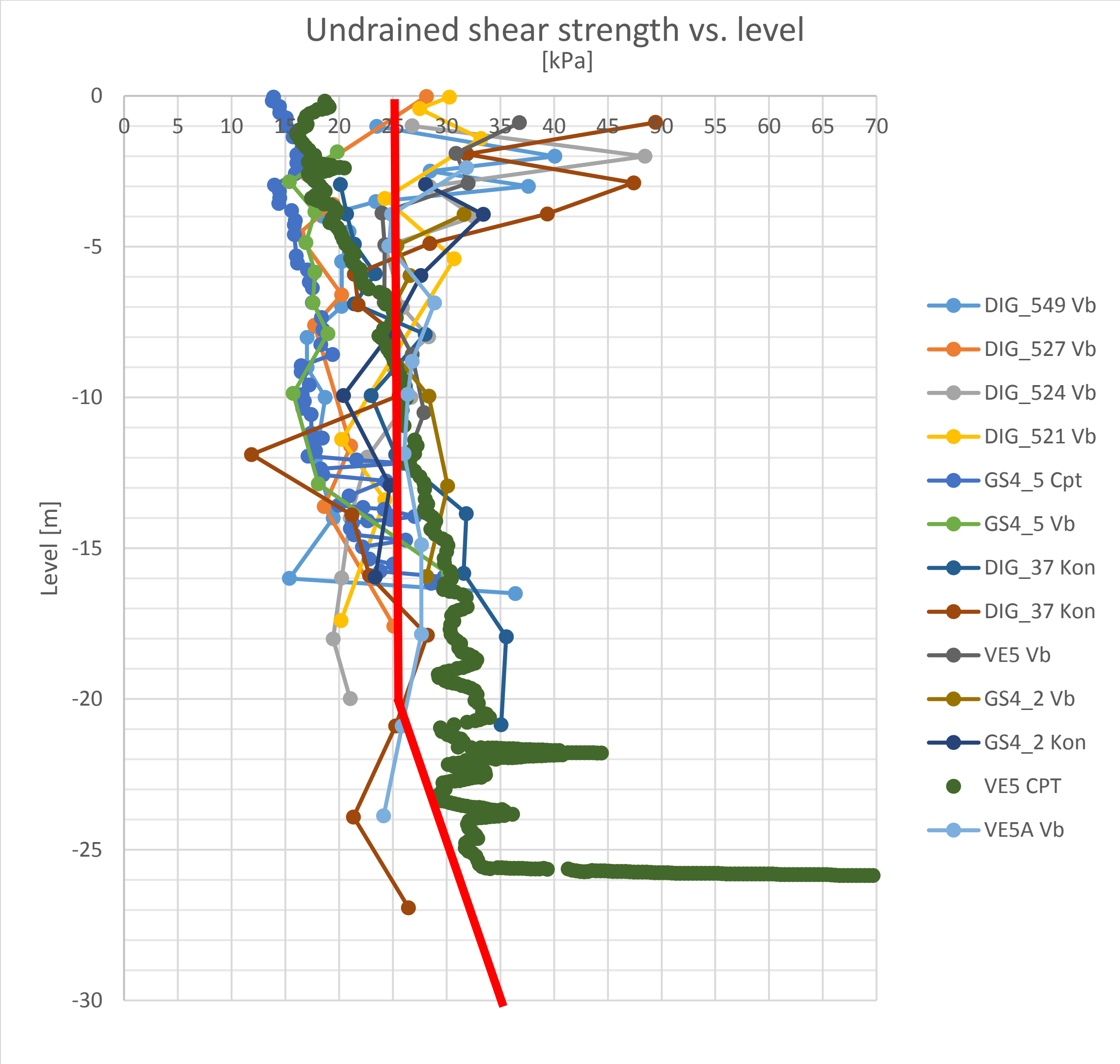


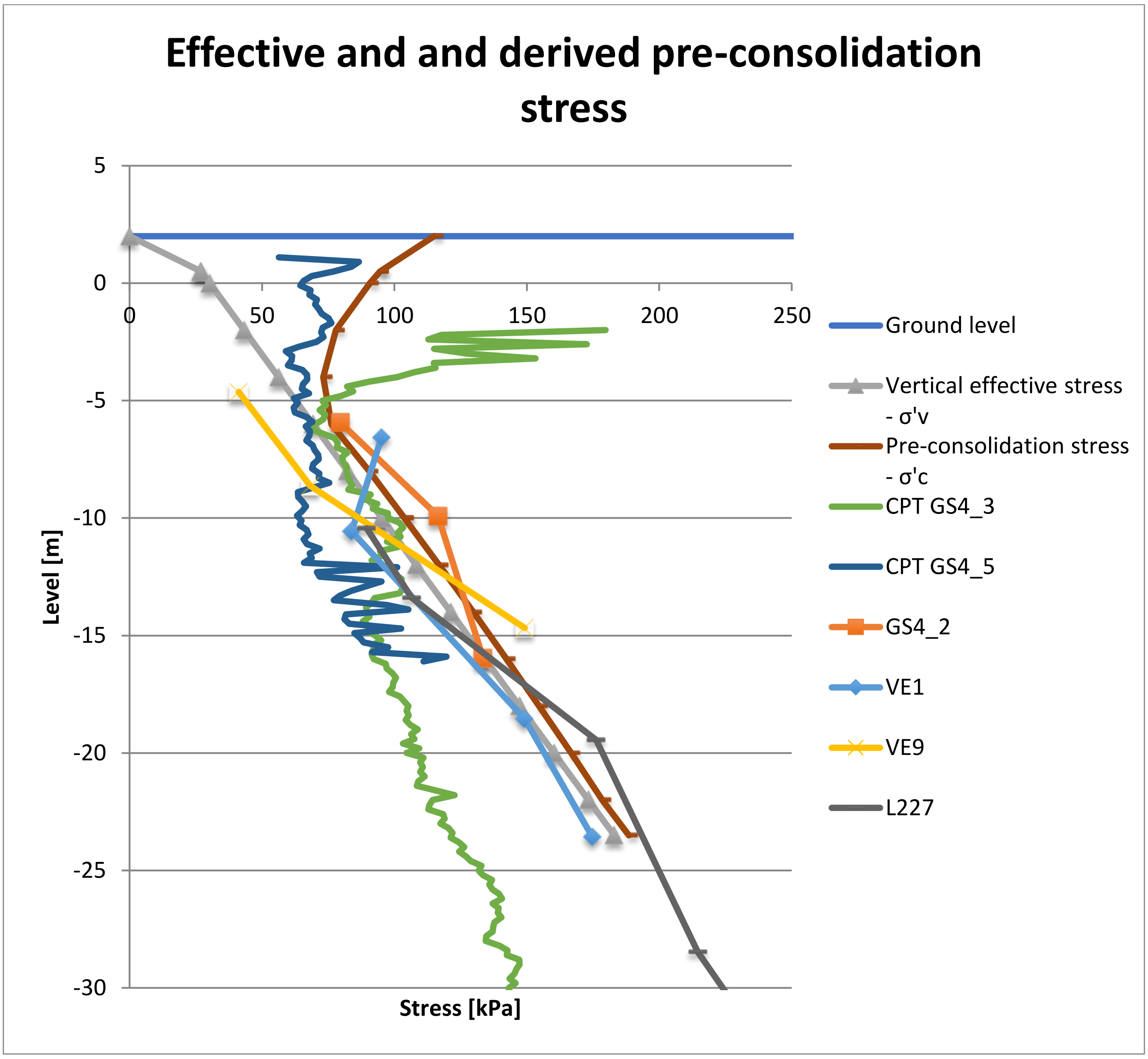
**Undrained shear strength**

Investigated undrained shear strength from test sites closest to the section of choice. The red line equals the chosen undrained shear strength, 25 kPa down to -20 and then increasing with 1.1*depth.



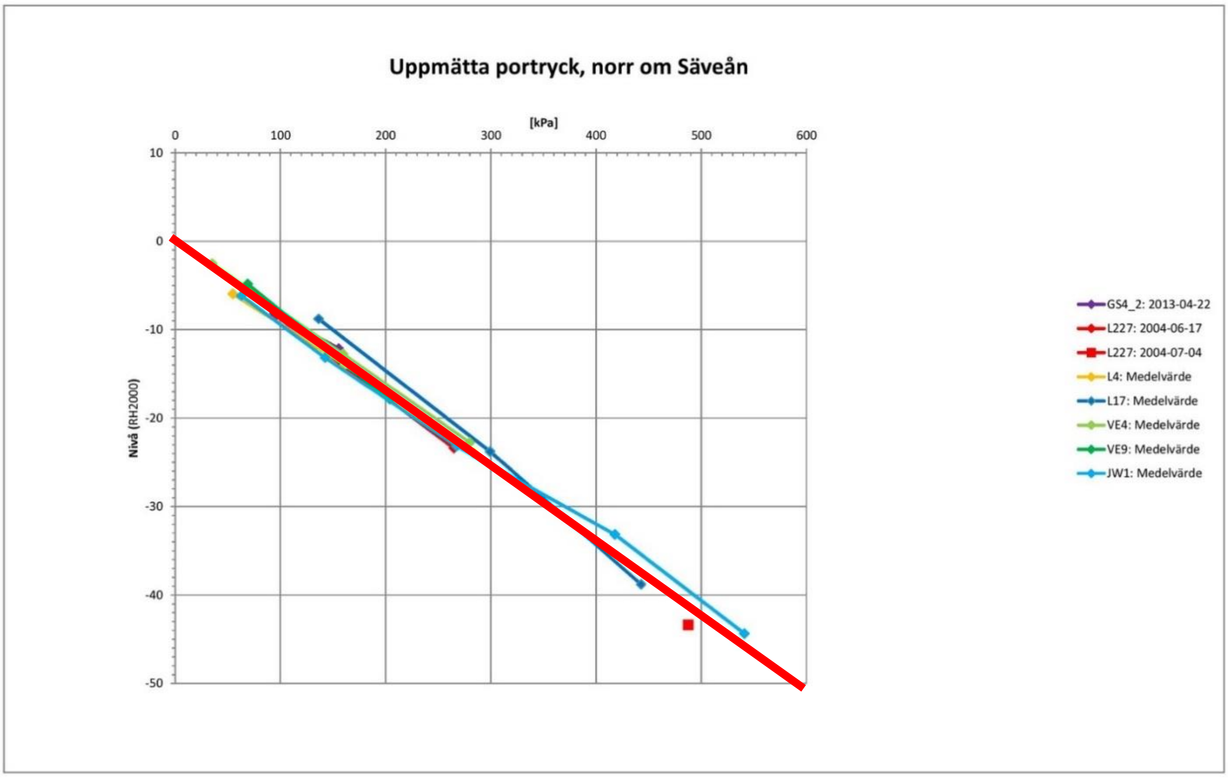
**Pre-consolidation stress**

Investigated pre-consolidation stress from test sites closest to the section of choice. The red line equals the approximated pre consolidation stress, $\sigma'_c$.

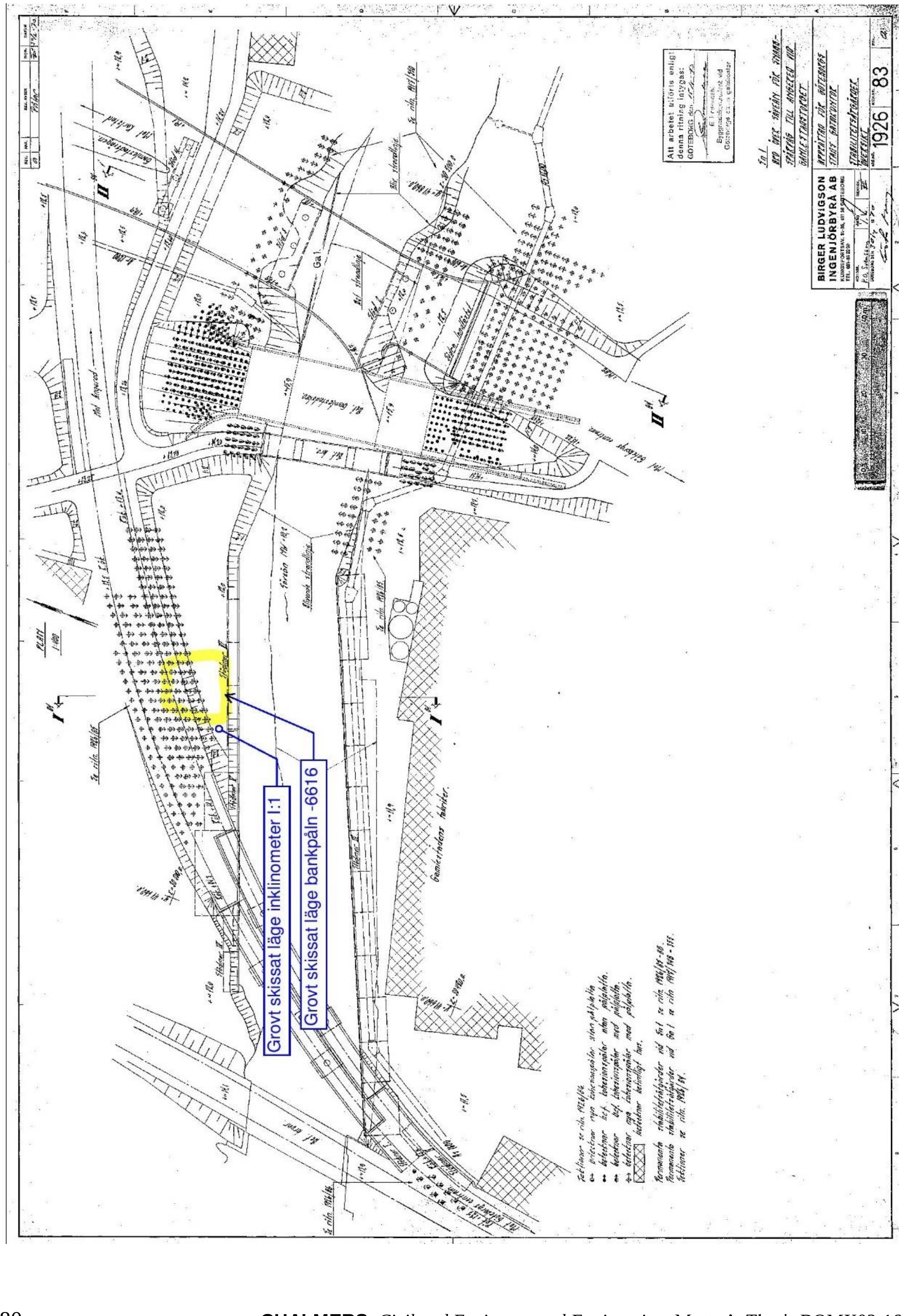


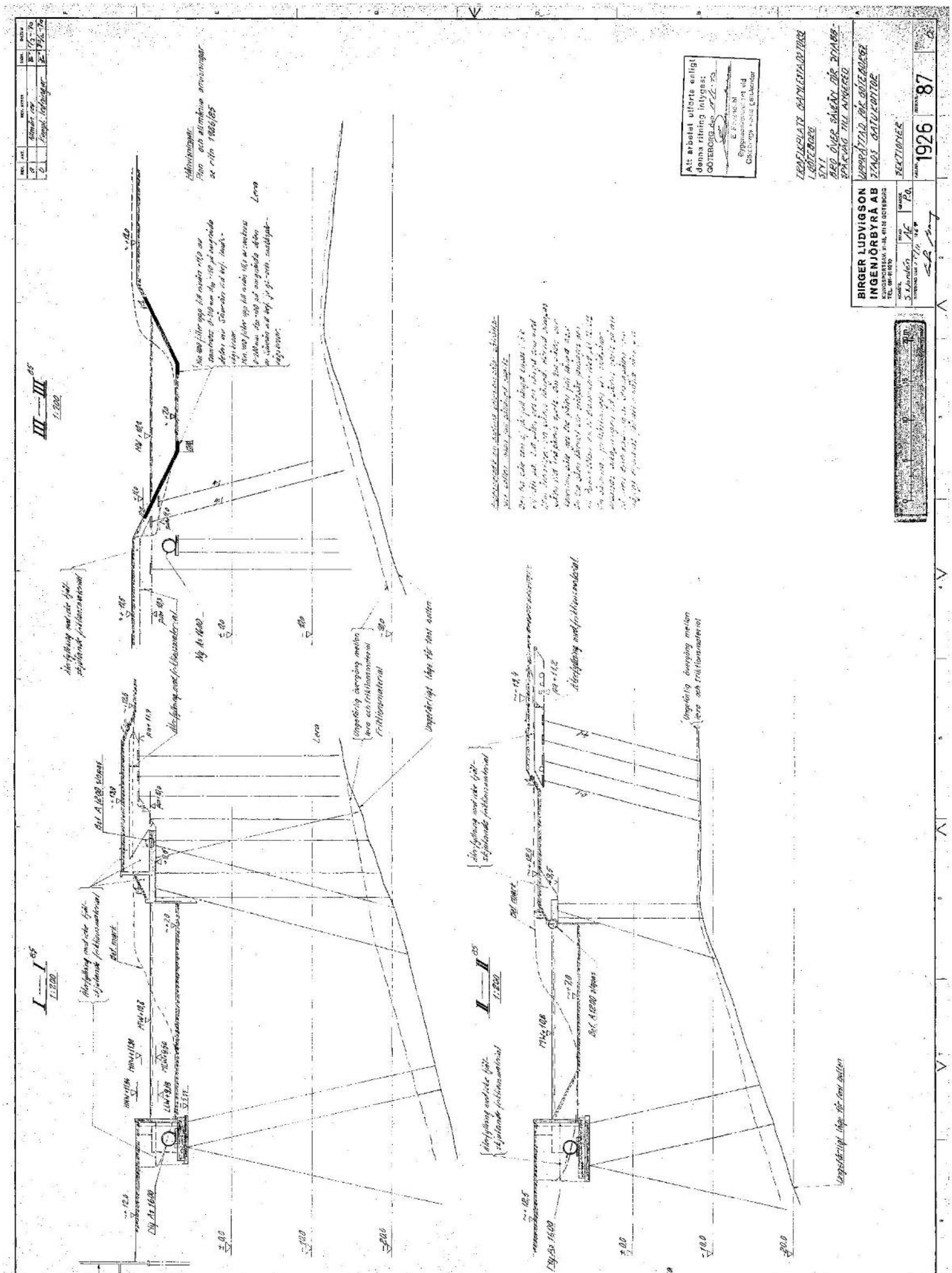
**Pore water pressure**

Investigated pore water pressure from test sites closest to the section of choice. The red line equals the approximated pore water pressure.



# Appendix 4 – Overview of pre-existing constructions



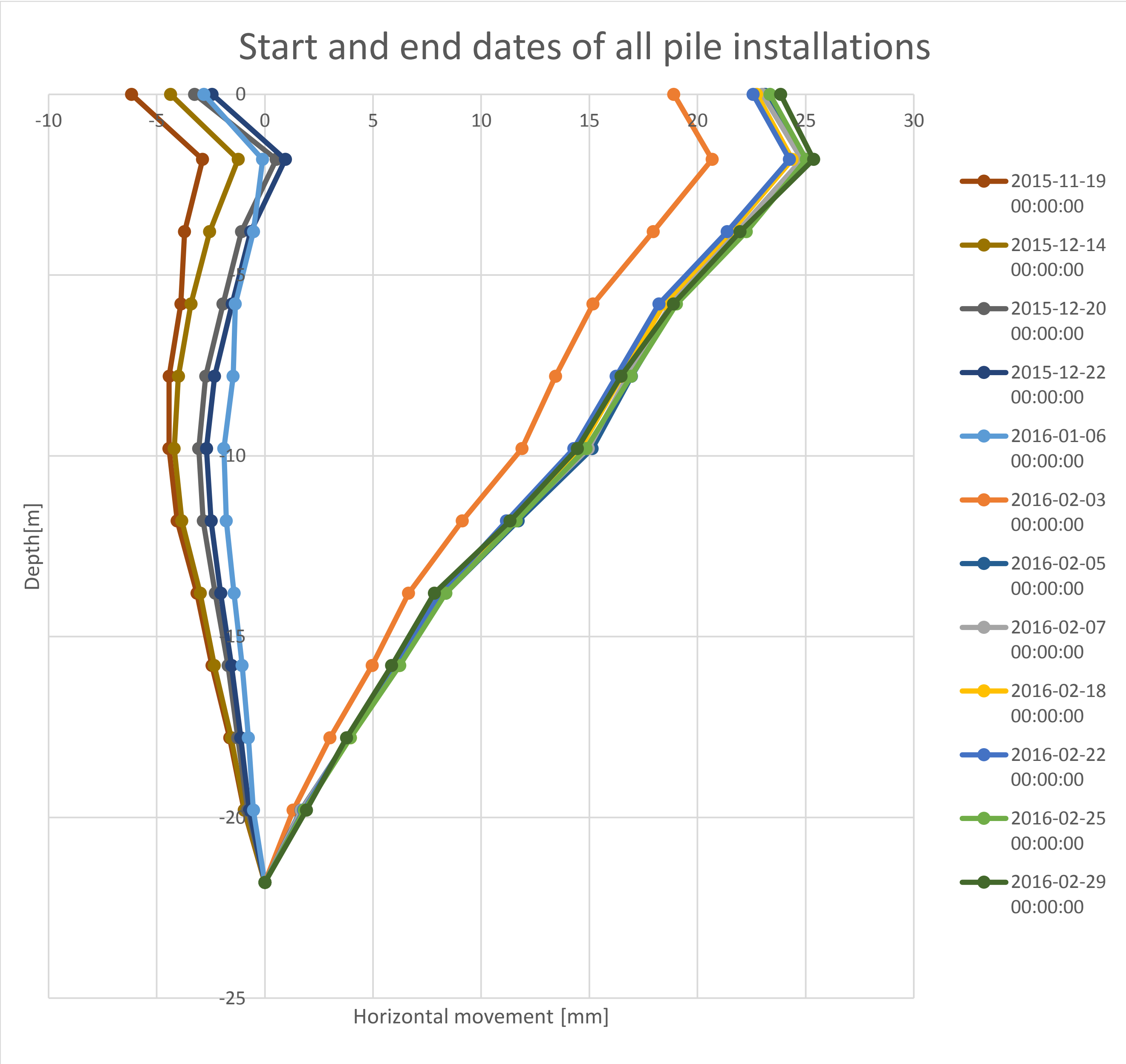

BIRGER LUDVIGSON
INGENJÖRBYRÅ AB
1926
87

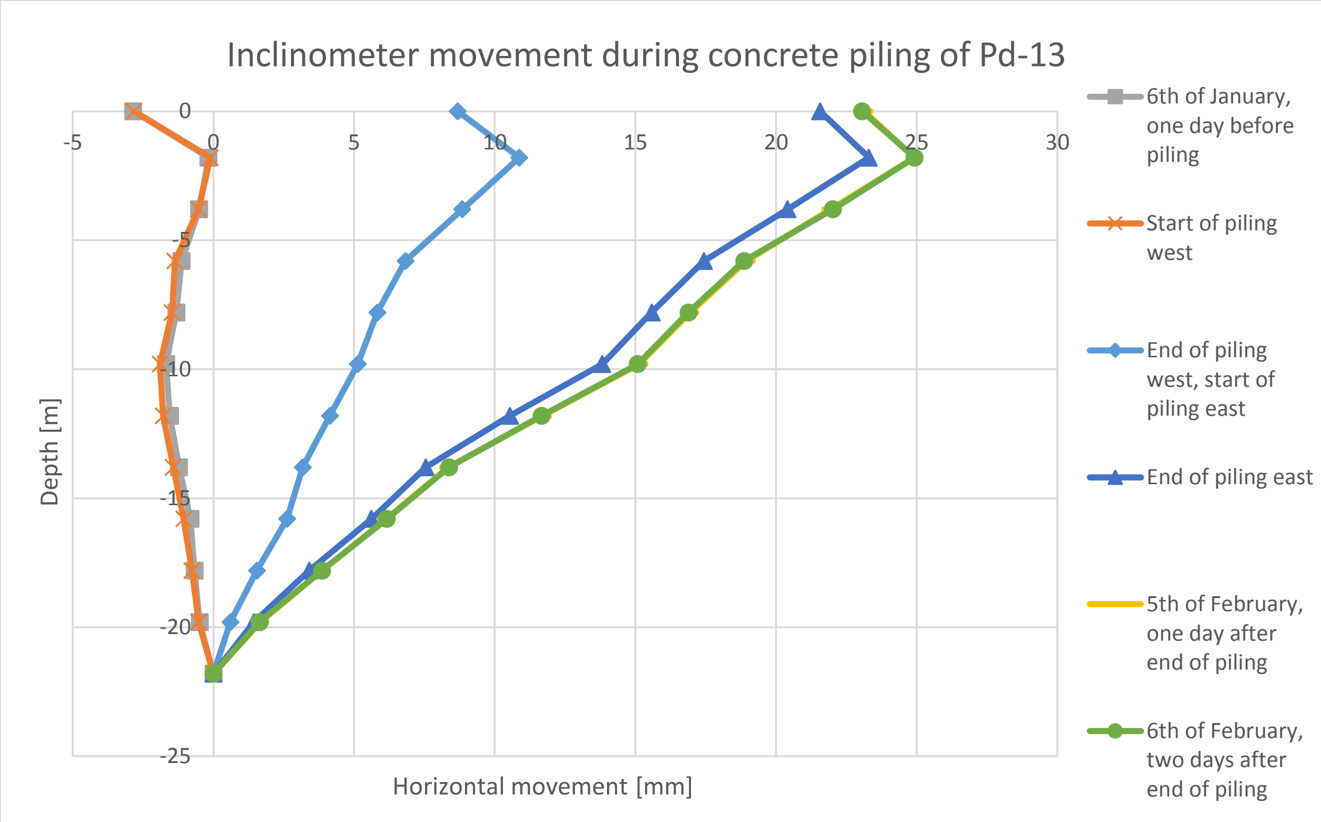
# Appendix 5 –Measurement readings

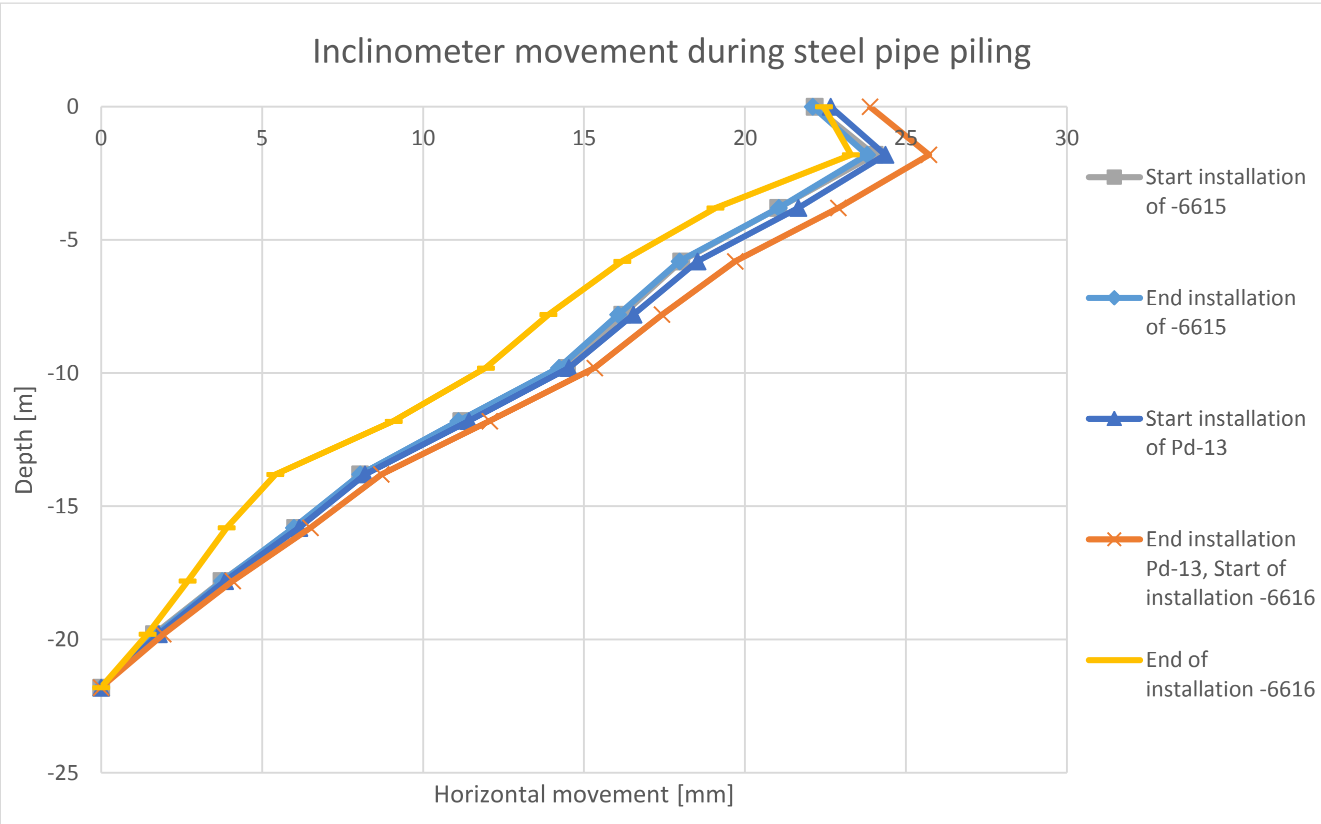
The measurements were performed with one inclinometer and a series of ground settlement gauges on the quayside.

### Inclinometer readings

Original, uncorrected, graphs of the inclinometer movements registered during the concrete piling of Pd-13 and the steel pipe piling of -6615, Pd-13 and -6616.

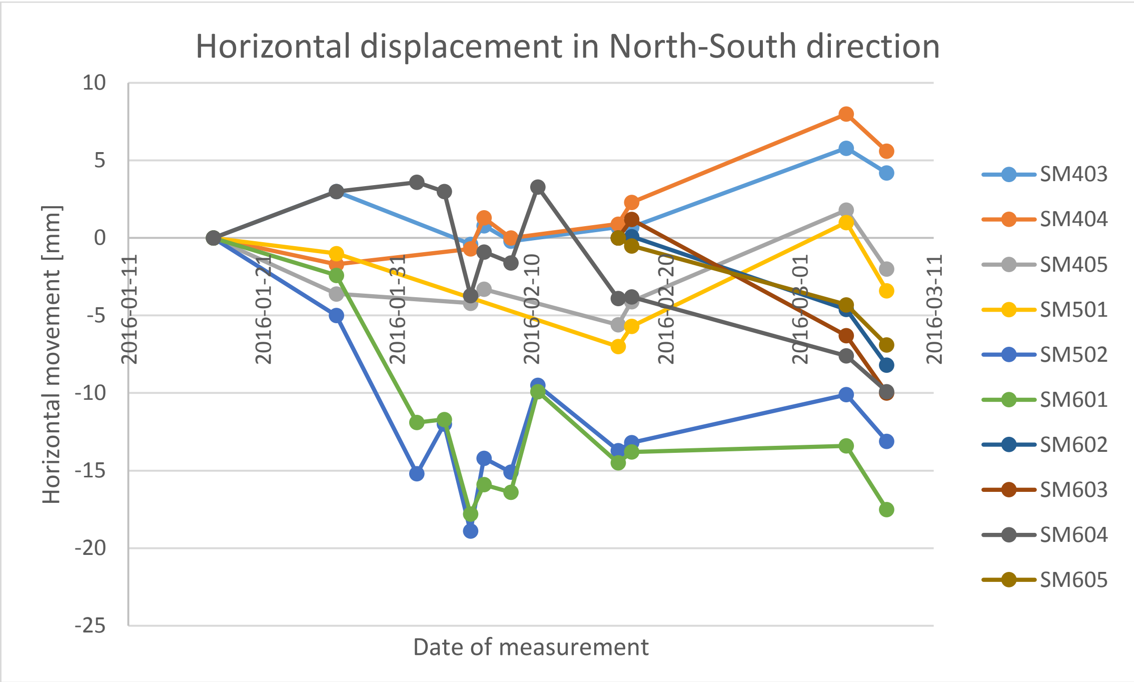


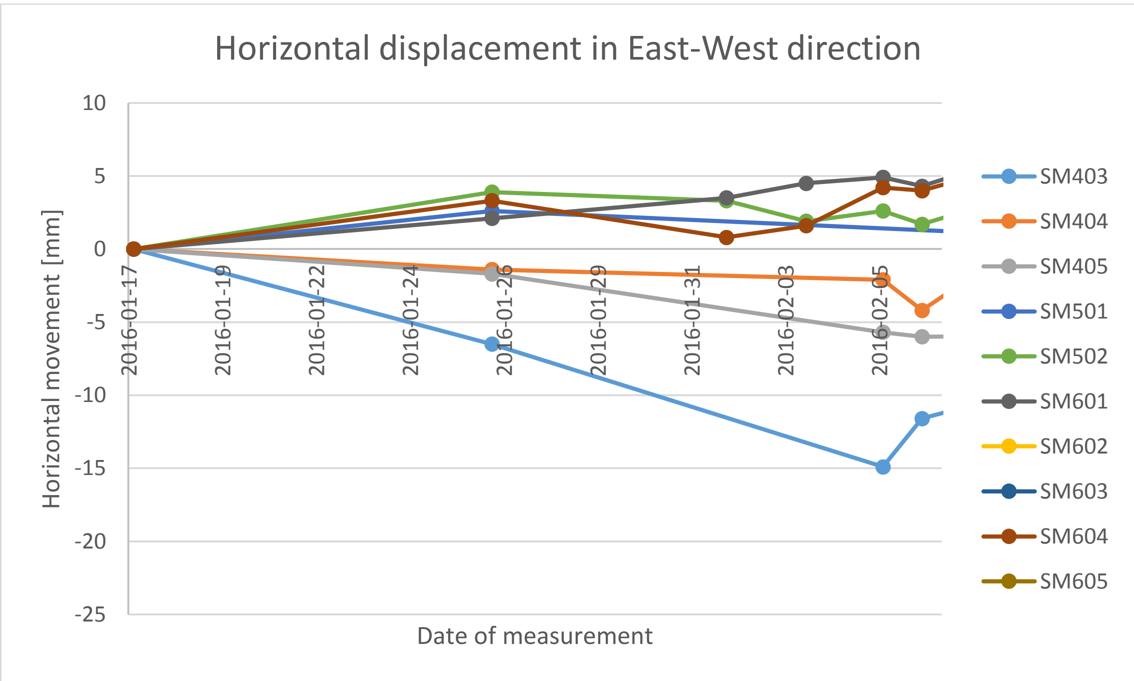

Inclinometer movement during concrete piling of Pd-13
Depth [m]
Horizontal movement [mm]
6th of January, one day before piling
Start of piling west
End of piling west, start of piling east
End of piling east
5th of February, one day after end of piling
6th of February, two days after end of piling

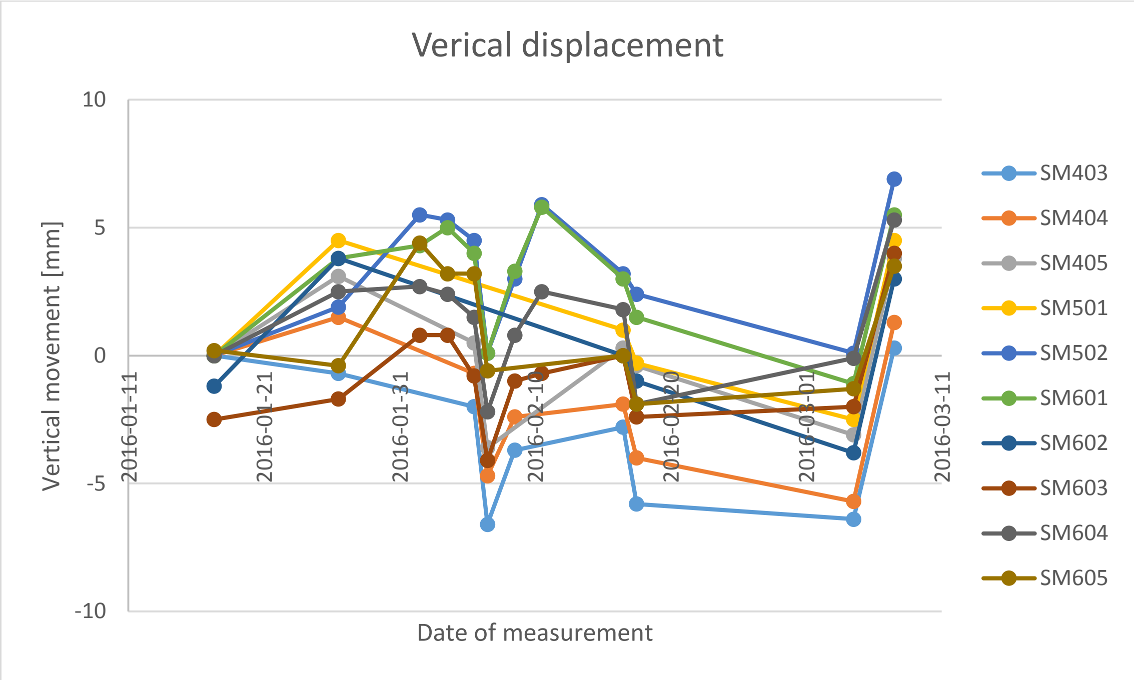

Inclinometer movement during steel pipe piling
Depth [m]
Horizontal movement [mm]
Start installation of -6615
End installation of -6615
Start installation of Pd-13
End installation Pd-13, Start of installation -6616
End of installation -6616

**Ground settlement gauges' readings**

All measurement readings from the ground settlement gauges during the period of interest are shown below. The measurements are divided into graphs showing the horizontal displacement in north-south direction and east-west direction as well as the vertical displacement.






**Recalculation of ground settlement gauges**

The recalculation was performed to correct for the movements of the piles installed before the measurements started. It was implemented for the vertical movement and horizontal movement in N-S direction for the two gauges SM502 and SM601, since these was of the most interest.

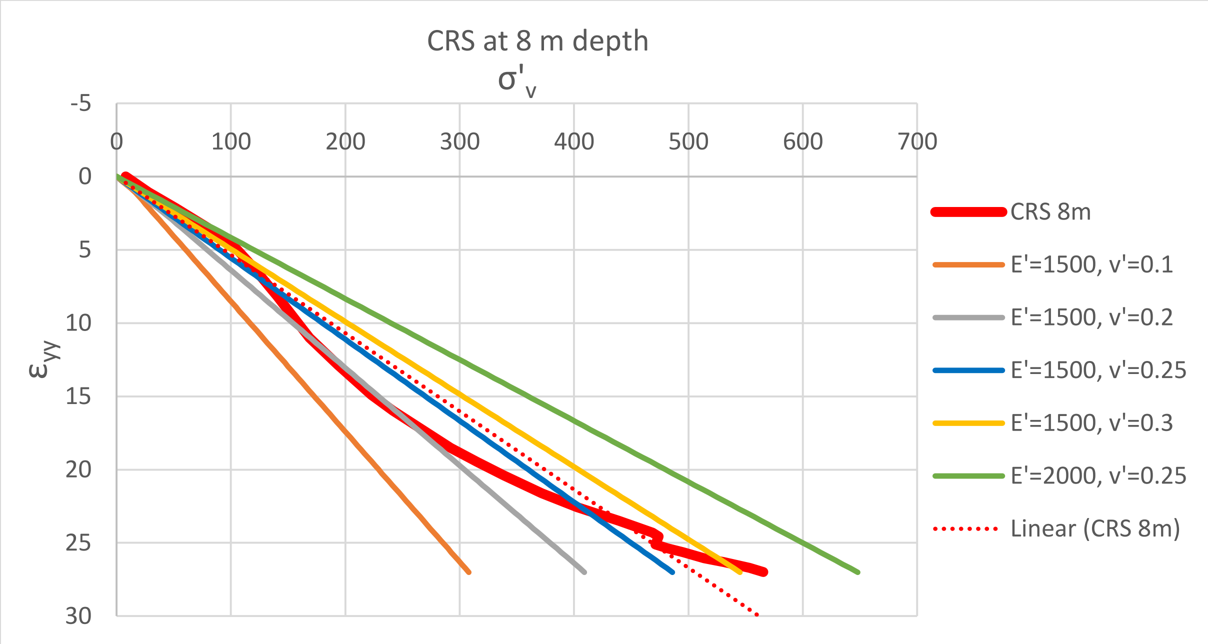
Pile volume installed before 17$^{th}$ of January: 1247.4 metres
Pile volume installed in total: 3291.7 metres

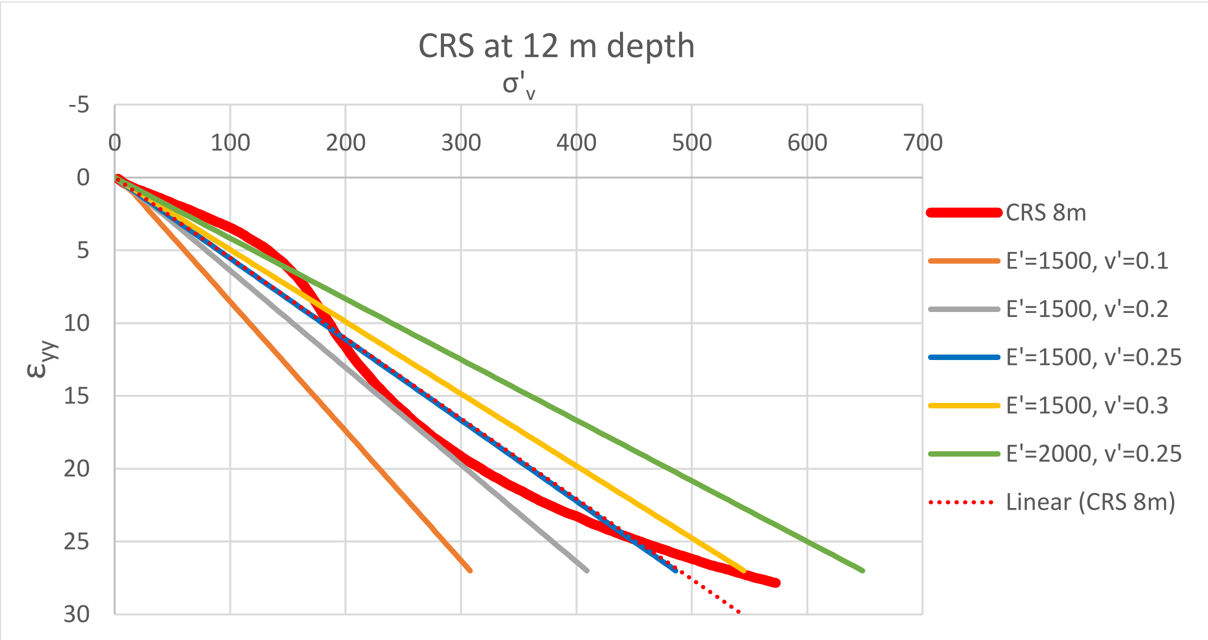
Percentage installed volume before 17$^{th}$ of January: 1247.4 / 3291.7 = 37.9 %

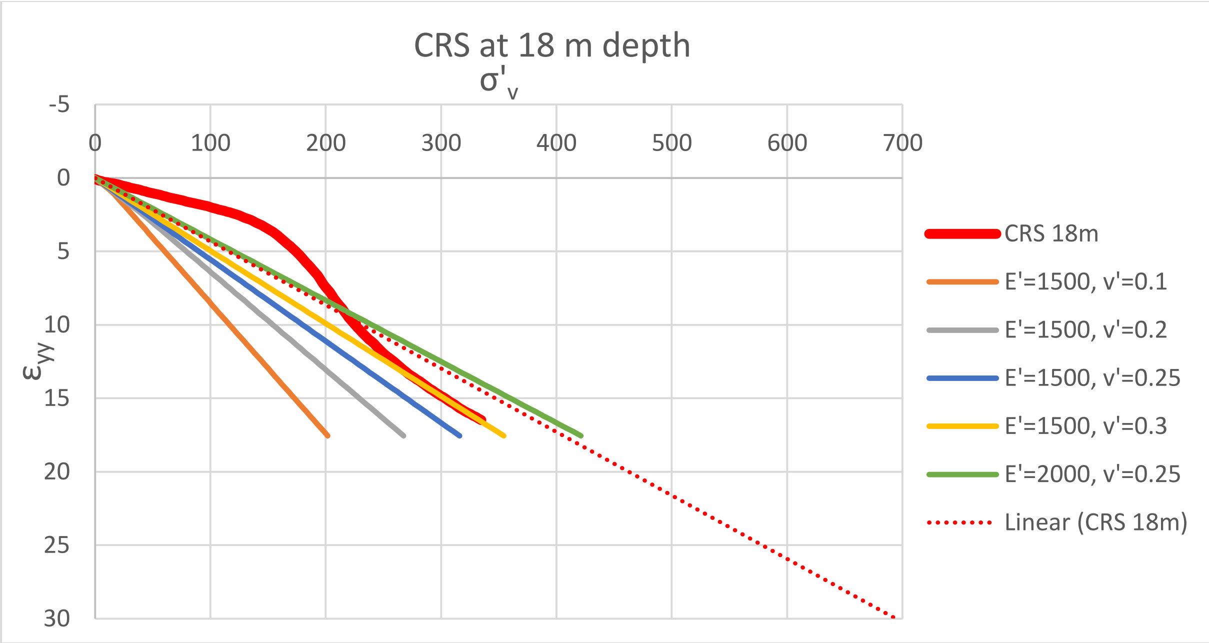
Recalculation was performed accordingly: $corrected\ value = \frac{measured\ value}{(1-0.379)}$

# Appendix 6 – CRS-test evaluation

Evaluation of stiffness parameters through comparison of CRS-tests performed at different depths in one of the bore holes nearby section 100/130.





CRS at 18 m depth
σ'v
εyy
CRS 18m
E'=1500, v'=0.1
E'=1500, v'=0.2
E'=1500, v'=0.25
E'=1500, v'=0.3
E'=2000, v'=0.25
Linear (CRS 18m)

# Appendix 7 – Calculation of Gamlestadstorget pile group -13

## Input parameters and calculation

**Piles**
Number of piles: 105
Total installed length of piles: 3,291.7 m
Average length of piles: 3,291.7 m / 105 piles = 31.35 m
Cross sectional area per pile: 275 x 275 mm = 0.0756 $m^2$

Total volume of piles: 3,291.7 x 0.0756 = 248.9 $m^3$

**Piling area**
Length: 59 m
Width: 18.5 m
Area: 59 x 18.5 = 1091.5 $m^2$

Total soil volume of piling area: 1091.5 * 31.35 = 34218.5 $m^3$

Percentage installed piles of total piling area: 248.9 / 34218.5 $m^3$ = 0.73%

| **Input data** | | | | | |
|---|---|---|---|---|---|
| Area piling area: | 1091.5 | $m^2$ | Volume piling area: | 34218.5 | $m^3$ |
| Area of piles: | 7.941 | $m^2$ | Volume piles: | 248.9 | $m^3$ |
| Area of piles + piling area: | 1099.4 | $m^2$ | Volume pre-auger: | 79.4 | $m^3$ |
| Average length of piles: | 31.35 | m | Volume of 50% pre-auger: | 39.7 | $m^3$ |

| Volume expansion effect of installed pile volume | Width w/o piles [m] | Width w/o piles/2 [m] | Width w. piles [m] | Δwidth [m] | Δwidth/2 [m] | Volumetric strain [%] | Volumetric strain for 100% pre-auger [%] | Volumetric strain for 50% pre-auger [%] |
|---|---|---|---|---|---|---|---|---|
| 100% | 18.5 | 9.25 | 18.635 | 0.1346 | 0.0673 | 0.730% | 0% | 0.365% |
| 75% | 18.5 | 9.25 | 18.635 | 0.1010 | 0.0505 | 0.546% | 0% | 0.273% |
| 50% | 18.5 | 9.25 | 18.635 | 0.0673 | 0.0337 | 0.364% | 0% | 0.182% |
| 25% | 18.5 | 9.25 | 18.635 | 0.0337 | 0.0168 | 0.182% | 0% | 0.091% |

# Appendix 8 – Calculation with Rehnman's method

## Input parameters and calculation

**Installed piles:**
Number of piles: 105
Width of each pile: 0.275 m
Average pile length: 31 m

**Pre-boring:**
Width of pre-boring: 0.275 m
Length of pre-boring: 10 m
Effectivity of pre-boring: 0-1
Pre-bored volume: 0.756 $m^3$
Number of pre-bored piles: 0

**Calculated volumes:**
Volume of piles: 246.2 $m^3$
Volume of pre-boring: 0.0-79.4 $m^3$

**Geometry of piling area:**
Length (l): 59 m
Width (b): 18.5 m

**Heave factors:**
$\alpha$: 1.0
$\beta$: 1.0
$\gamma$: 1.0
$\delta$: 1.0

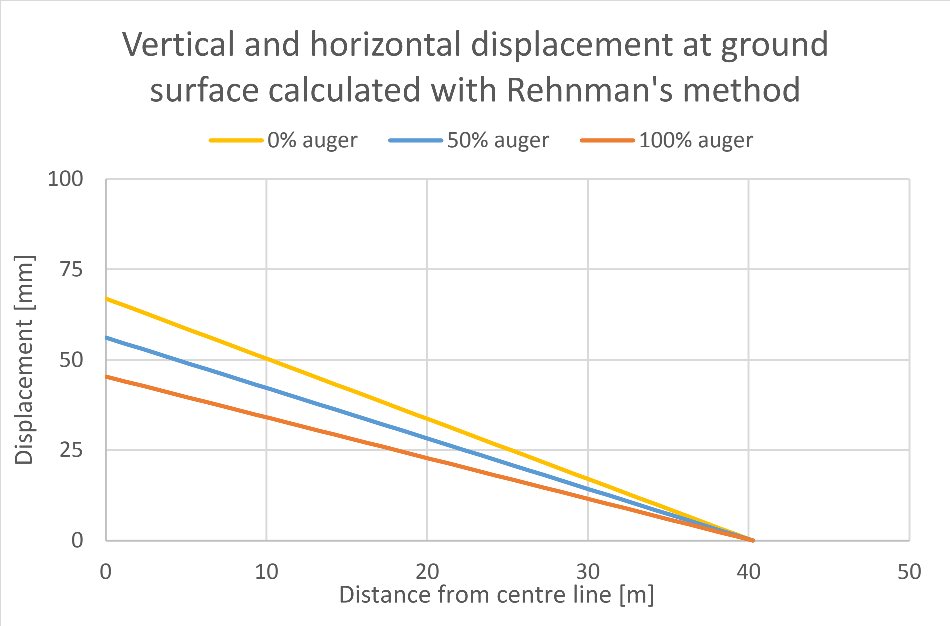
**Effectiveness factor:**
$\eta$: 1.0

**Calculated heave at edge of piling area (9.25 m from centre line)**
x = 0.0515 m
x = 51.55 mm

**Calculated heave at centre line of piling area (-9.25 m from edge)**
x = 0.0669 m
x = 66.93 mm

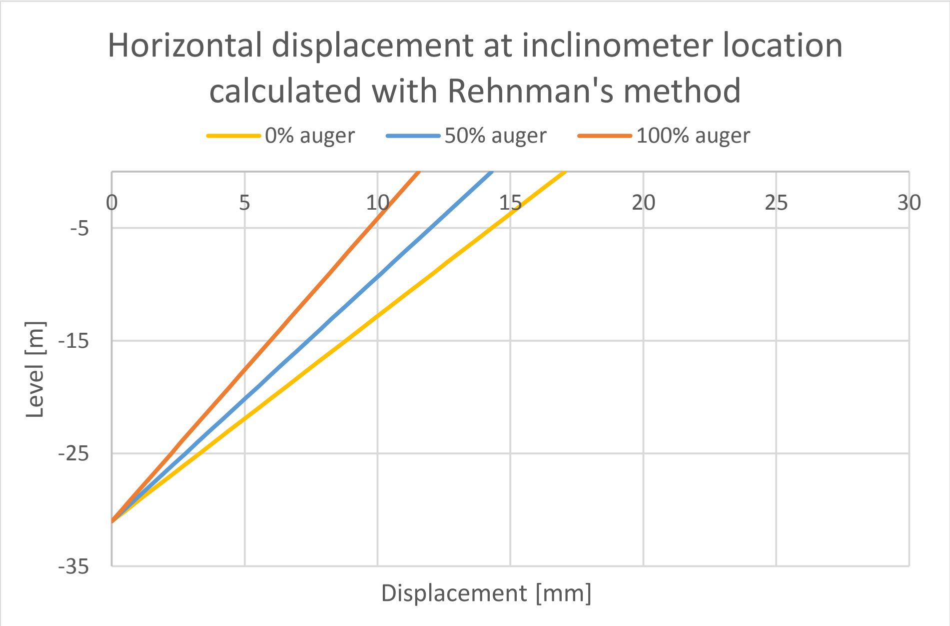
**Ground surface displacements**

The horizontal and vertical movement at the ground surface are equal to each other when calculated using Rehnman's method. Calculated using different degrees of effectiveness for the pre-augering.




**Horizontal displacement at inclinometer location**

Calculated using different degrees of effectiveness for the pre-auger.



# Appendix 9 – Calculation with Sagaseta's method

## Input parameters and calculation

**Installed piles**
Number of piles: 105
Average length: 31 m
Cross sectional area per pile: 0.0756 $m^2$

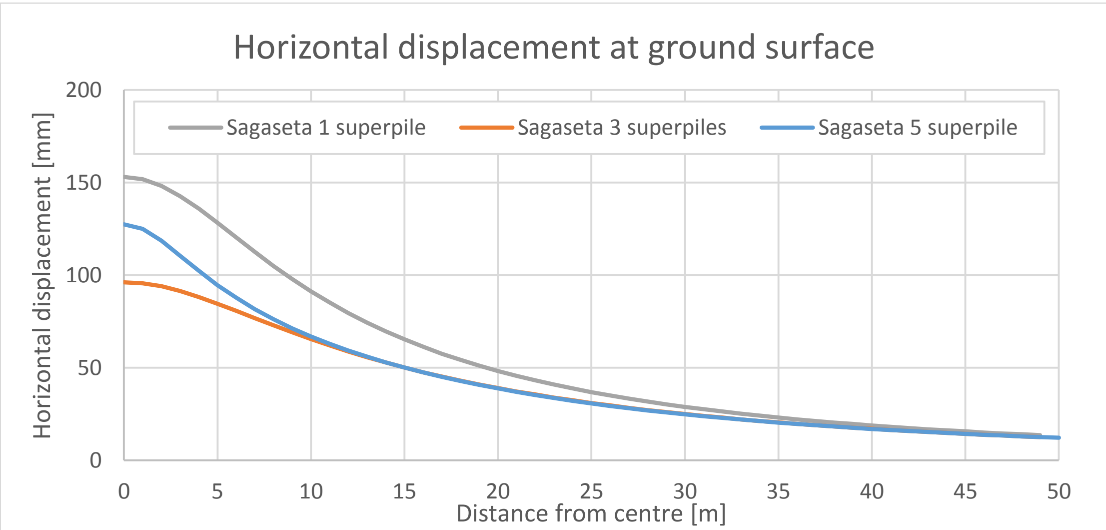
**Pile radius calculation**
Area of circular pile: $A = \pi r^2$
Radius of circular pile: $r = \mathrm{sqrt}(A/\pi)$

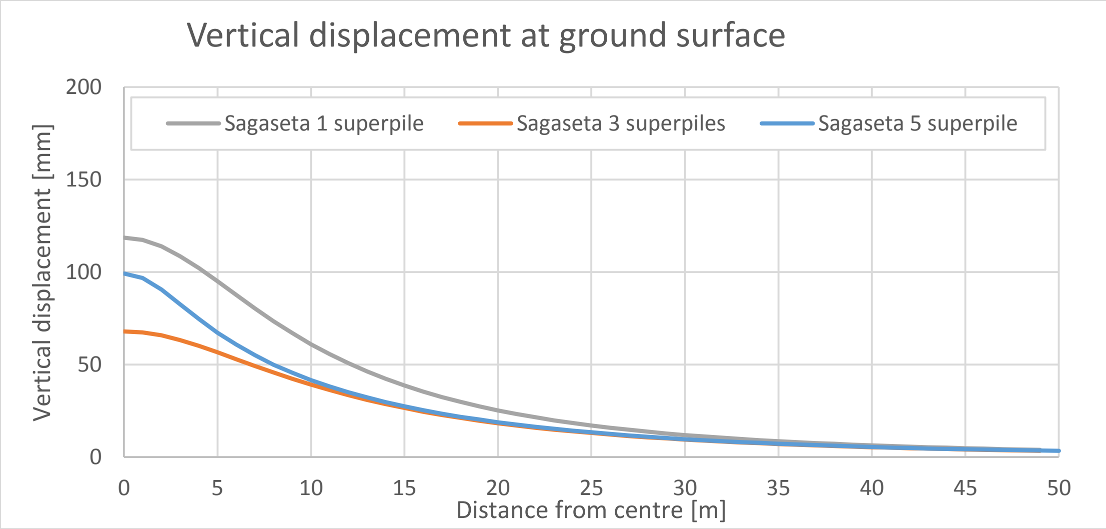
Area of 1 super pile: 7.941 $m^2$
Pile radius 1 super pile: 1.590 m

Area of 3 super piles: 2.647 $m^2$
Pile radius 3 super piles: 0.918 m

Area of 5 super piles: 1.588 $m^2$
Pile radius 5 super piles: 0.711 m

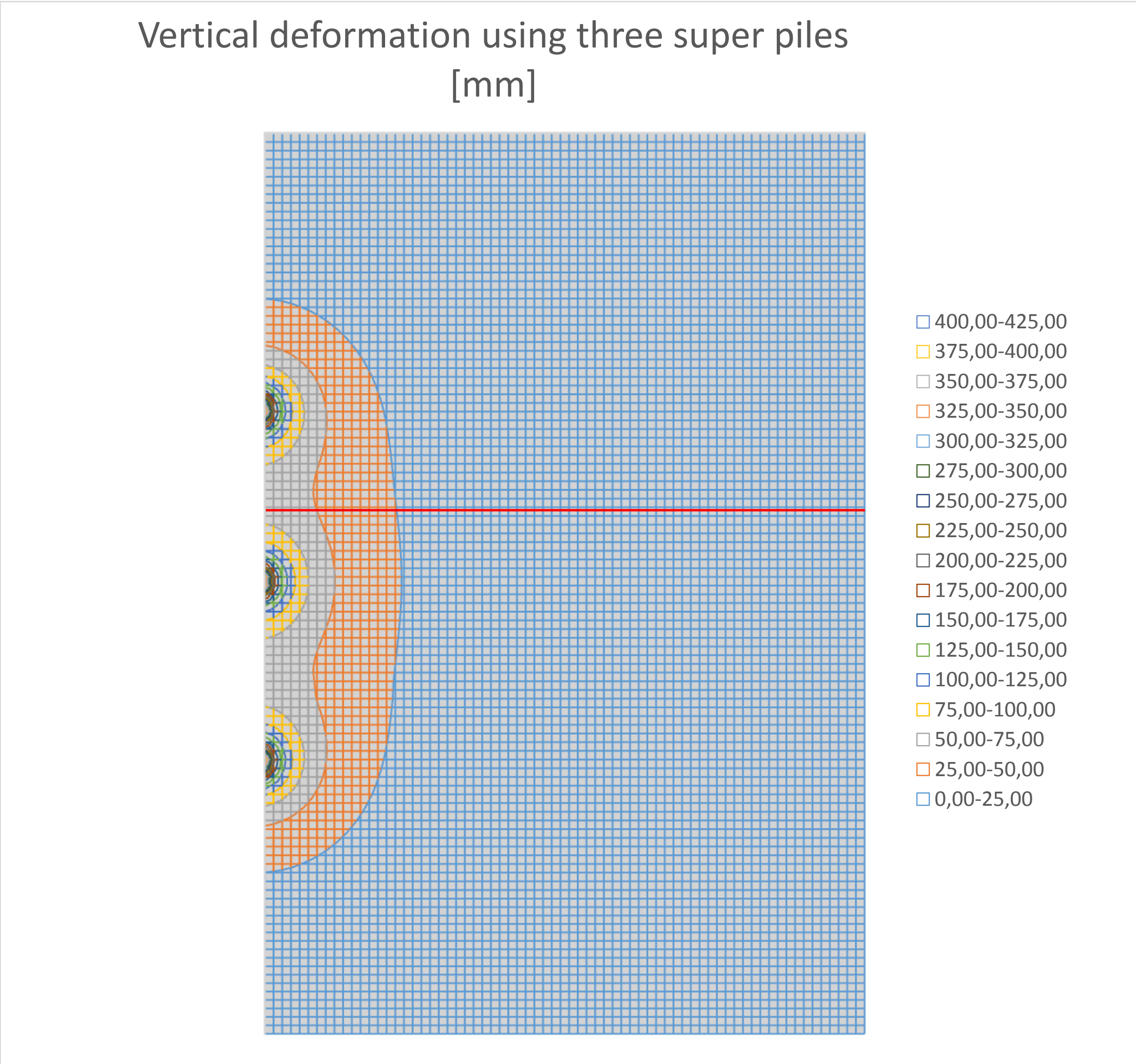
**Results in sectional graphs for three different super pile approximations**
Three different simplifications was tried with Sagaseta's method as seen below. A calculation with each individual pile would be the more exact but take more time, a good approximation is to use a set of super piles. The more super piles to be used should give a more accurate result. However, the reason that the three super pile solution gives a lower initial value than the five pile solution is because one of the super piles are located closer to the measured section in the five pile solution, thus giving a larger initial value.






**Results in Cartesian coordinate plane for calculation with three super piles**

Calculation performed with three super piles. The red line represents the section of interest.



## Skanska's method using Sagaseta for calculating displacement towards depth

### Markrörelser och rörelser mot djupet vid installation av pålar

(Ekvationer hämtade ifrån Sagaseta, Whittle, Santagata 1997):

Pålning för påldäck 13:
Avstånd inklinometer I:1 till närmsta massundanträngande pålrad i Pd13 är ca 20 m. I aktuell analys ansätts pålarna förenklat som 6 st skivor med ett inbödes avstånd på 3,5 m totalt till ett avstånd på 17,5 m från bef pålar.

Skivdeformationen w viktas för att matcha resultatet av utförda kontrollmätningar (inklinometer I:1 max ytdeformation ca 25 mm då pålning för Pd13 avslutades i början av februari 2016). Nyttoeffekten av proppdragningen i beräkningarna har satts till 50% baserat på erfarenhet från tidigare projekt (Gamlestan spårskede 2 och 3, Södra Marieholmsbron, m.fl.).

$w := 0.0089$ $L := 25$ $x_1 := 3.5$ $w_{propp} := 0.50 \cdot w$ $L_{propp} := 10$ (L 25 m ca medellängd pålar i läge vinkelrät inklinometer I:1)

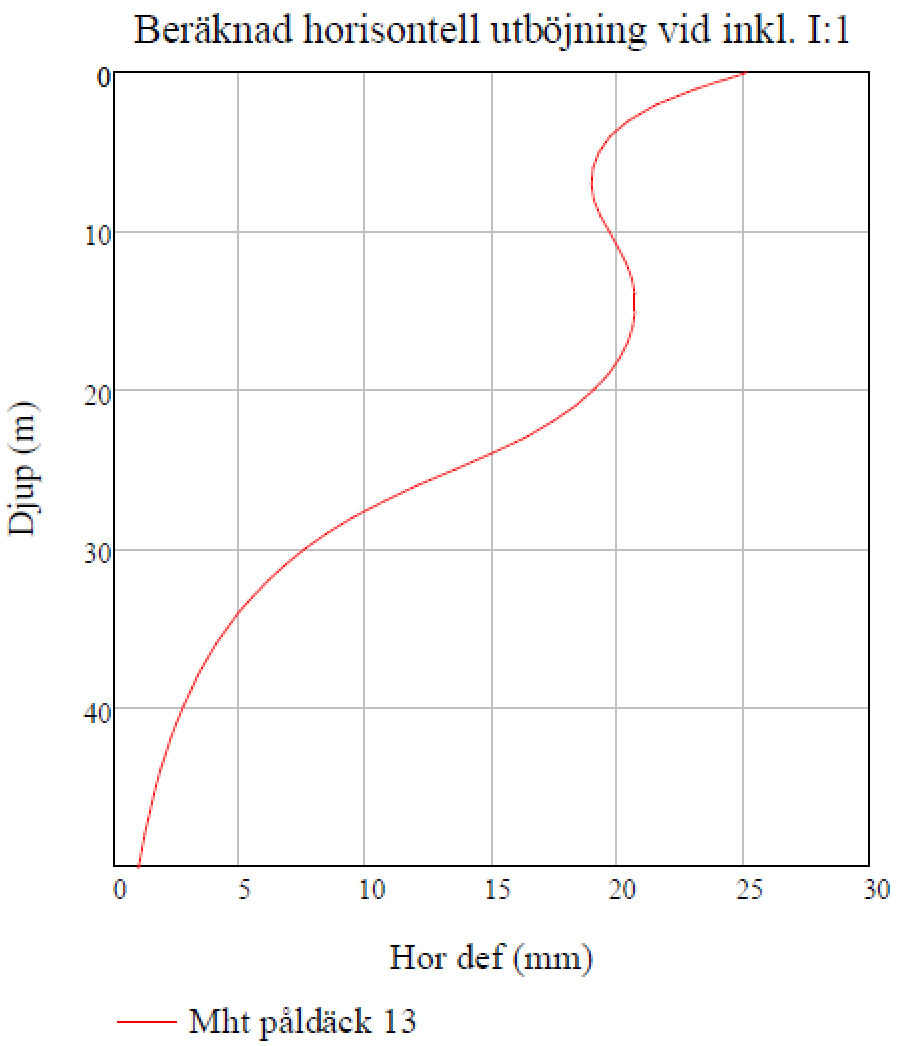
$x_2 := 7.0$

$x_3 := 10.5$

$x_4 := 14.0$

$x_5 := 17.5$

$x_6 := 21.0$


Beräknad horisontell utböjning vid inkl. I:1
Djup (m)
0
10
20
30
40
0
5
10
15
20
25
30
Hor def (mm)
Mht påldäck 13